Strength Characterization of Soft Marine Deposits off East Africa Using the CPT-Stinger Method

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ABSTRACT: The CPT-Stinger was recently developed for in-situ investigation of very soft marine soils. This tool is a deep-water, piezocone penetrometer testing (PCPT) system that is dynamically inserted into the seabed using conventional Jumbo Piston Coring (JPC) deployment techniques. A series of nineteen CPT-Stinger tests were performed in the vicinity of a new gas field development located off east Africa (EA) in water depths ranging from 1395 m to 1575 m. Soils in the study area are characterized by high liquid limits and carbonate content in excess of 20%. This paper discusses the evaluation of undrained shear strength (S_u) profiles derived from the cone tip resistance (q_c) and measured excess pore-pressure (u_2) by means of the cone factors N_{kt} and $N_{\Delta u}$. Cone factors were determined by comparing CPT-Stinger results with undrained shear strengths obtained from miniature vane (MV) tests performed on undisturbed JPC samples collected at nearby locations. An average N_{kt} value of 28, with a range of 18 to 37, fits the EA soils. These high values are possibly due to inaccuracies in the method used to calibrate the Stinger cone tip readings within the "dynamic" interval. It is shown that the pore pressure method using $N_{\Delta u}$ gives a closer prediction of the undrained shear strength of EA clay compared to that using N_{kt}. Over-consolidation Ratio (OCR) profiles were also developed with data collected from this tool. Results compare very well with OCR data obtained from conventional Constant Rate of Strain (CRS) tests performed on high quality samples. Shear strength profiles based on SHANSEP concepts were also determined and compared with those determined by means of cone factors N_{kt} and $N_{\Delta u}$. The findings confirm that a high degree of reliance can be given to pore-pressure measurements for estimating S_u for offshore testing. Comparisons also demonstrate that the CPT-Stinger is a reliable tool that can be used for site investigation of very soft deep water marine soils.

INTRODUCTION

The strength characterization of soft deep water sediments can be difficult to assess through conventional drilling and sampling methods. *In situ* tools such as the piezocone penetration test, flat dilatometer and vane shear test are conventional methods that are not prone to sample disturbance issues. For marine applications, however, these methods have limitations and can become expensive alternatives as water depth increases. The CPT-Stinger is increasingly used in offshore site investigations to measure in-situ properties of very soft marine sediments. The CPT-Stinger is a deepwater, PCPT system that is dynamically inserted into the seabed using conventional jumbo piston core (JPC) deployment techniques that can be deployed from non-drilling vessels.

The fundamental advantage of a PCPT lies in the continuous assessment of soil properties. It also provides significant insight into the *in situ* conditions of calcareous soils where the degree of cementation can be misinterpreted or overlooked due to sample disturbance during conventional sampling and associated laboratory testing (Beringen et al. 1982). There is an increasing appreciation for the simplicity of the CPT-Stinger, with the test becoming popular for use in offshore site investigations throughout the world. The aim of this paper is to present examples that demonstrate advantages of the system and to discuss engineering interpretation of tests results. Typical results of CPT-Stinger testing in soft marine, predominately calcareous, sediments located off east Africa are illustrated. Interpretation of Stinger-CPT data is juxtaposed with test results performed on undisturbed JPC samples collected at nearby locations.

CONE PENETRATION TESTING: CPT-STINGER

The CPT-Stinger provides significant insight into the in situ conditions of soft marine deposits. The CPT-Stinger has been widely described in the literature (e.g., Young et al. 2011; Jeanjean et al. 2012). A short description of the test, main components and interpretation is provided below.

The CPT-Stinger is designed to operate as a standard cone as described by Lunne et al. (1997) measuring cone tip resistance (q_c), sleeve friction (f_s), and induced penetration pore-pressure (u_2) in accordance with ASTM standards (ASTM 2012). The cone used has an internal memory which stores cone data acquired during dynamic penetration and standard penetration rate. Deployment and triggering of the CTP-Stinger is very similar to the operation of a JPC. Thus, after proper installation of the CPT-Stinger system in the JPC weight-head and deployment from the vessel, the system is triggered near the seabed (approximately 1.0 m above the seafloor) and is allowed to free-fall/penetrate ballistically into the seafloor. Free-fall cone data is acquired during the initial 3-second ballistic penetration into the seafloor at cone velocities approaching 9 m/sec (Young et al. 2011). Dynamic cone data, represented by the interval through which the Stinger vastly exceeds the 2 cm/sec penetration rate, is collected as the cone ballistically penetrates into the seafloor. After complete penetration into the seafloor, the embedded JPC barrel and weight-head serve as a reliable reaction mass for pushing the CPT cone. During this phase, the cone is pushed deeper into the formation at a controlled rate of penetration of approximately 2 cm/sec in general accordance with ASTM standards. Standard cone data is collected during this controlled-rate push. Dynamic data for tip resistance, sleeve friction, and pore pressure collected during ballistic penetration are individually corrected for the velocity effect. To accomplish this, the dynamic values for tip resistance are individually adjusted using a user-selected discount factor between 0 and 30% per base-10 logarithm of velocity. Dynamic values for Sleeve Friction and Pore Pressure are likewise adjusted using their own discount factors (Young et al. 2011).

STUDY AREA AND SUBSURFACE CONDITIONS

A reconnaissance geotechnical investigation was performed to acquire geotechnical and geological data as required to define the soil conditions of a deepwater development located off east Africa in water depths ranging from 362 m to 1578 m. The geotechnical program consisted of Box Cores (BC), Piston Cores (PC), Jumbo Piston Cores (JPC), and *in situ* Piezocone Penetrometer Tests (PCPT). Sample locations were sited to meet a number of objectives including: reconnaissance site characterization of the field development area, assessing potential constraints to field development activities, establishing environmental baseline information, and to provide preliminary information to support engineering design.

In general, the geological structure and stratigraphy in this portion of east Africa is complex, reflecting active tectonics and major shifts in the Rovuma River delta's depocenter along with fringing carbonate reefs. Soil types encountered range from very soft to firm high plasticity clay, to sandy clay and clayey sand and various ranges of each. Shell and coral fragments were periodically present in recovered soil samples, in addition gravel and rock fragments were recovered from a few samples.

For this study, a total of nineteen CPT-Stinger tests were performed collecting data continuously from the seafloor to penetrations ranging from approximately 24.9 m to 29.1 m below mudline (BML). Data was used to evaluate the shear strength of the cohesive soils and to aid with soil classification and stratigraphy definition.

STANDARD AND ADVANCED LABORATORY TESTING RESULTS

Soil Index and Physical Soil Properties

To identify the pertinent index and engineering properties of the soils a comprehensive onshore laboratory testing program was conducted on select samples from JPC collected nearby the subsequently discussed CPT-Stinger locations. The index property tests included moisture content, Atterberg limits, unit weight, specific gravity, carbonate content and organic content tests. The strength tests included undisturbed and remolded motorized miniature vane (MV) strength tests together with Unconsolidated Undrained (UU) Triaxial tests.

The variation of moisture content as a function of depth below seafloor is presented in Figure 1a. Test results indicate that the moisture content ranges from 60 to 129 percent in the top 5 meters, and from 55 to 75 percent from this depth to the final penetration, 16.45 m approximately. The variation of the Liquid Limit and Plastic Limit with depth is shown in Figure 1b. Liquid limits vary from about 24 to 88 percent and plastic limits vary from 17 to 65 percent. To aid in the visual identification and classification of the soils plasticity indices were plotted in the plasticity chart (Figure 2). Examination of the data reveals that samples recovered from the nearby JPC cores are predominately high plasticity (CH) clays. There are also a few low plasticity (CL) clays, as well as some elastic silt (MH) and organic (OH) clays. It is recognized that liquid and plastic limits, together with the natural moisture content, are typically useful indicators of the stress history and undrained shear strength of soils. Thus, Liquidity Index values are presented on Figure 1c. Data suggest the clays appear to be normally to lightly over-consolidated.

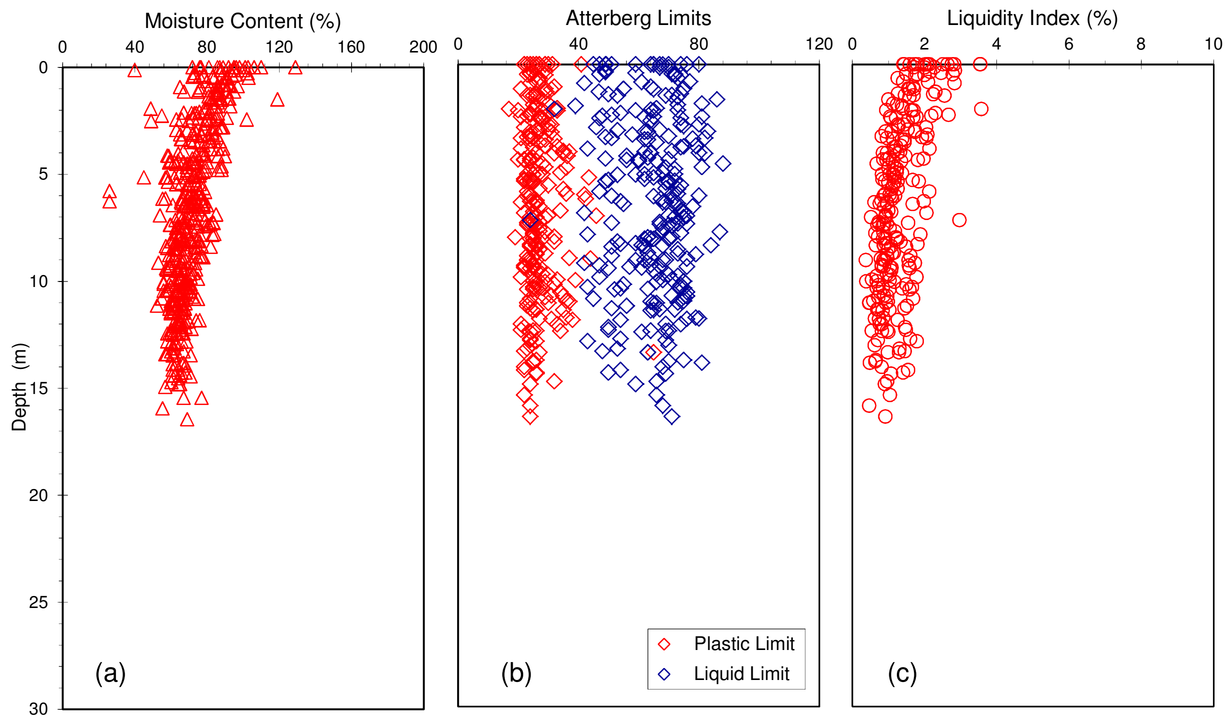

Figure 1: (a) Moisture content; (b) Atterberg Limits; (c) Liquidity Index

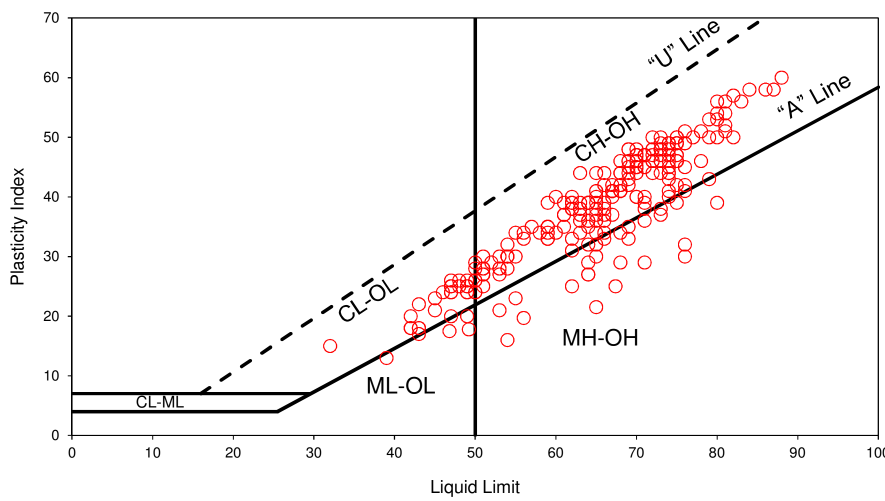

Figure 2: Plasticity Chart

The theoretical submerged unit weights are plotted on Figure 3a together with their corresponding submerged unit weights computed from the total unit weights of the soil samples measured in the laboratory. The theoretical values were calculated from moisture content and specific gravity with the assumption that the soils are 100 percent saturated *in situ*. A typical specific gravity value for the study area of 2.73 was used in the calculations. Carbonate content measurements are presented in Figure 3b. The test results indicate that the carbonate content of the soil samples ranges from 33.3 to 76.7 percent.

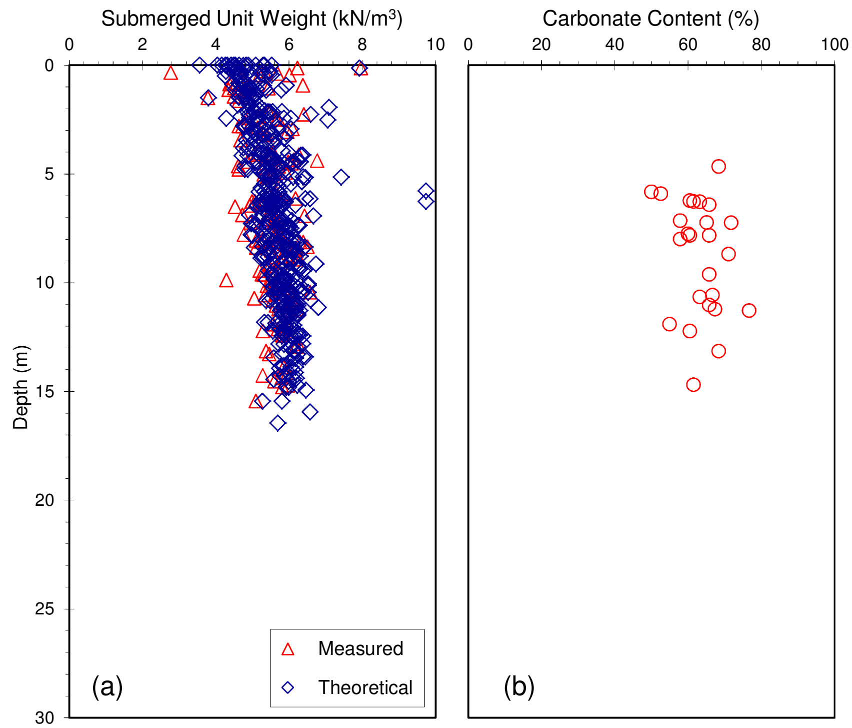

Figure 3: (a) Submerged unit weight; (b) Carbonate content

Consolidation and Static Simple Shear Strength Characteristics

To investigate the soil deposit's stress history (pre-consolidation stress) and compressibility characteristics 27 Constant Rate of Strain (CRS) tests were performed on selected high-quality cohesive specimens from JPC cores.

The pre-consolidation stress (σ_p') was determined from the classic method proposed by Casagrande (1936) using the empirical construction from the void ratio (e) versus log effective stress (σ') and the Work Per Unit Method proposed by Becker et al. (1987). The data were reviewed to evaluate the potential sample disturbance using the procedure proposed by Lunne et al. (1997). The procedure suggests that the computed values of change in void ratio to reconsolidate the sample to its in situ vertical effective stress(σ_{vo}') versus the change and initial void ratio ($\Delta e/e_0$), designated the Sampling Quality Designation (SQD), can be used to quantify sampling disturbance.

Figure 4 shows the average of the over-consolidation ratios (OCR) determined from the methods described above compared with their corresponding SQD. It can be seen that higher values of SQD are typically associated with lower values of OCR, which is an indicator of sampling disturbance.

Pre-consolidation stresses at a given depth together with the computed in-situ vertical stress profile are plotted in Figure 5. Data points in Figure 5 are grouped by level of disturbance with the open symbols having high SQD values. The figure shows some pre-consolidation stress values are slightly lower than the in-situ vertical stress suggesting an apparent under-consolidated soil. However, the Liquidity Index (LI) at the depth of the tests (LI from 0.9 to 0.93) indicated that the soils should not be under-consolidated. In general, LI of 1.0 or more is representative of very soft unconsolidated soil, whereas a value close to 0.0 is an indication of a very stiff over-consolidated soil. Thus, the apparent under-consolidation is probably a result of sampling disturbance. The specimen at 8.75 m in JPC-P18 has a very high OCR (average 14.9), which is probably from cementation as the specimen had a carbonate content of 71.1 percent.

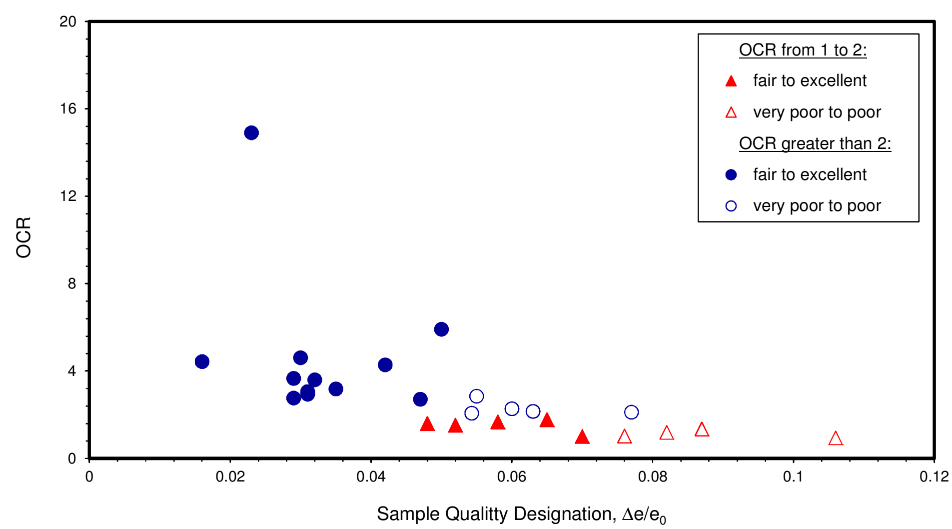

Figure 4: Relationship between OCR and SQD

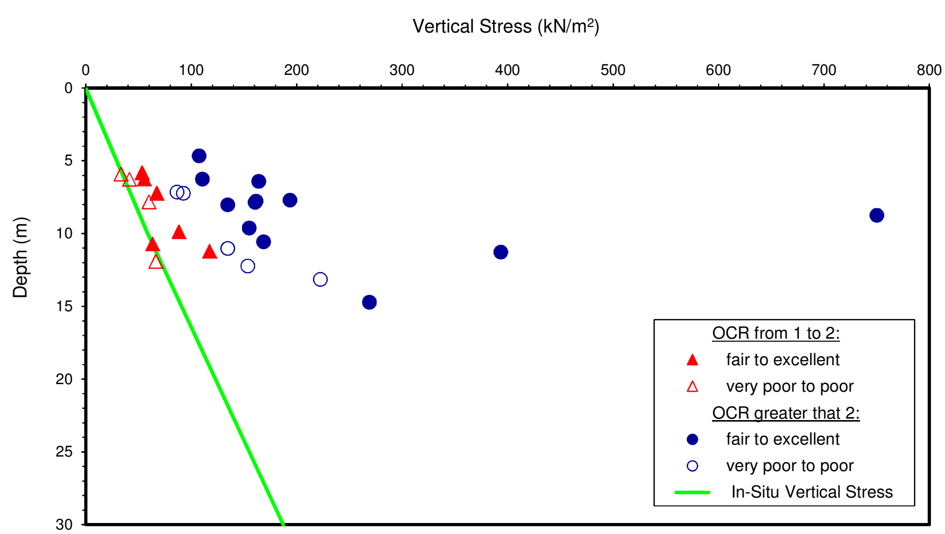

Figure 5: Derived (σ'_p) and (σ'_{vo})

Direct Simple Shear (DSS) tests were also performed on selected high-quality cohesive specimens to determine the normalized shear strength parameters to further evaluate the static undrained shear strength of the soil (S_u). DSS testing was conducted on samples consolidated to approximate in-situ stresses (σ'_{vo}) as well as on samples consolidated to laboratory induced normally consolidated and overconsolidated stresses in accordance with the SHANSEP (Stress History and Normalized Soil Engineering Properties) methodology proposed by Ladd and Foott (1974). The goal of the program was to confirm the appropriateness of the SHANSEP concept in these EA soils, thus allowing mitigation of sampling and specimen preparation disturbance effects detrimental to shear strength interpretation.

Typically, the first step in the SHANSEP methodology calls for the consolidation of each test specimen to an induced over-consolidation ratio (OCR) of either one (1) or greater. An induced OCR of 1 is obtained by consolidating the specimen well into the virgin consolidation region by applying a stress level greater than the pre-consolidation stress (σ'_p) by a factor of about 1.5 to 3.0 to assure an axial strain of at least 12%. In the second step the sample is sheared to obtain normalized undrained stress-strain and shear strength parameters (S_u/σ'_c). For subsequent DSS tests at higher OCR, soil specimens from the same core segment were consolidated to the vertical stress applied to the test specimen at OCR=1, and then unloaded to the assigned OCR (2 or 4). This ensured that all tests were normally consolidated to the same stress. Each increment of vertical stress, except the last increment, was allowed to remain on the sample until 100 percent consolidation (t_{100}) was reached. The final vertical stress remained on the sample for 24 hours past t_{100}. After this, the samples were sheared at a strain rate of about 5% per hour.

A total of three (3) series of DSS tests with three (3) tests per series (at induced OCR = 1, 2 and 4) were conducted for the study. An additional DSS test designated (D) in Table 1, where the specimen was consolidated only at an induced OCR=1 was also completed. The normalized shear strength ratio for DSS tests is expressed as $\left(S_{u,DSS}/\sigma'_c\right)_{nc}$ when tests are conducted at OCR = 1.0. Thus, the normalized DSS shear strength ratios from the four (4) DSS tests conducted at OCR = 1.0 are presented in Figure 6, and below in Table 1 where S denotes $\left(S_{u,DSS}/\sigma'_c\right)_{nc}$. As can be seen, the peak normalized shear strength data varies between 0.237 and 0.281. A $\left(S_{u,DSS}/\sigma'_c\right)_{nc}$ value of 0.2592 was adopted for the EA clayey soils.

Table 1: Normalized Shear Strength Ratio for DSS Tests at OCR=1

DSS specimen	S
A	0.265
B	0.237
C	0.281
D	0.252

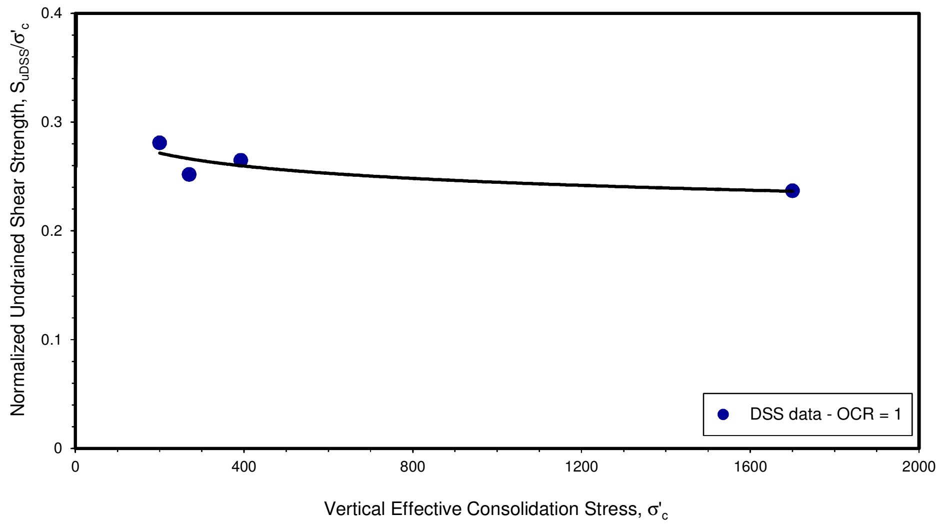

Figure 6: Normalized Shear Strength with a lab-induced OCR of 1.0

Figure 7 shows a plot of the test results for the respective OCR and normalized strength ratio. From the figure, the tests show that the strength rebound exponent m, which relates the over-consolidated state to the normalized shear strength ratio, may be approximated by 0.86. Ladd and DeGroot (2003) recommended $S = 0.25$ with a standard deviation of 0.005 (for simple shear loading) and $m = 0.8$ for most soils. Thus, it can be seen that the S and m values of 0.2592 and 0.86, respectively, align with the values suggested by Ladd and DeGroot (2003).

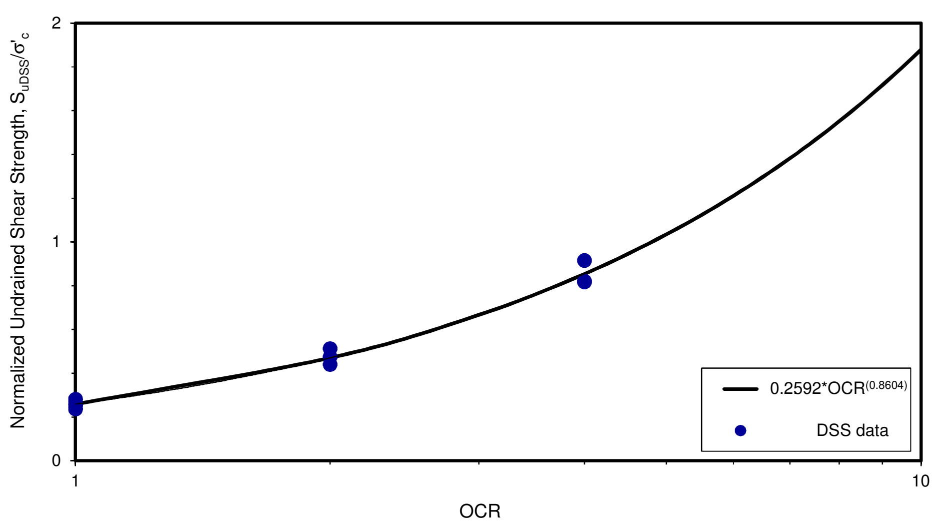

Figure 7: Direct Measurement of SHANSEP m-value from DSS Testing

CPT-STINGER DATA AND INTERPRETATION

Undrained Shear Strength Relations

The measured values of cone tip resistance (q_c), sleeve friction (f_s), and excess pore pressure (u_2) with depth (m) provide important information for interpreting soil properties. Thus, *in situ* undrained shear strength values, (S_u), were determined using the conventional relationship between (S_u) and net cone tip resistance (q_{net}), expressed as follows (Lunne et al. 1997)

$$S_u = \frac{q_{net}}{N_{kt}} = \frac{q_t - \sigma_{vo}}{N_{kt}} \qquad (1)$$

Where S_u is in-situ undrained shear strength, q_{net} is net cone tip resistance, q_t is total cone tip resistance (this includes corrections for pore pressure effects and cone shape using a net area ratio $a_n = 0.8$), σ_{vo} is total vertical overburden stress at the depth of penetration, and N_{kt} is empirical cone factor that is analogous to a bearing capacity factor

The N_{kt} factor is conventionally determined by relating the net tip resistance (q_{net}) of the CPT to in-situ strength data and laboratory measured shear strength data on soil samples obtained at nearby locations. Undrained shear strength profiles developed with the N_{kt} factors presented in Table 2 with the corresponding measured miniature vane data (MV) for three example CPT-Stinger profiles are shown in Figure 8. It can be seen that the prediction of the undrained shear strength using the N_{kt} approximation when comparing with the measured MV shear strength values presents significant scatter mainly below 10 m. Some of the scatter with the measured MV shear strength values is probably due to disturbance of the specimens. It also calls to attention the wide range of the empirical N_{kt} values selected for these CPT soundings (from 21 to 28). In general, the overall best-fits N_{kt} for the MV strength data for all the 19 CPT tests ranged between 18 and 37.

Table 2: Empirical Cone Factors N_{kt} and $N_{\Delta u}$ for S_u predictions shown in Figure 8

CPT ID	N_{kt}	$N_{\Delta u}$
CPT-01	28	8
CPT-04	21	7
CPT-05	23	7

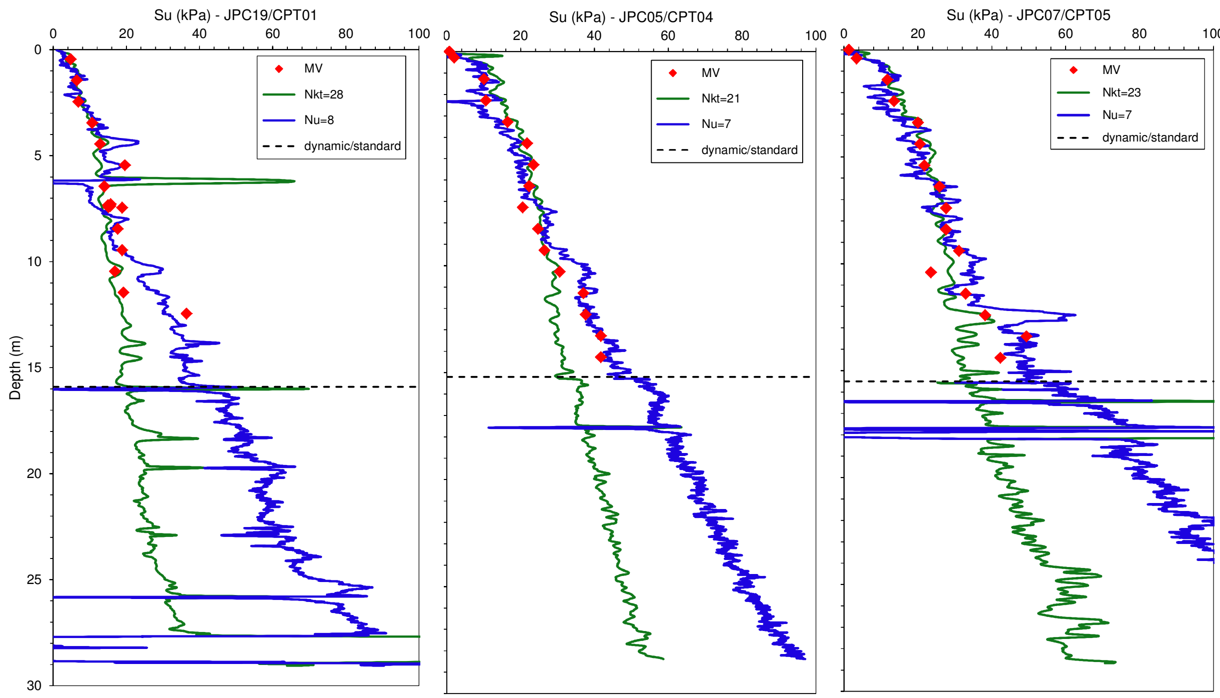

Figure 8: Undrained shear strength profiles derived from cone tip resistance (q_c) and measured excess pore pressure (u_2) by means of the cone factors N_{kt} and $N_{\Delta u}$

The discrepancy in the empirical and theoretical N_{kt} values has long been recognized. No single value of N_{kt} covers all types of clays, type of cone, field conditions, penetration rate and laboratory testing method (Lunne et al. 1997; Low et al. 2010; Alshibli et al. 2011). However, it seems likely that for a given clay deposit, CPT, and test condition there is a more unique relationship between net cone tip resistance (q_{net}) and shear strength then suggested by the Stinger CPT results (Lunne et al. 1979; Stark and Delashaw 1990). Lunne et al. (1997) and others have shown that normally consolidated marine clays with field vane as the reference test suggest the empirical cone factor N_{kt} varied between 10 and 20 with an average value of 15. Kjekstad et al. (1978) using triaxial compression tests as the reference strength found that N_{kt} was 17 for non-fissured over-consolidated clays. N_{kt} values up to 30 have also been reported in the literature, especially when scale effects were considered, in stiff fissured clays with triaxial compression tests as the reference strength (Powell and Quarterman 1988; Terzaghi et al.1996). These findings suggest the wide range and high values of the empirical coefficient N_{kt} found for the EA clays, given the uniform soil conditions and generally increasing soil strength with depth from very soft to firm, are beyond the expected range.

It is suggested that these high N_{kt} values are due to the method used for correcting the dynamic tip resistance data during the ballistic interval. As described previously, the logarithmic method was used by the acquisition company as recommend by (Young et al. 2011). However, it is important to note that although this adjusting method has been successfully validated on many projects, mainly for the very soft marine clays in the Gulf of Mexico by overlaying the corrected dynamic data with existing standard CPT data at nearby sites (Young et al. 2011; Jeanjean et al. 2012), additional site-specific corrections might be required for more reliable values of N_{kt} in frontier areas where no previous experience is available. It is out of the scope of this paper to review the actual adjusting method or propose new methodologies.

The laboratory measured remolded shear strength (MV_r) versus depth for the three example CPT locations are shown in Figure 9. The measured CPT sleeve frictional resistance (f_s) for these locations is also shown in the figure. The figures show the strong correlation between the sleeve resistance and the undrained remolded shear strength. However, it is important to note that this type of correlation might not be dependable with other CPT systems (Lunne and Andersen 2007).

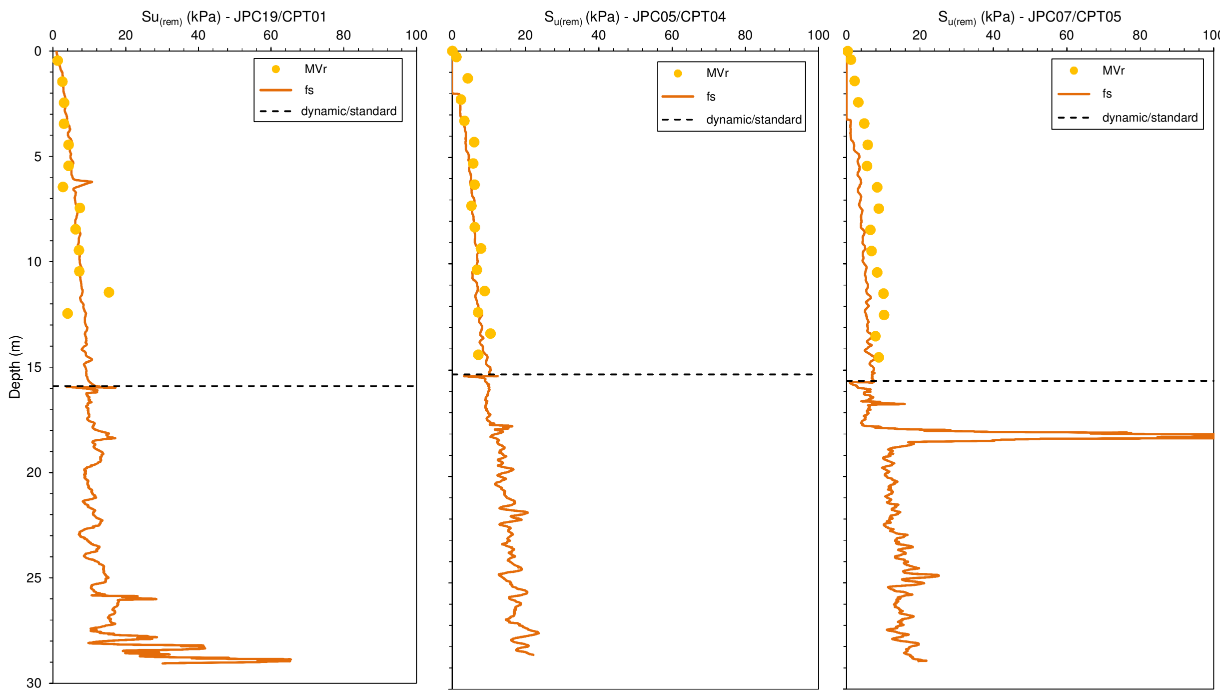

Figure 9: Remolded shear strength profiles compared to sleeve friction

In recent years, conventional CPT testing with pore-pressure measurements has become more widespread, and the high resolution of pore pressure measurements with less than 1% error (Campanella et al. 1985) are more suited to correlation with the low undrained shear strength of shallow marine deposits (Lunne et al. 1997). It has been documented (e.g., Robertson 2009 and 2012) that the PCPT pore-pressure measurements are almost always reliable in offshore testing due to the high ambient water pressure that ensures full saturation of the pore-pressure element, especially where the deposition is predominately soft marine clays. In order to investigate the accuracy of the N_{kt} values for the EA clays and validate the undrained shear strength profiles based on the equation (1), a new set of undrained shear strength profiles were independently evaluated based on the excess pore pressure measured behind the cone (u_2) using the following relationship (Lunne et al. 1997):

$$S_u = \frac{u_2 - u_o}{N_{\Delta u}} = \frac{\Delta u}{N_{\Delta u}} \qquad (2)$$

Where u_2 is the pore pressure measured between the cone and the friction sleeve, u_0 is the equilibrium pore pressure in situ, Δu is the excess pore pressure, and $N_{\Delta u}$ is the empirical pore pressure N-factor. According to Lunne et al (1997), $N_{\Delta u}$ is theoretically shown to vary between 2 and 20. La Rochelle et al (1988), using uncorrected field vane test results as the reference strength found that $N_{\Delta u}$ varied between 7 and 9, even if OCR ranged between 1.2 and 50. Karlsrud et al. (1988) obtained $N_{\Delta u}$values varying between 6 and 8 for normally to lightly over-consolidated clays. In very soft clays values of $N_{\Delta u}$ ranging between 7 and 10 are commonly reported (Lunne et al 1997).

The use of equation (2) allows for an independent determination of S_u profiles entirely from the pore-pressure measurements. According to Mayne (2008), if two independent profiles of undrained shear strength agree, the two profiles will support each other findings and then a higher degree of reliance might be afforded in the results.

For comparison purposes, the predicted values using equation (2) have also been plotted in Figure 8. In general, the PCPT predicted shear strengths using equation (2) correlate well with the MV test results with the cone factors $N_{\Delta u}$ shown in Table 2. The overall best-fits $N_{\Delta u}$ for the MV strength data for all the 19 CPT tests range between 7 and 12, which aligns with the $N_{\Delta u}$ range indicated by Lunne et al (1997) and others.

Since the S_u profiles using equations (1) and (2) do not agree, especially along the standard CPT data interval, undrained shear strength profiles were also estimated according to the SHANSEP concept as follows (see Ladd and Foott 1974):

$$S_u/\sigma'_{v0} = (S_u/\sigma'_{vo})_{OCR=1}\,(OCR)^m = S(OCR)^m \qquad (3)$$

Where S = 0.2592, and m = 0.86 as previously described. OCR profiles to be used in equation (3) were obtained from the hybrid cavity-expansion and critical-state theory proposed by Mayne (1991). In this formulation, OCR profiles can be established using either the total cone tip resistance (q_t) or the pore pressure (u_2) by the following expressions:

$$OCR_1 = 2\left[\frac{\left(\frac{2}{M}\right)(q_t - \sigma_{vo})/\sigma'_{vo}}{\frac{4}{3}(ln(I_r)+1)+\frac{\pi}{2}+1}\right]^{(1/\Lambda)} \qquad (4)$$

$$OCR_2 = 2\left[\frac{(u_2 - u_o)/\sigma'_{vo}}{\left(\frac{2}{3}M\right)\ln(I_r)}\right]^{(1/\Lambda)} \qquad (5)$$

Alternatively, Burns and Mayne (2002) proposed a third approximation that combines both q_t and u_2 as follows:

$$OCR_3 = 2\left[\frac{1}{1.95M+1}\left(\frac{(q_t-u_2)}{\sigma'_{vo}}\right)\right]^{(1/\Lambda)} \qquad (6)$$

Where ϕ'= effective friction angle expressed in terms of $M = 6sin\phi'/3 - sin\phi'$, I_r= undrained rigidity index, and $\Lambda = 1 - C_s/C_c$. Thus, 19 OCR profiles were determined for each of the 19 CPT-Stinger by using equations (4), (5) and (6) and the parameters presented in Table 3.

Table 3: Parameters for OCR calculation

Parameter	Value	Test
ϕ'	31°	Assessed experimentally from DSS tests from high quality samples
M	0.86	Calculated from $\phi' = 31°$
I_r	100	Assumed (Mayne, 2008)
Λ	0.9521	Assessed experimentally from CRS tests from high quality samples

Figure 10 presents separate assessments of the OCR profiles by using equations (4), (5) and (6) for the same PCPT profiles shown in Figure 8. Results based on u_2 (Equation 5) compare very well with OCR data obtained from CRS tests from high quality samples. In contrast, the approximations given by (4) and (6) can be seen to be shifted significantly to the right. It is important to note that a consistent and compatible OCR profile is attained by all three approximations within the standard CPT data interval. This tends to indicate again that a certain degree of inaccuracy is present in the corrected q_c data within the dynamic interval.

Based on these observations, OCR profiles estimated using equation (5) were chosen for determining a third family of undrained shear strength profiles for the EA clays according to the SHANSEP methodology given by equation (3). Results are presented in Figure 11 together with the set of S_u profiles previously presented in Figure 8. The results favorably compare with respect to the S_u profile based on $N_{\Delta u}$ approach. Therefore, the method using $N_{\Delta u}$ most likely gives a closer prediction of the true undrained shear strength of EA clays as compared to that using N_{kt}. Or, a higher degree of reliance can be given to pore-pressure measurements for estimating S_u using the CPT-Stinger for offshore testing.

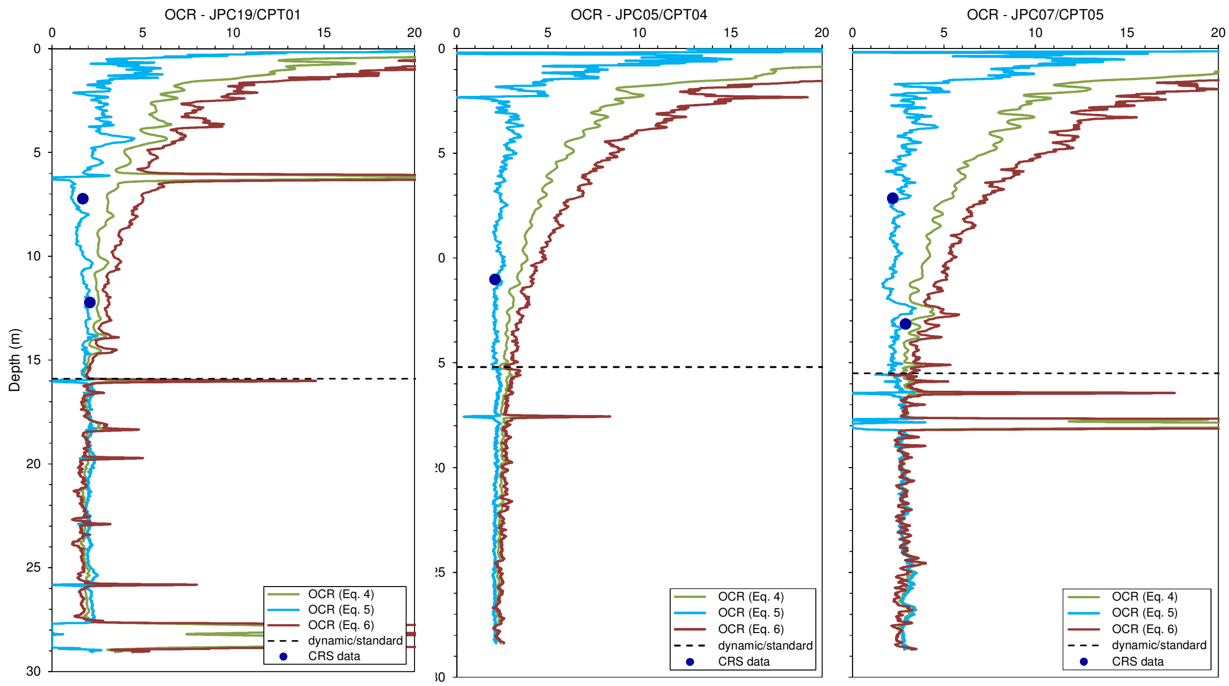

Figure 10: OCR profiles derived from cone tip resistance (q_c), measured excess pore pressure (u_2), and (q_c) - (u_2)

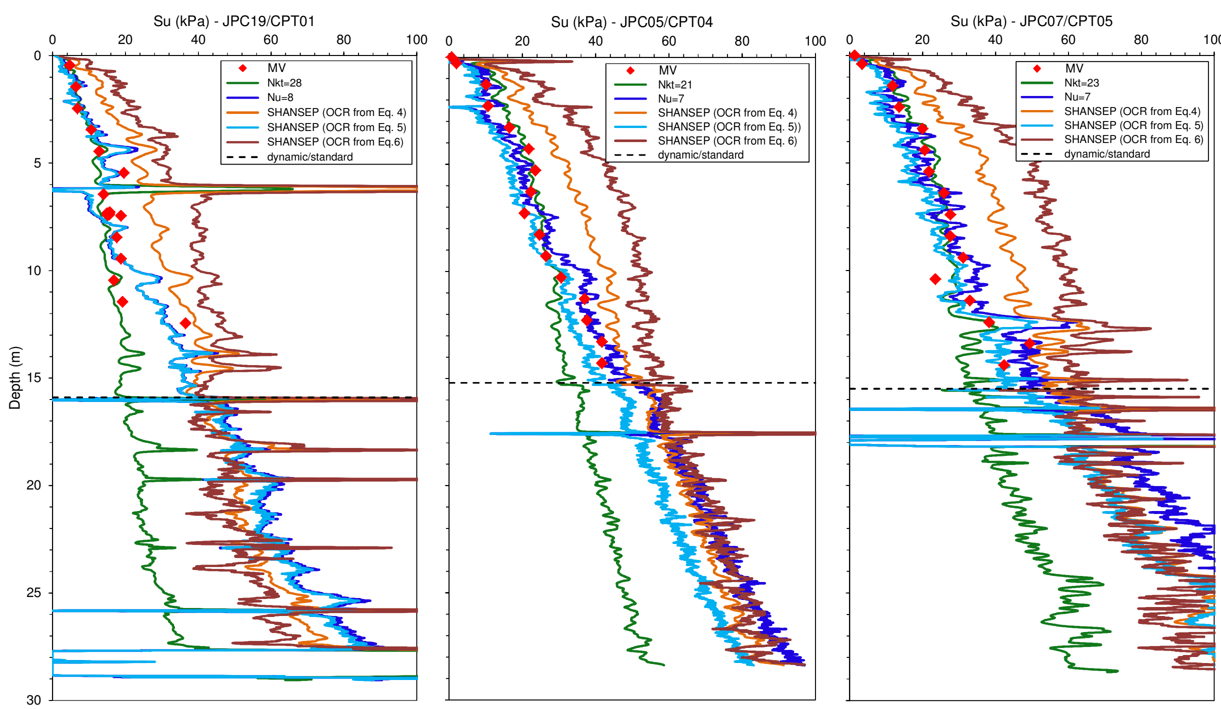

Figure 11: Undrained shear strength profiles derived by means of cone factors N_{kt} and $N_{\Delta u}$ and SHANSEP concepts

Finally, Figure 12 shows separate assessments of S_u profiles given by equations (1) and (2) by using constant cone N-factors of $N_{kt} = 14$ and $N_{\Delta u} = 8$. For comparison purposes S_u profiles based on SHANSEP concept utilizing OCR profiles from equations (4) and (5) are also presented. It can be seen that both tip resistance and pore pressure based predictions show noteworthy similar findings throughout the standard CPT interval. However, although S_u profiles based on tip resistance from equations 1 and 4 are compatible within the dynamic interval, data are not comparable to the measured MV data used as the reference strength. Conversely, S_u predictions based on pore pressure measurements can be seen to nicely compare with the reference MV values in the dynamic interval.

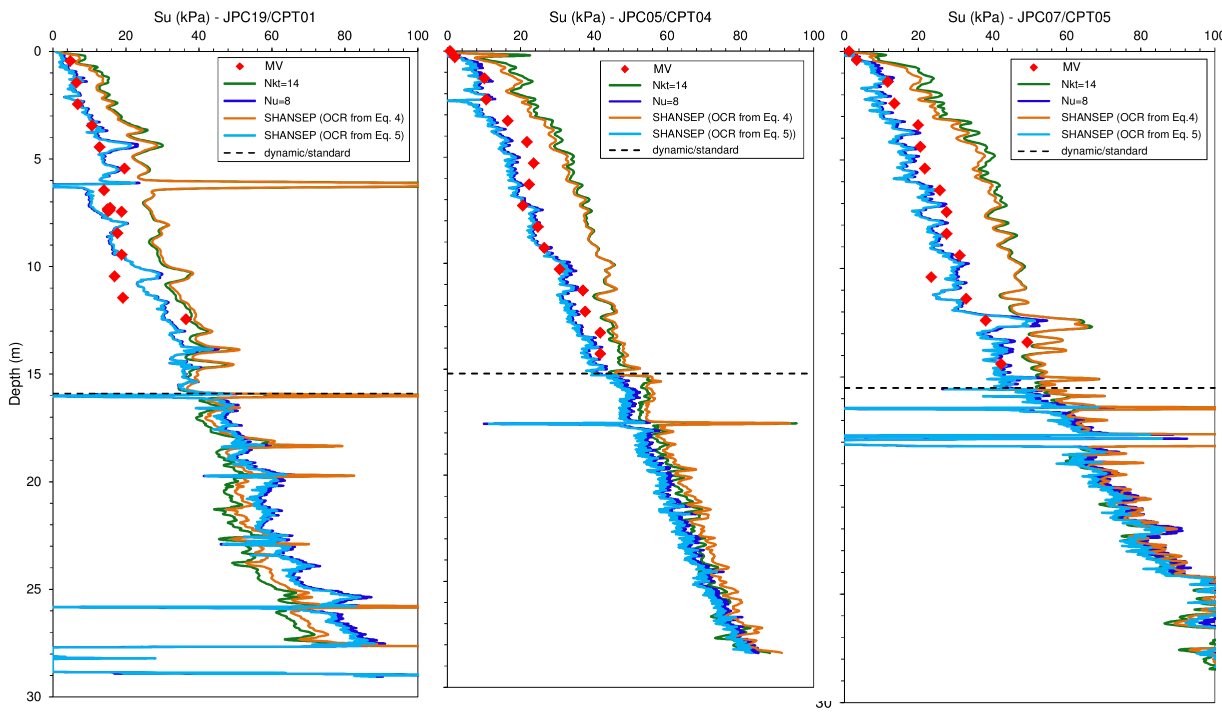

Figure 12: Undrained shear strength profiles derived by means of constant cone factors $N_{kt} = 14$ and $N_{\Delta u} = 8$ and SHANSEP concepts

CONCLUDING REMARKS

The CPT-Stinger provides a relative simple and economic alternative to other more conventional methods for site investigation in deep water conditions. The empirical cone factor N_{kt} ranged between 18 and 37 with an average value of 28 by using equation (1). The average N_{kt} is much greater than 20, which is significantly out of the bound of 10 to 20 as observed by Lunne et al (1997) and others for soft marine clays. The problem of using q_c in very soft clays and deposits with no previous experience is also highlighted. On the other hand, the cone factor $N_{\Delta u}$ ranged from 7 to 12 using pore pressure based equation (2). This range aligns reasonably well with the normally consolidated clay range indicated in the literature.

The findings for this case study of soft marine deposits off east Africa confirm that a high degree of reliance can be given to pore-pressure measurements (u_2) for estimating S_u for offshore testing. For projects where limited geotechnical knowledge but high-quality field and laboratory data are available it is highly recommended to develop site-specific correlations for an appropriate and reliable assessment of values of S_u.

Preliminary observations also suggest that a unique relationship between net cone tip resistance (q_{net}) and shear strength (S_u), similar to that postulated for other soft marine clay deposits around the world (Lunne et al. 1997), may well be devised when additional experimental evidence of this kind becomes available for these African deposits.

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