Interpretation of the CPT in engineering practice

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ABSTRACT: Many empirical and theoretical CPT interpretation methods are broadly accepted and used in practice. These approaches tend to consider whether the cone penetration is drained or undrained, and then will consider the soil as either "sand" or "clay," respectively. Most fundamental research into the CPT and its interpretation considers penetration through sands or clays separately, and includes verification tests in materials with close to ideal sand or clay behavior. However, in our engineering practice over the past five years, we have encountered several instances where the soils are not typical and the usual methodologies are inadequate or lead to inconsistent conclusions. This paper describes three of these case histories and our approach to using the CPT in each case.

1 INTRODUCTION

Soil behavior is in general complex. In addition to physical interactions at the particulate or clay mineral scale, there is coupling of the pore fluid flow with the solid matrix deformation; however, only macro-behavior is generally observed in soils testing. In triaxial testing, for example, we impose well-controlled and known boundary conditions on soil samples to measure the shear strength of the sample directly. Cone penetration into soils imposes a completely different and complex set of boundary conditions, and we take indirect measurements of the soils' response to these imposed boundary conditions (i.e. parameters such as tip resistance or sleeve friction) to determine the shear strength of the soil. As a result, interpreting the cone penetration test (CPT) is largely empirical, with the best empirical approaches having a sound theoretical basis. Relatively simple soil and interpretation models tend to gain the widest acceptance, but also fail to capture some of the complexities of soil behavior and the imposed CPT boundary conditions. Robertson (2009) implicitly captures this dilemma and presents an overview of currently available models for CPT interpretation. Robertson (2009) suggests that the empirical models are suitable as a screening tool for "low risk" projects. However, in our geotechnical practice, we seldom encounter "low risk" projects where these screening methods are sufficient.

In general, our projects are large, involve capital expenditures in the hundreds of millions of dollars, and tend to be in remote areas of the developing world, rather than in developed urban areas of North America and Europe. Due to the scale and location, the cost of site investigation for our projects is high, but contracting agencies still have a tendency to squeeze the budget and minimize spending on geotechnical testing. However, the most important consequences for CPT interpretation stem from the location of our projects. First, because of the remote locations, it is difficult to get high-quality laboratory testing on undisturbed samples. Ideally testing would be carried out in field laboratories on site, but it is costly to exercise good quality control and to provide geotechnical oversight of the testing on site. The preferred option is to ship samples to permanent laboratories for advanced testing and to pay the (unknown) penalty of sample disturbance during transport. Another significant issue in remote sites is a complete lack of local experience and prior knowledge of what methods and approaches are suitable for the soils in the region; for example, what typical values of N_{kt}, the coefficient relating tip resistance to undrained shear strength in Equation 1, might be appropriate.

Our experience is therefore that site-specific relationships for the CPT interpretation are always needed to determine soil properties from the CPT. The CPT is a very good tool to give an indication of soil behavior, continuous vertical profiles, variability between locations, and of course undrained shear strength of clays and density or state of sandy soils. However, the CPT can only be used with confidence when supported by all of the other tests and information at our disposal from the site investigation. There have been several instances where the soils are not typical sands or clays, and the usual methodologies are inadequate or lead to inconsistent conclusions. In this paper, we will discuss three examples where traditional CPT methodologies were not sufficient for the location of the project or the type of soils encountered. We will present the data and discuss our methodologies for using the CPT results in conjunction with the other tests.

2 RESIDUAL SOIL SITE

The first example is a site where the soils of interest were residual soils derived from weathered basalt. The soil column is typically 15 m thick overlying the weathered basalt bedrock profile. Near the ground surface, these soils lack texture or traces of the parent rock and consist of brown to red and yellow, high plasticity silts. At a slightly greater depth, the residual soils transition to weathered basalt and are typically dark brown, black and dusky red, high plasticity silts. The liquid limit of all these soils is generally greater than 50, and the plasticity index is such that they plot below the A-line on the Casagrande plasticity chart. According to the Unified Soil Classification System (USCS) in ASTM D2487, these soils have the abbreviated descriptor MH, and are called elastic silts (or high plasticity silts outside the USA).

2.1 *Index Tests*

Figure 1 shows the index properties of the upper 15 m of the soil profile, including the SPT N value and the undrained shear strength. Undrained shear strength was generally measured with unconsolidated undrained (UU) triaxial compression tests in a field laboratory, but Figure 1 also presents results from several pressuremeter tests, which are shown as hollow squares. The moisture content ranges between 19% and 79% and increases slightly with depth. The liquid limit (LL) ranges from 59 to 119, and the plasticity index (PI) ranges from 9 to 47. The average undrained shear strength (c_u) measured in UU tests is approximately 120 kPa, with relatively wide scatter and a standard deviation of 88 kPa. The high standard deviation is an indication of a large variability within this residual soil, which may be attributed to sampling disturbance, failure along micro discontinuities, natural variability, and maybe other factors. It is also noted that stiffer soils in the profile are difficult to sample, and therefore UU tests are considered to be representative of the weaker parts of the soil column. The groundwater table at this site was at a depth of about 8 m at the time of drilling and CPT testing.

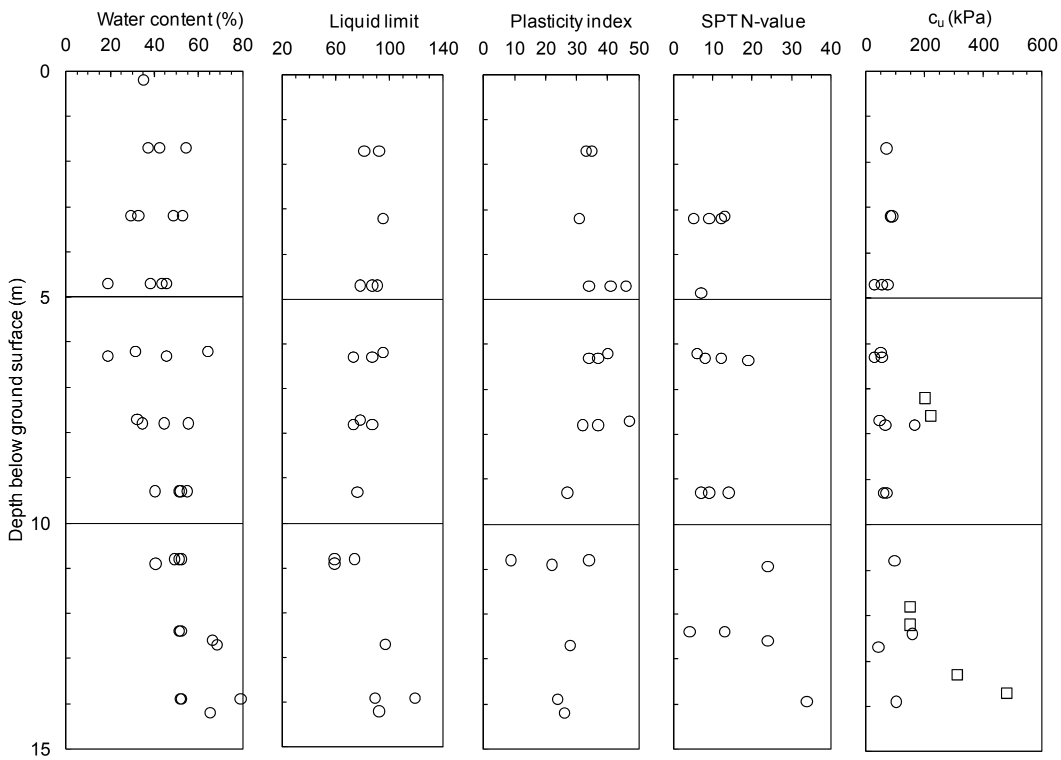


Figure 1. Soil properties at the residual soil site in West Africa (in undrained shear strength plot circles indicate UU test results and squares indicate pressuremeter test results).

2.2 *CPT Data*

Several CPTs were carried out at the site, but Figure 2 shows only one CPT. This CPT was situated close to the soil borings sampled to provide the index properties and undrained shear strengths on Figure 1. Figure 2, shows the tip resistance (q_t, corrected for end area effects) as well as the common normalized parameters; friction ratio (F), pore pressure parameter (B_q), normalized tip resistance (Q) and soil behavior index (I_c). Both the Robertson and Wride (1998) and the Jefferies and Davies (1993) versions of I_c are shown, illustrating that in this case there is very little difference between the two (because the main difference between the two equations for I_c is that Jefferies and Davies include the pore pressure parameter B_q which is generally less than about 0.1 for these soils). In the upper 5 m, the behavior type is between “clayey silt to silty clay” and “clay”, and below 5 m the behavior type is solidly in the “clay” zone, according to the Robertson and Wride (1998) classification. (The Robertson and Wride classification is preferred in this instance because of the potential for desaturation of the pore pressure stone during penetration above the ground water level). Figure 3 shows the CPT measurements on a soil behavior chart, which further illustrates the soil classification.

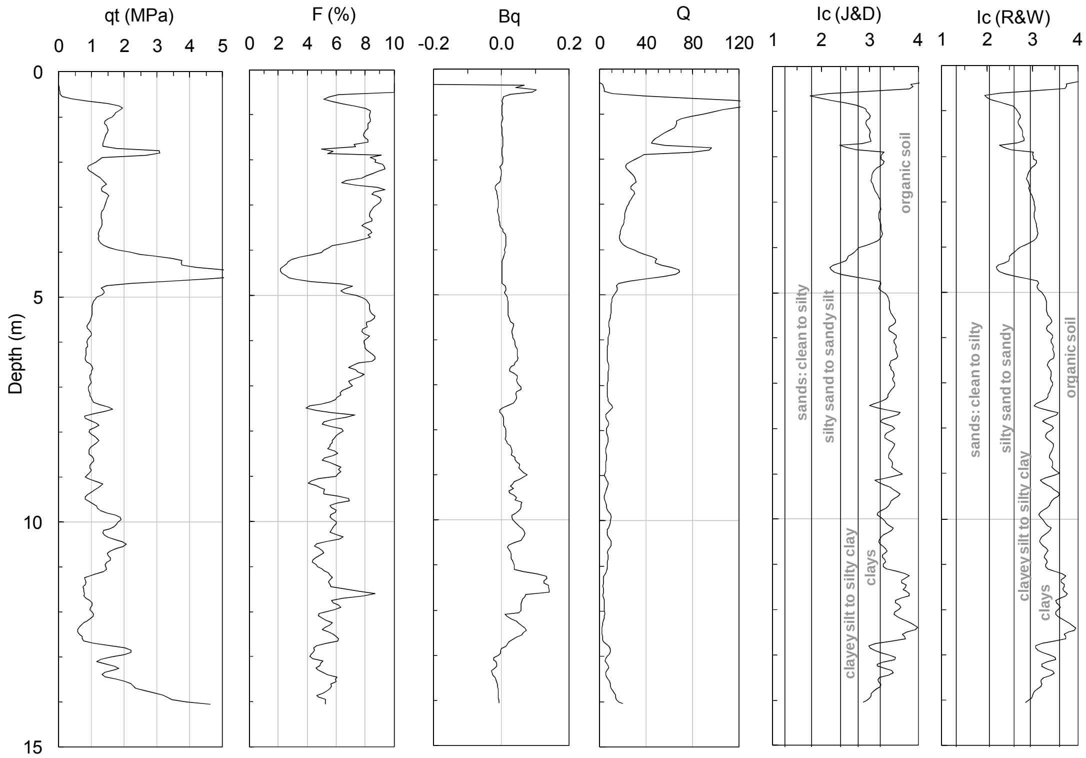
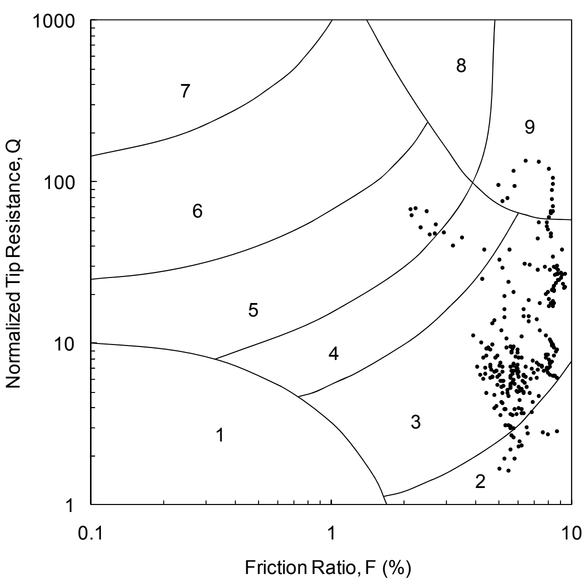


Figure 2. CPT profile at residual soil site (soil behavior type I_c shown for both Jefferies and Davies, 1993, and Robertson and Wride, 1998, equations)



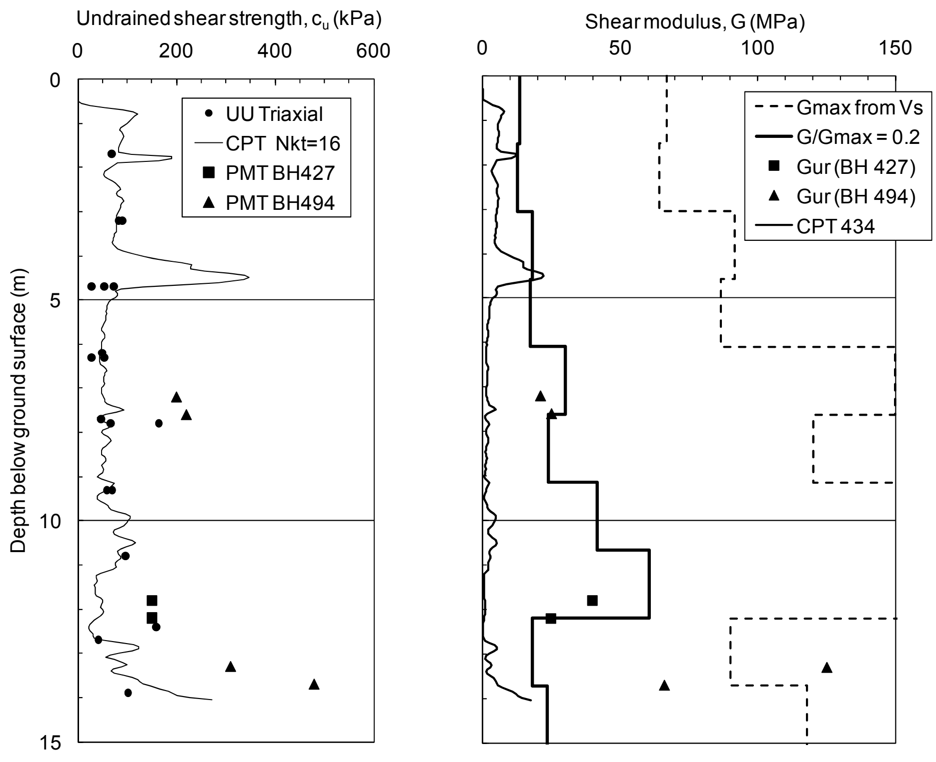
1. Sensitive fine grained
2. Organic soils – peats
3. Clays – clay to silty clay
4. Silt mixtures – clayey silt to silty clay
5. Sand mixtures – silty sand to sandy silt
6. Sands – clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Figure 3. Plot showing where data falls on the Robertson (1990) classification of soil behavior types.

The soil behavior type indicated by the CPT for this residual soil seems to be appropriate despite being different from the soils used to develop the CPT charts. While the material is classified as an elastic or high plasticity silt based on its Atterberg limits, the mechanical response of the soil is similar to that of a clay, i.e. the engineering parameter used for characterization of its strength during static loading is undrained shear strength.

2.3 *Engineering Interpretation of the CPT*

This particular location was for a large (850,000 barrel) settlement-sensitive tank, and the key foundation considerations were bearing capacity and settlement. Therefore, the primary engineering parameters of interest from the CPT were undrained shear strength and compressibility. While overconsolidation is a meaningless concept in a residual soil, oedometer tests were carried out on the material to determine the compressibility. These showed that the yield stress in compression (analogous to preconsolidation pressure) was greater than double the design tank loading. Settlement was therefore characterized by elastic parameters for the soil.



a) Undrained shear strength b) Shear modulus

Figure 4. Interpretation of CPT in residual soils; b) shows G_{max} from V_s, 0.2 × G_{max}, G_{ur} from pressuremeter tests, and G from CPT tip resistance.

As illustrated in Figure 4a, a relatively simple technique was used to estimate N_{kt}, the empirical parameter required to calculate undrained shear strength (c_u) in triaxial compression from CPT. N_{kt} is defined in Equation 1, where q_t is the cone tip resistance and σ_v is the total geostatic vertical stress. We varied N_{kt} until the CPT graph of undrained shear strength with depth on Figure 4a provided the best visual match to the UU tests on samples from nearby borings. A value of N_{kt} = 16 provided the best match. It is apparent that the small number of pressuremeter measured undrained shear strengths (abbreviated as PMT in Figure 4a) are higher than UU test results, and this approach of using UU tests as a reference might be somewhat conservative in this soil, i.e. N_{kt} could be less than 16.

$$c_u = \frac{q_t - \sigma_v}{N_{kt}} \tag{1}$$

Given that the bearing capacity exceeded the design load even using conservative values, settlement of the tank, and hence modulus of the residual soils, became the focus of the interpretation. In situ test information on modulus was available from shear wave velocity (V_s), pressuremeter tests, and CPT-based empirical relationships, in addition to laboratory tests. As a first step, we noted that laboratory tests suggested moduli that were significantly lower than those suggested from in situ tests. When we identified this distinct difference between the results, we did not consider the laboratory values further when selecting design parameters, as they were likely affected by sampling bias (i.e. selective sampling and sub-sampling of laboratory specimens) and sample disturbance. Figure 4b shows the range of shear modulus estimates from:

- G_{max} determined from downhole shear wave velocity measurements, $G_{max} = \rho V_s^2$, where ρ is the mass density of the residual soil determined from tube samples.
- G from shear wave velocity with a degradation factor of G/G_{max} = 0.2 to approximate 1% shear strain.
- G_{ur} determined from the slope of an unload/reload cycle during a pressuremeter test.
- G from the CPT considering the constrained modulus, M, from Equations 2 and 3 with α_M = Q when Q ≤ 14 and α_M = 14 when Q > 14. (Robertson, 2009)

$$G = \frac{M(1-2\nu)}{2(1-\nu)} \tag{2}$$

$$M = \alpha_M (q_t - \sigma_v) \tag{3}$$

The final CPT-based method results in much lower modulus than the degraded shear wave velocity method and the pressuremeter, while the pressuremeter and shear wave velocity methods give similar results.

The above derivation of G from the CPT passes through the intermediate step of constrained modulus M. However, there are direct correlations between G_{max} and tip resistance for cohesionless soils, for example Equation 4 (Robertson 2009)

$$G_{\max} = \alpha_G (q_t - \sigma_v) = 0.0188 \times 10^{(0.55 I_c + 1.68)} (q_t - \sigma_v) \tag{4}$$

While Equation 4 would not strictly be applicable for undrained, cohesive soils, Figure 5 shows a comparison between G_{max} from Equation 4 and G_{max} derived from the shear wave velocity. As before, the CPT gives significantly lower values than from shear wave velocity measurements, but not as low as when using Equations 2 and 3 for cohesive soils. Schnaid (2005) has suggested that such differences might be the result of aging or cementation in cohesionless soils, which is also plausible for residual soils but caution is needed in its implementation. There may be factors related to pore pressures and drainage that affect the interpretation in cohesive versus cohesionless soils.

We decided to use the pressuremeter and shear wave-based values, since they result from direct measurement of shear modulus. In contrast, the CPT measures a complex mix of shear strength and stiffness response to the cone penetration and the CPT-based method is indirect. This choice to use the pressuremeter and shear wave values was justified during subsequent monitoring of tank settlements during hydrotesting and operation of the facility, as settlement predictions compared favorably with measurements.

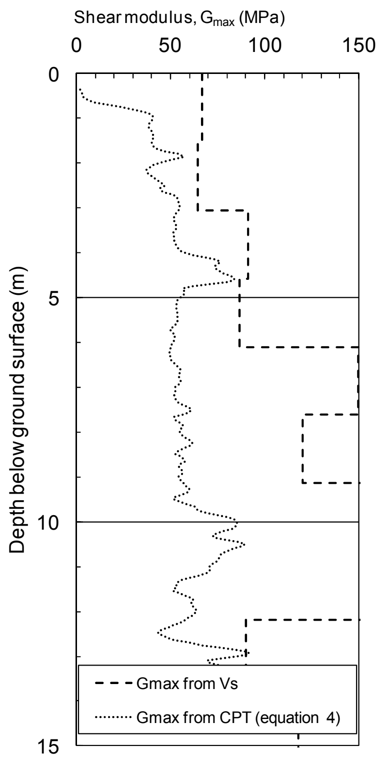


Figure 5. Comparison of G_{max} from for residual soils from a) shear wave velocity and b) CPT correlation for cohesionless soils (Equation 4)

2.4 *Summary*

The residual soil case history is interesting in that CPT correlations are typically developed for sedimentary soils, not for residual soils. The soil behavior index, I_c, is appropriate in this case; undrained clay-like behavior was observed in laboratory tests and was correctly identified by the CPT, but the CPT was incapable of showing that the material was a highly plastic silt, rather than a clay. The estimated N_{kt} value of 16 or less to determine undrained shear strength is well within the band of expectation for CPTs in stiff clays and presents no surprise. However, the problem of using the CPT to estimate modulus in an atypical soil is also highlighted. The CPT primarily results in shearing of the soil, and its use to estimate modulus is limited, to some degree, to how well modulus and undrained shear strength are related. Even for sedimentary materials, there is a wide range of modulus to undrained shear strength ratios, and there is no reason to believe that a residual soil should fall within the same range. We do not recommend estimating soil stiffness using empirical correlations from the CPT in a new area or in a soil for which there is no prior experience, even at a screening level. However, in this case deploying a seismic CPT (SCPT) to obtain measurements of the shear wave velocity would have been helpful to identify the high stiffness relative to the measured tip resistance (e.g. Schnaid, 2005).

3 ESTUARINE CLAY

In this section, we present an example of a nearshore organic clay layer that was investigated for a large gas processing site. The engineering considerations for this project included undrained shear strength for slope stability and compressibility. CPTs and soil borings were advanced through the organic clay layer and samples recovered for testing. Index tests were carried out on most samples, however, it was impossible to acquire sufficient undisturbed samples for laboratory compressibility and strength testing to characterize the material fully. Given the paucity of data, we relied on calculations of preconsolidation pressure and shear strength using CPT tip resistance and empirical relationships, but verified that the choice of N_{kt}, k, and SHANSEP parameters were compatible.

The clay was deposited in what was likely a mangrove area of a bay within the estuary of a large river. Based on the condition of the soil, it is unlikely that the clay was subjected to mechanical preconsolidation in this quiescent depositional environment. The clay was lightly overconsolidated due to aging and creep, but not due to prior overburden stresses.

3.1 *Index Tests*

The clay was characterized by carrying out moisture content, Atterberg limits, and particle size distribution tests. The "organic clay" (OH) designation was based on visual observations of the presence of organic matter, measured organic content in laboratory tests, slight odor, and the comparison of the liquid limit value of the material at its natural state with the liquid limit measured after oven-drying the specimen. The moisture content of the material was highly variable (Figure 6), and the liquid limit was typically greater than 50, reaching values as large as 160. The plasticity index typically ranged between 60 and 101, and the material plotted slightly above or below the A-line on the Casagrande plasticity chart (Figure 7). The liquidity index (LI) ranged between 0.3 and 2.4 (Figure 6). In this instance the groundwater was above the surface and the average unit weight of the clay was approximately 14 kN/m^3, giving a relatively low submerged unit weight of 4.2 kN/m^3 for CPT interpretation.

3.2 *CPT Data*

The plots on Figure 8 present typical values of the tip resistance q_t, friction ratio F, B_q, and I_c for the organic clay layer (which occurs in the upper 11.5 m of the profile). The material exhibited increasing penetration resistance with depth (at an average rate of about 50 kPa/m) as expected in this geological environment. The friction ratio is between 2% and 3% and positive excess pore water pressure occurred during testing such that typically $0.2 < B_q < 0.6$. The soil behavior index varies between $2.8 < I_c < 3$); therefore, the material classifies as "silty clay" and "clay" according to both the

Jefferies and Davis (1993) and the Robertson and Wride (1998) systems. Figure 9 shows the organic clay on the Robertson (1990) soil behavior type chart, indicating an accurate behavior type classification. (The material is not a peat and the organic content is relatively small.)

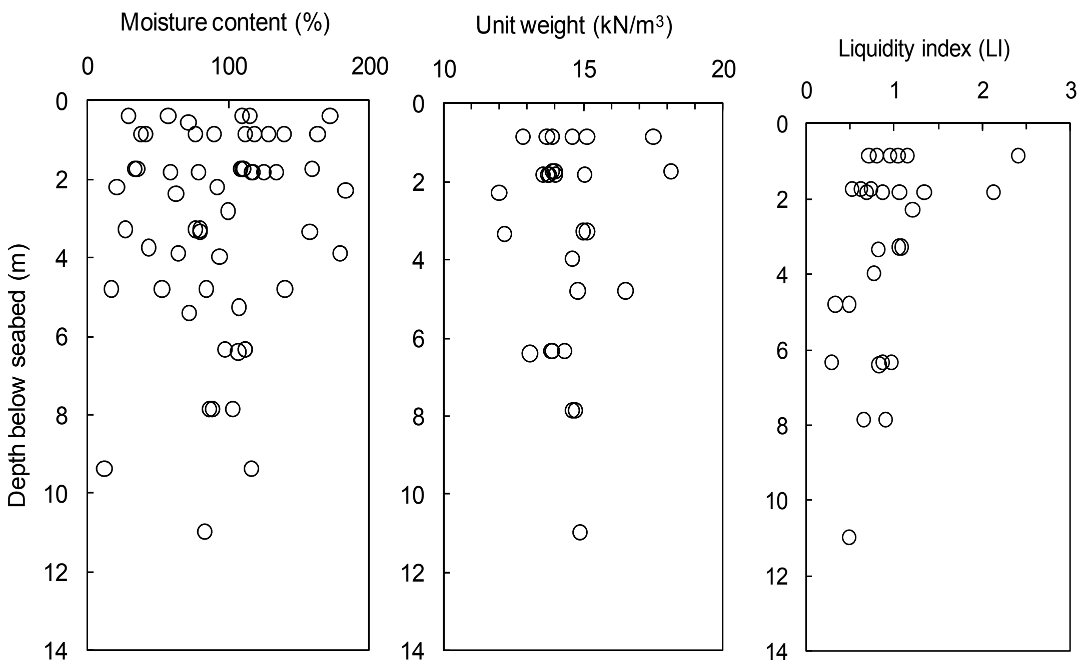


Figure 6. Plot of moisture content, unit weight and liquidity index with depth, estuarine organic clay.

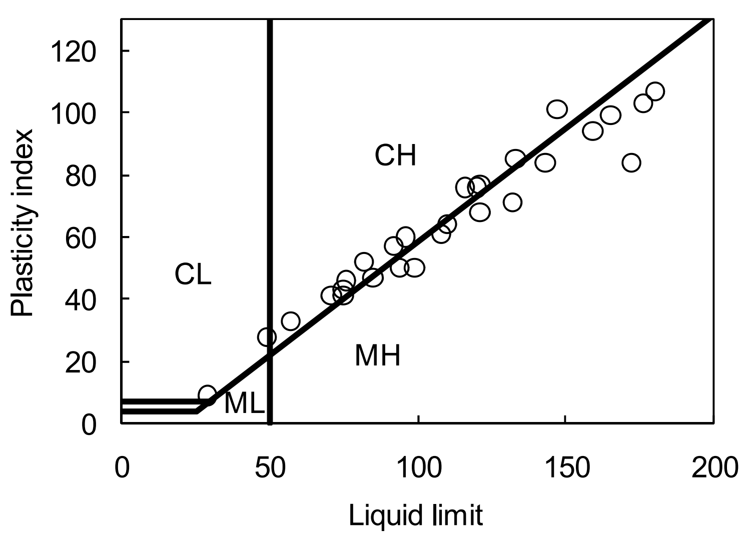


Figure 7. Plasticity chart showing location of organic clay on Casagrande plasticity chart.

3.3 *Engineering Interpretation of the CPT*

The strength and compressibility of the organic clay were sought for engineering evaluation. Given the dearth of laboratory test data, a value for N_{kt} (Equation 1) between 12 and 16 was initially selected, with a preferred value of 14. A value of k = 0.2 was then selected to estimate the preconsolidation pressure (σ'_p) using Equation 5.

$$\sigma'_p = k \cdot (q_t - \sigma_v) \tag{5}$$

However, in clays, c_u and σ'_p, or the overconsolidation ratio (OCR = σ'_p/σ'_v), are generally also related. Ladd and Foote (1974) empirically developed the relationship in Equation 6, where $(c_u/\sigma'_v)_{OCR=1}$ may also be designated as S. The values of the SHANSEP parameters S and m should be calculated through curve-fitting laboratory test data using the procedure recommended by Ladd and Foote (1974); however, in this case, these parameters also had to be estimated given the shortage of strength test data for the material.

$$\frac{c_u}{\sigma'_v} = \left(\frac{c_u}{\sigma'_v}\right)_{OCR=1} \cdot \left(\frac{\sigma'_p}{\sigma'_v}\right)^m \tag{6}$$

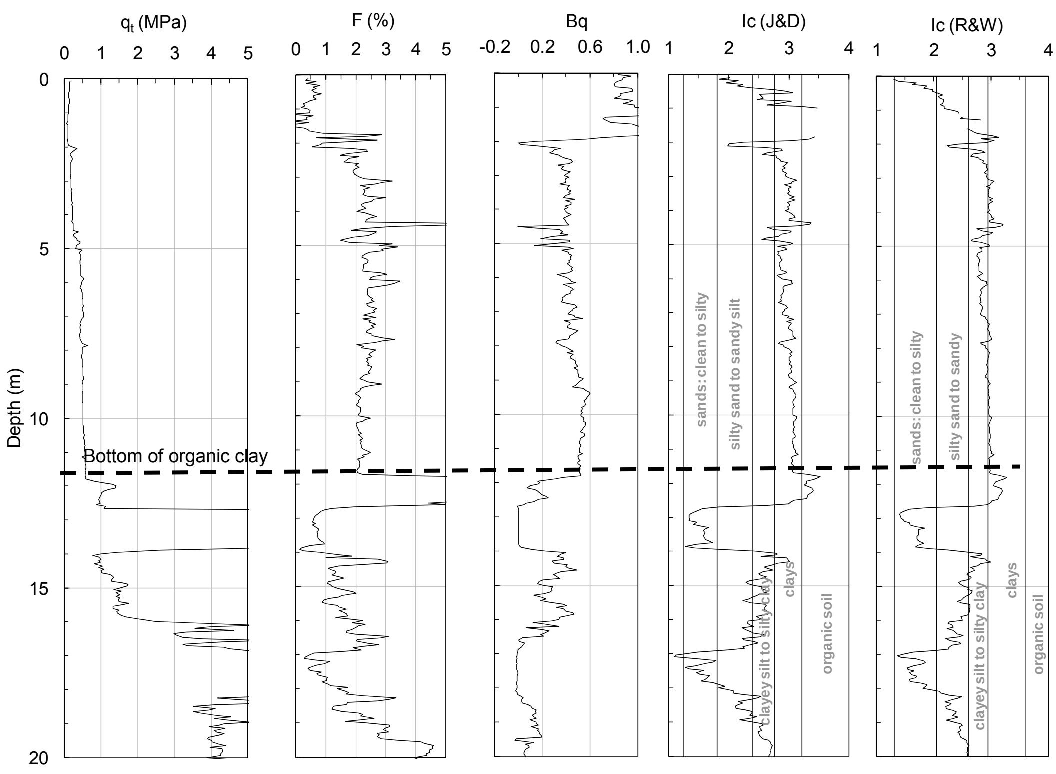
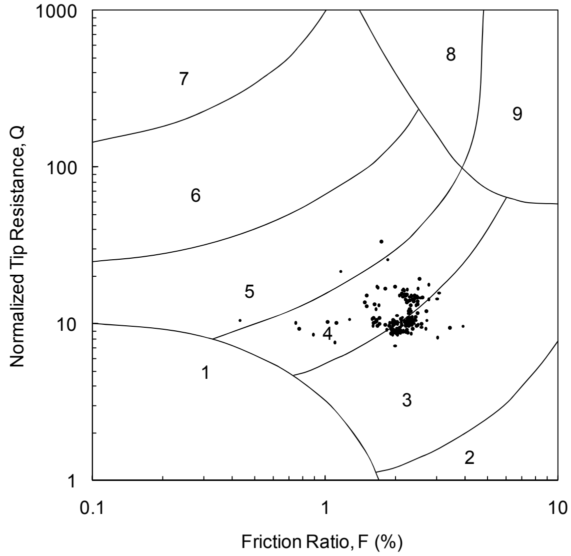


Figure 8. Plots of tip resistance, friction ratio, normalized pore water pressure, and soil behavior index (according to both the Jefferies and Davies and the Robertson and Wride systems).



1. Sensitive fine grained
2. Organic soils – peats
3. Clays – clay to silty clay
4. Silt mixtures – clayey silt to silty clay
5. Sand mixtures – silty sand to sandy silt
6. Sands – clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Figure 9. Organic clay CPT results on the Robertson (1990) classification of soil behavior types.

The term S varies slightly as a function of the failure mode (testing method, strain rate), but may be considered to be constant in this case. Ladd and De Groot (2003) recommend S = 0.25 with a standard deviation of 0.05 (for simple shear loading) and m = 0.8 for sedimentary deposits of silts and organic soils (Atterberg limits plot below the A-line), excluding peats and clays with shells. S may take a value of approximately 0.30 to 0.35 for triaxial compression loading (e.g. Ladd and DeGroot, 2003).

Only certain combinations of the empirical parameters N_{kt} (tip resistance to c_u), k (tip resistance to OCR), and S and m are compatible with all of Equations 1, 4, and 5. Substituting Equation 1 and 5 in Equation 6 to determine the appropriate combination of parameters gives:

$$N_{kt} \cdot S \cdot k^{m} = \left(\frac{q_t - \sigma_v}{\sigma'_v} \right)^{1-m} \qquad (7)$$

The left hand side of Equation 7 is a constant for a given soil layer, therefore, the magnitude of this constant may be calculated by first selecting an appropriate value for m and then by plotting the right hand side of the equation, as shown on Figure 10 for eight CPT soundings, where m = 0.85 was chosen. Significant variability in the data is noticeable on Figure 10 due to variability of the tip resistance around the project site. The median value of the right hand side of Equation 7 is 1.45, estimated from the measured data which are plotted on Figure 10 and shown as a bold vertical line. Therefore, the constants N_{kt}, S, and k on the left hand side of Equation 7 should be chosen accordingly.

Since there is no single solution to Equation 7, different scenarios or combinations of parameters were considered for engineering analyses. Table 1 shows combinations of compatible parameters, where m = 0.85 in all cases, that were used to calculate c_u and σ'_p for subsequent stability and settlement analyses. N_{kt} and S were selected and the value of k calculated corresponding to the median of the data, Figure 10. Scenario A has a typical value of S for strength in simple shear, but although k has a reasonable value, the magnitude of N_{kt} appears to be too large for a soft, lightly overconsolidated clay. The combination of Scenario B in Table 1 has a reasonable value of N_{kt} and S for triaxial compression, but k appears to be high. Scenario C has high N_{kt}, reasonable S for triaxial compression, but low k. Scenario D has reasonable N_{kt} and k, but S appears to be too high. Consideration of these four scenarios provided bounds on the settlement and stability calculations that would have been much broader if this approach had not been adopted.

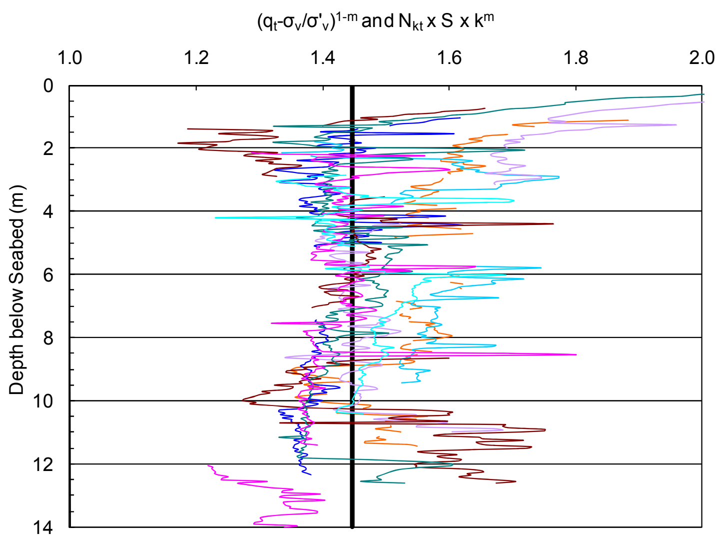


Figure 10. Plot of CPT tip resistance term on the right hand side of Equation 7, with the median value shown as a bold vertical line.

Table 1. Combinations of compatible empirical parameters for CPT interpretation.

Parameter	Scenario A	Scenario B	Scenario C	Scenario D
N_{kt}	20	14	20	14
S	0.25	0.30	0.30	0.36
m	0.85	0.85	0.85	0.85
k (median)	0.246	0.302	0.198	0.244

3.4 *Summary*

The standard of practice to develop empirical factors to correlate undrained shear strength and preconsolidation pressure to CPT tip resistance is to use site-specific laboratory test data. However, with only a handful of unconsolidated undrained triaxial tests and consolidation tests available, there was an insufficient amount of data. Therefore, we found ourselves in the undesirable situation to have to rely heavily on index test data, experience, and judgment to select appropriate values for N_{kt}, k, and SHANSEP parameters. Obviously, interpreting the CPT without laboratory test data is a process that introduces significant uncertainty in the choice of engineering parameters.

A procedure was developed to verify the compatibility of the choice for N_{kt}, k, and SHANSEP parameters through the manipulation of Equations 1, 5, and 6. We recommend the application of this verification process to analyze CPT data, as it is useful even when a good quantity and quality laboratory test data are available.

The value of the parameter N_{kt} typically depends on the value of OCR and clay plasticity (e.g. Karlsrud et al., 2005). Given that the SHANSEP parameters depend mostly on the index properties of the clay layer (and it is reasonable to assume that they do not vary within a clay layer), then the value of k should also vary with OCR within the layer, a somewhat circular proposition. Thus, it appears that neither k nor N_{kt} should be constant in a clay with varying OCR (the changes in k and N_{kt} could, however, be inversely proportional). The normalized tip resistance, Q, could be used as a clue to changes in N_{kt} and k, but we have not explored this in any detail.

A comprehensive laboratory test program should always be completed to determine the values of N_{kt}, k, S, and m. A comprehensive laboratory program should include a sufficient number of oedometer tests to determine the overconsolidation profile as well as undrained shear strength tests to determine the SHANSEP parameters S and m for each clay soil. In addition, undrained shear strength measurements at in situ conditions or on undisturbed samples consolidated to the estimated in situ stresses are required to provide another (direct) check on N_{kt} and as a check on the SHANSEP tests and procedures. Despite the experience on this particular project, the CPT should not be used as the sole measurement.

4 OFFSHORE SILT

This example considers a thick offshore silt deposit in about 140 m of water where we had information to a depth of about 100 m below the seabed. The full depth penetrated consisted of recent marine sediments, which typically presented as grayish green silt interbedded with fine sand and shell fragments. The soil properties and CPT results are presented in Figure 11 and Figure 12. The CPT was a downhole wireline device with a 1.5-m stroke, and the small stroke unfortunately results in some spikes and some discontinuities in the profile. Looking at both the CPT and soil classification tests, there are essentially three substrata, all silts; an upper silt, a high plasticity silt and a lower silt.

4.1 *Index Tests*

From seabed to a depth of about 64 m, low plasticity silt is the prevalent material. The liquid limit is less than 38 with a plasticity index less than 9 and this material is therefore classified as ML, according to the USCS. The natural moisture content is typically from 35% to 40% and is in general close to or above the liquid limit as indicated by the liquidity index on Figure 11. Sand layers are present as inferred primarily from the CPT records, but each of these layers is relatively thin and interbedded with the silt. The sand layers are more prevalent and seemingly denser between depths of 28 m and 38 m below seabed. Clay layers were not observed in the samples.

The second stratum between 64 m and 80 m is mostly high plasticity silt (MH in USCS) with a natural moisture content range from 37% to 61%, and the respective liquid limits between 43 and 84, leading to liquidity indices from a low of -0.4 to a high of 1.42. The upper part of this subunit is more plastic and is uniform on the CPT trace, which also shows the likely presence of sandy or silty interbeds in the lower part the stratum.

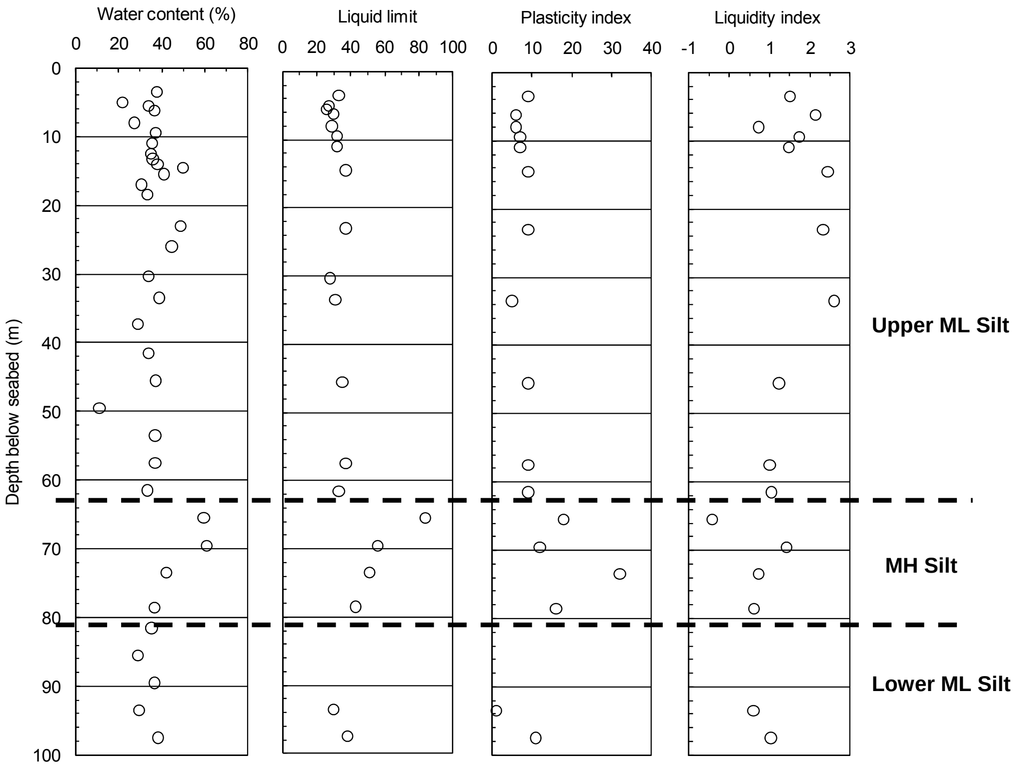


Figure 11. Soil index properties at marine silt site.

Below 80 m, the plasticity drops again to that of low plasticity (ML) silt. The liquidity index is very close to 1, with a liquid limit between 30 and 38, and natural moisture content between 29% and 38%. The plasticity index is between 27 and 29. The CPT records show the presence of abundant sand lenses.

4.2 *CPT Data*

In the CPT on Figure 12, a highly stratified deposit ranging between clays and sands is apparent from the tip resistance, friction ratio, pore pressure parameter, and material behavior index. However, some caution is needed in the interpretation, as the measurements in one layer may be heavily influenced by the layers above and below. The same variability was not apparent in the index tests on the soil samples, which were predominantly silts with very little sand or clay present in any of the samples. Visual examination of the samples (good quality, thin walled push samples) also did not indicate the stratification noted in the CPT. Plotting the CPT data on a soil behavior chart shows a confusing picture of material types ranging from clays through silts to clean sands, that looks like a shotgun blast on Figure 13. The pore pressure parameter B_q (Figure 12) shows both drained penetration ($B_q \sim 0$) and undrained or partially drained penetration, with B_q values as high as 0.4. In addition, the liquidity indices are generally greater than one (Figure 11).

The initial interpretation of this rather confusing picture was that there might be something fundamentally different about the silt. Diatomaceous materials are found

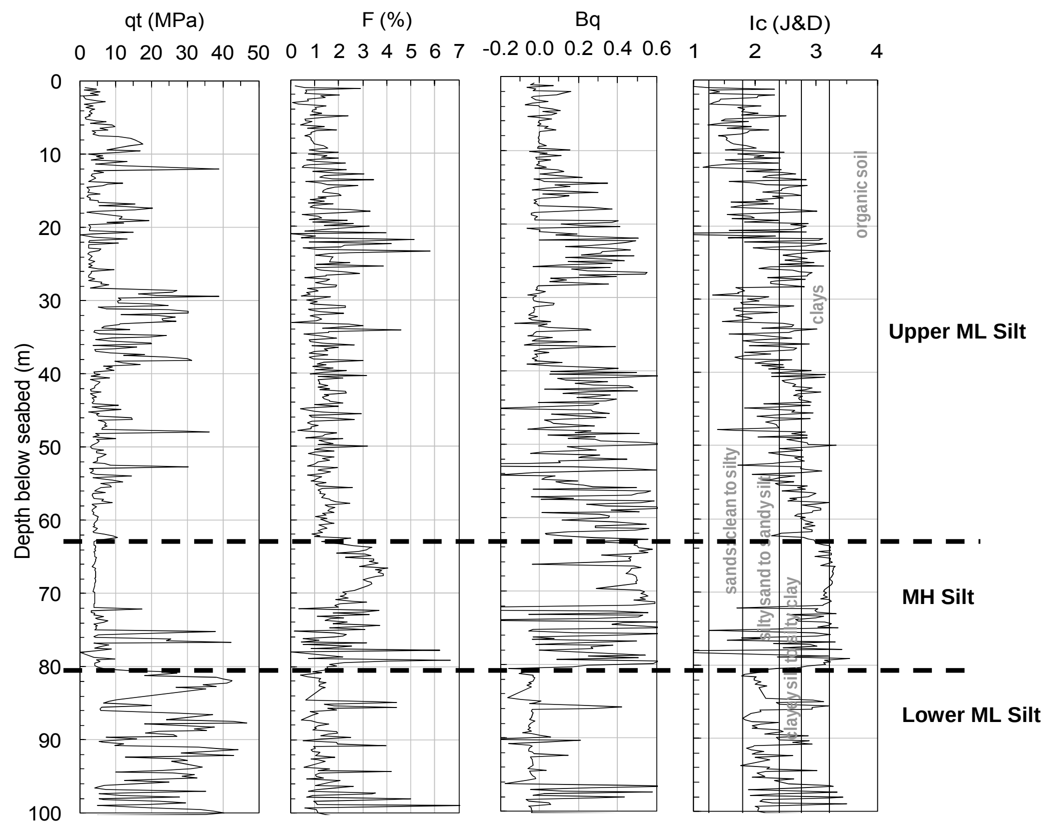
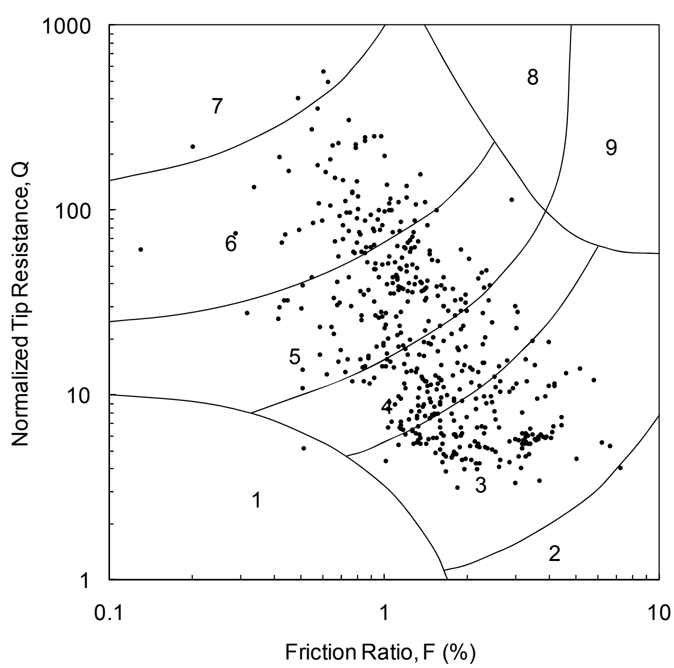


Figure 12. CPT profile at marine silt site.



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8. Very stiff sand to clayey sand
9. Very stiff fine grained

Figure 13. Plot showing where data for marine silt falls on the Robertson (1990) classification diagram.

in deeper offshore waters, and may have been transported to these shallower depths by deep seabed currents. However, microscopic examination of the silt grains indicated nothing unusual; this was a textbook siliceous silty material. Assuming then that both the CPT tests and the laboratory testing and soil descriptions were accurate, we concluded that these offshore silts were indeed strongly bedded, but that the primary difference between each layer was its strength and stiffness, rather than its visual appearance or soil classification. This variability in strength and stiffness was readily apparent in the CPT, although it may be muted by the influence of layering on CPT parameters. The denser or stiff silt layers exhibit drained "sand-like" behavior, while the less dense, less stiff silts exhibits "clay-like" behavior.

4.3 *Engineering Interpretation of the CPT*

A first step in the interpretation was to separate sand-like behavior from clay-like behavior, and the obvious way to do this with the CPT (assuming it includes pore pressure measurement) is based on the material behavior index. In this case, the Jefferies and Davies (1993) index was used, and the data were separated at an I_c value of 2.4, corresponding approximately to the boundary between zones 4 and 5 on the soil behavior type chart. On the sandy side of the divide (soil types 5, 6, and 7), the state parameter ψ was determined, while on the clayey side (soil types 3, and 4) undrained shear strength was calculated, as shown on Figure 13.

The CPT tip resistance is highly variable. Undrained shear strength (c_u) from triaxial compression test results also show a wide range from about 100 kPa to 900 kPa. Both tip resistance and undrained shear strength seem high and inconsistent with a liquidity index greater than 1. The highs and lows in undrained strength may reflect differences in the proportions of sand and clay within the silt, as high undrained strength values would be typically associated with the more granular (sand-like) silts, while low values would be typically associated with a more cohesive (clay-like) silt. The variability in undrained strength reduces between depths of 40 m to 80 m, which coincides with an increase in plasticity. Here triaxial compression undrained shear strength values have a range from about 150 kPa to 350 kPa.

From the above description of soils at the site, it is reasonable to consider that the low points in the CPT tip resistance are most representative of the undrained shear strength of the silts. In many cases, "lows" in the tip resistance might not be representative of the undrained shear strength in a thin layer and will tend to overpredict the shear strength, so it is appropriate to consider something close to the lower bound of the tip resistance profile. On Figure 14, the laboratory undrained shear strength measurements and the CPT undrained shear strength for I_c values greater than 2.4 are shown, using a value of N_{kt} = 14, on the same axis. There is a large variability in both the CPT and the laboratory shear strengths, as expected, but considering the variability, the laboratory data line up reasonably well with the low points of the CPT data. This alignment confirms that the selected value of N_{kt} = 14 is applicable. There is also an indication in the higher plasticity material between 60 m and 70 m that a slightly lower N_{kt} can be used, since the CPT plots below the two laboratory strength measurements in this depth interval. A rational design undrained strength profile in the depth intervals that are predominantly clay-like is indicated as a dashed line on Figure 14.

The state parameter values on Figure 14 for material with I_c values less than 2.4 have been calculated using the approach in Jefferies and Been (2006). The I_c value was used to determine values for the constants m and k, and then the state parameter was calculated directly from Q. Figure 14 shows that the silt has a state parameter generally smaller than ψ = -0.1 in the upper 40 m of the profile, and at greater depths, the state parameter lies between about 0 and -0.1. Under static loading, the silt is expected to be dilatant at these states, but under the high seismic loading possible at this site, it is predicted that high cyclic pore pressures could be generated. A design profile for state parameter in the depth intervals that are predominantly sand-like is shown on Figure 14, noting that a less negative state parameter indicates looser material and that the line is therefore at about a 10 percentile exceedance level.

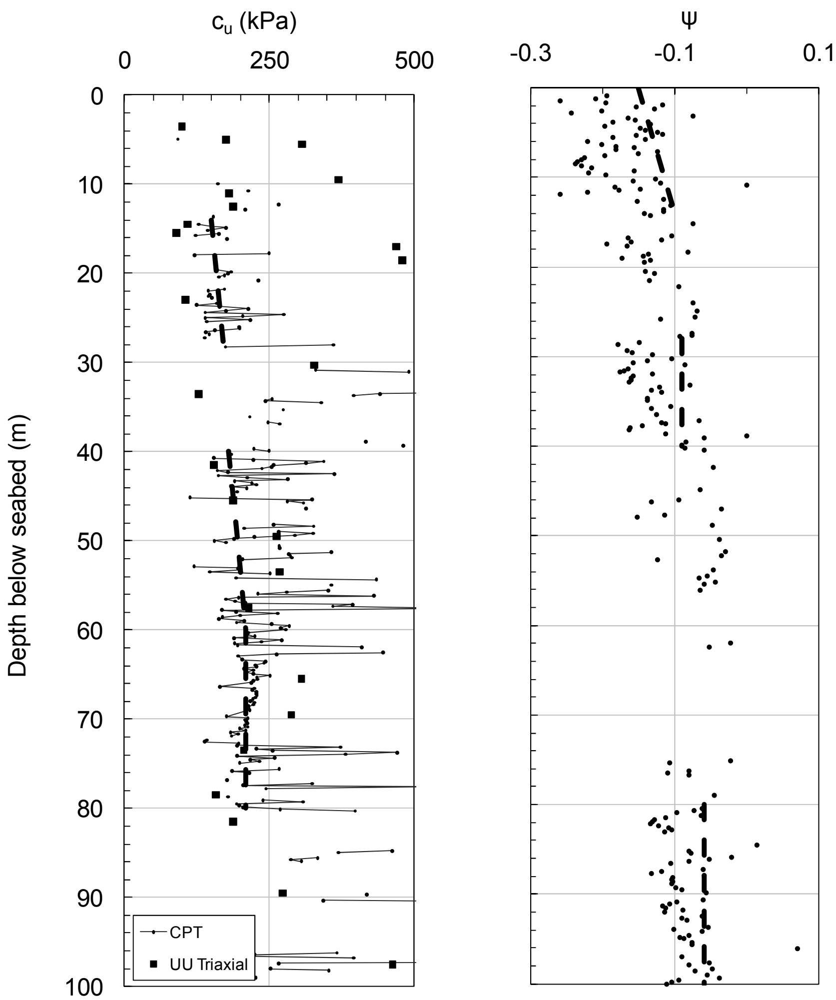

Figure 14. Plots of undrained shear strength and state parameter for marine silt (N_{kt} = 14 for undrained shear strength when $I_{c(J\&D)}$ >2.4)

4.4 *Summary*

This case study presents an interesting quandary for pile design. It is possible either to treat the whole soil profile as a nearly uniform silt (based on the soil properties) and select design parameters accordingly, or to try to determine what percentage of the profile is clay-like and what is sand-like, and then select appropriate sand or clay parameters for each soil type as was done above. It may not be important which approach the designer chooses, but it is certainly important to use a combination of the information from the examination of the samples, laboratory testing, and the CPT to present a wider picture of the soil composition.

5 CONCLUSIONS

The experiences described in this paper confirm the much repeated advice that the CPT cannot be used in isolation and requires an appropriate program of sampling and laboratory or related testing to confirm the selected soil parameters.

The residual soil example yielded an N_{kt} = 16 for undrained shear strength, which is well within the expected range for soils of the same plasticity. However, the modulus would not have been predicted well from the CPT and the example highlights that the CPT should not in general be used to determine parameters through indirect correlations. Application of a seismic CPT can be helpful in this context.

Classical soil behavior could be expected for the lightly overconsolidated estuarine clay site, and thus OCR and undrained shear strength are related through the relationships suggested by Ladd and Foote (1974). As a result, only certain combinations of parameters N_{kt} and k are compatible and were used in the absence of extensive laboratory strength testing during the early stages of this project.

The marine silt case provides a particular challenge for interpretation, both because of the layering and because it is a silt (i.e. not a clearly identified sand or clay). The material behavior type index does, however, provide a rational basis to separate sand-like from clay-like behavior in the CPT profile, and undrained shear strength and state parameter profiles were then developed corresponding to clay-like and sand-like behaviors.

It is also apparent from these conclusions that there are many soils that do not easily fit the idealized "sand" or "clay" behavior, on which much of soil mechanics depends. Therefore, the geotechnical engineer needs to have a flexible approach. In addition, without laboratory testing, visual examination of the samples and consideration of the geological history of the soil, the CPT can easily mislead in terms of soil type, strength and particularly modulus.

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