Characterization of Sensitive Sedimentary Deposits using the Dilatometer Test and Seismic Piezocone

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ABSTRACT
Accurate characterization of soft, sensitive coastal soils can be very challenging. In the past 30 years, several empirical correlations have been established to compare in-situ test results with commonly used geotechnical parameters. Many of the most popular correlations used a very wide range of soil consistencies to develop by using a "best fit" approach. Because of this, these correlations may not be very representative for the extreme ends of the soil consistency spectrum. This paper examines and compares two of the most common in-situ exploration methods, seismic piezocone (SCPTu) and dilatometer (DMT) characterize soft sediments (s_u <60 kPa) at two sites using the established correlations.

RÉSUMÉ
La caractérisation des sols sensibles et mous peut devenir très ambigüe. Durant les 30 dernières années, plusieurs corrélations empiriques ont été établies afin de comparer les résultats des essais in situ et les paramètres de consistance généralement utilisés en géotechnique. Plusieurs des relations les plus utilisées ont été développées en utilisant la technique d'ajustement de la courbe de tendance. De cette façon, les corrélations ne sont pas toujours très représentatives, particulièrement en ce qui concerne les valeurs limites de la corrélation. Cet article examine et compare comment deux méthodes de mesure in situ, les essais au piézocônes sismiques (SCPTu) et le dilatomètre, caractérisent les sédiments mous (s_u<60 kPa) sur deux sites en utilisant les corrélations pré-établies.

1 INTRODUCTION

The seismic piezocone penetrometer and the flat plate dilatometer are widely accepted as an accurate means to characterize a soil profile. In the past thirty years, a number of correlations have been introduced that equate the field measurements into usable geotechnical parameters. The correlations have either been theoretical based (and generally proven) or were developed empirically using laboratory data. Some of the most popular are general equations designed for a wide range of soil consistencies and while proven a good "first estimation" for most consistencies, they may not be as accurate in the extreme ends of the soil consistency spectrum.

2 BACKGROUND

Historically, the characterization of soft deposits has been difficult due to many factors. Two of the main factors are resolution of measuring equipment and the influence of soil response caused by probe insertion and in-situ stresses.

2.1 Cone Penetration Test (CPTu)

Because of its speed, repeatability and accuracy, cone penetration testing has become the preferred method for characterizing a soil profile for many practitioners. The basic measurements taken during a cone penetration test are tip resistance (q_c), sleeve friction (f_s), and dynamic pore pressure (u). The most popular location for pore pressure measurement is directly behind the tip (u_2 position). This allows for correction of the tip measurements for the effects of pore pressure. This is especially important in soft, fine grained soils.

Today, the measurement resolution of the CPTu is typically no longer an issue due to advances in load cell construction and 16-bit data acquisition systems. However, excess pore pressure effects generated during probe insertion and the influence of in-situ soil stresses can continue to overshadow the soil response driven by strength and stress history in soft soils, especially in the tip measurements. Methods have been developed to account for the effects pore pressure. However, with a conventional piezocone, the relatively large influences from in-situ stresses on the tip measurements remain. To reduce the influence of in-situ stresses, Stewart & Randolph (1991) changed the tip geometry to provide a larger projected area and allowed the soils to flow around the tip. These are known as full flow penetrometers. The "flow" of the soils around the tip allowed the in-situ stresses to act essentially in all directions around the tip and thereby significantly reducing their effect on the measurements. While full flow probes have been shown to provide accurate estimations of soil strength, they are limited to extremely soft soils that will exhibit flow behaviour around the probe. They also pose installation challenges when the entire soil profile is not extremely soft.

One of the main advantages the CPTu offers is an essentially continuous profile. The soils are classified into soil behaviour types using the basic CPTu readings. In North America, the most popular method is based on charts developed by Robertson (1990) which uses normalized tip resistance, Q_t, normalized friction sleeve resistance, F_r, and pore pressure parameter, B_q. Jefferies and Davies (1993) suggested a soil behaviour type index, I_c was a useful parameter in soil classification. Later, Robertson and Wride (1998) presented a modified version of I_c defined by equation [1].

$$I_c = \left[(3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2\right]^{0.5} \quad [1]$$

One of the most fundamental parameters of geotechnical design is in-situ stresses. In-situ stresses can be determined using an estimated total unit weight, γ_T, from the seismic piezocone. Lunne et al (1997) gives a table of typical γ_T values for each soil behaviour type (SBT) value based on Robertson (1990). Mayne (2005) provides an estimation of γ_T using shear wave velocity, Vs. This is given in equation [2].

$$\gamma_T = 8.63\log(Vs) - 1.18\log(z) - 0.53 \quad [2]$$

Because the CPTu test directly measures the mechanical response of the soils, it should naturally provide means for good estimations of soil stress history and undrained strength in fine grained soils. Lunne et al (1997) presents estimations for undrained shear strength using equations [3] and [4]. The most popular correlation incorporates the corrected tip resistance, q_t and a bearing factor, N_{kt}. N_{kt} typically varies from 10 to 20.

$$s_u = \frac{q_t - \sigma_{vo}}{N_{kt}} \quad [3]$$

When the accuracy of tip measurements may be uncertain for very soft soils, Lunne et al (1997) suggested using equation [3] where $N_{\Delta u}$ varies between 6 and 10.

$$s_u = \frac{u_2 - u_o}{N_{\Delta u}} \quad [4]$$

Additionally, for NC or lightly over-consolidated soils (OCR<2), s_u can be derived by critical state soil mechanics theory using the maximum preconsolidation pressure, σ_p'. The simplified estimation is given in equation [5] and is presented in Mayne (2007).

$$s_u = 0.22\sigma_p' \quad [5]$$

Several correlations for determining the stress history of a soil by defining σ_p' were developed using spherical cavity expansion and critical state soil mechanics theories. Mayne (2009) presented simplified versions using CPTu parameters and are given by equations [6], [7] and [8].

$$\sigma_p' = 0.33(q_t - \sigma_{vo}) \quad [6]$$

$$\sigma_p' = 0.53(u_2 - u_o) \quad [7]$$

$$\sigma_p' = 0.60(q_t - u_2) \quad [8]$$

Mayne et al (1998) also showed that preconsolidation stress can be evaluated using shear wave velocity. The correlation is given by equation [9].

$$\sigma_p'(kPa) = 0.106Vs^{1.47} \quad [9]$$

where Vs is in meters per second (m/s).

2.2 Dilatometer Test

Developed by Silvano Marchetti in the 1970's, the dilatometer test consists of a flat, steel blade with a circular, steel membrane mounted on one side of the blade. At each test depth, the membrane is inflated via a flexible tube connected to a readout box at the ground surface. The initial lift off pressure, A, and expansion pressure, B, are recorded as the membrane travels through certain positions. The field measurements, A and B, are then corrected for membrane stiffness and gage offset to come up with the parameters p_o and p_1. A less used measurement, the C-reading, is taken when the membrane returns to the A-position. The C-reading is also corrected for membrane stiffness and gage offset to determine the p_2 value. These corrected values are the basis for all DMT empirical correlations. The dilatometer is recognized as a tool providing accurate estimates of soil modulus, soil strength, and stress history. The correlations to total unit weight (Mayne, 2002) and K_o (Marchetti, 1997) are also well accepted.

In the dilatometer test, the primary factors that affect measurements in soft soils are membrane stiffness and improper test procedure. In very soft soils, membrane stiffness can account for over 40% of initial lift off pressure. Not allowing pressures uphole to equilibrate with pressures at the blade by inflating the membrane too fast can cause even greater errors (especially when longer cables are used).

Softer membranes are available to reduce the effect of membrane stiffness, however they are more easily damaged by harder soils that may be encountered before the soft soils are reached. Proper training of field personnel will also reduce the effect of excessive inflation rates.

As described by Marchetti (1997), there are several correlations available to that relate the corrected test measurements to useful geotechnical parameters. The most basic is material index, I_D. It is defined by equation [10].

$$I_D = \frac{p_1 - p_o}{p_o - u_o} \quad [10]$$

For calculation of in-situ stresses, Mayne (2002) estimates the total unit weight (γ_T) using equation [11].

$$\gamma_T = 1.12\gamma_w \left(\frac{E_D}{p_a}\right)^{0.1} I_D^{-0.05} \qquad [11]$$

Where E_D is defined by equation [12] (Marchetti, 1997).

$$E_D = 34.7(p_1 - p_o) \qquad [12]$$

An intermediate parameter used in strength and stress history correlations is horizontal stress index, K_D. It is defined by equation [13].

$$K_D = \frac{p_o - u_o}{\sigma_{vo}'} \qquad [13]$$

Well established correlations for undrained shear strength have been developed by Marchetti (1980) and Schmertmann (1981). Marchetti suggests that s_u be estimated by equation [14].

$$s_u = 0.22\sigma_{vo}'(0.5K_D)^{1.25} \qquad [14]$$

And Schmertmann estimates s_u by equation [15]

$$s_u = \frac{p_o - u_o}{10} \qquad [15]$$

The dilatometer has also been shown to reliably predict stress history in some soil deposits. Marchetti (1997) presents a correlation for stress history in fine grained soils (I_D< 1.2) in equation [16].

$$OCR = (0.5K_D)^{1.56} \qquad [16]$$

Mayne (1995) reports equation [17] as an estimation of σ_p' using DMT data.

$$\sigma_{p'} = 0.51(p_o - u_o) \qquad [17]$$

3 TEST SITES AND TEST PROGRAM

The sites used in this study are Mud Bay in Surrey, BC and the Lesner site in Virginia Beach, VA. The testing program at each site consisted of seismic cone penetration testing with shear wave measurements every 0.5 meters and pore pressure dissipation every 2 meters, dilatometer testing with pore pressure dissipation test every 2 meters using the A-method and vane shear testing at select depths. Additionally, samples were taken with a Shelby tube at select depths for laboratory analysis.

3.1 Mud Bay; Surrey, BC

This soil profile consists of very soft, sensitive (sensitivity ranges between 10 and 30), fine-grained marine deposits from the ground surface to depths exceeding 30 meters. The depth to ground water is approximately 1 meter. CPTu testing indicates tip values on the order of 0.2 to 0.5 MPa and sleeve values approximately 1 to 4 kPa. Shear wave velocities, Vs, were between 40 m/s and 125 m/s. Figure 1 show the results of the CPTu testing.

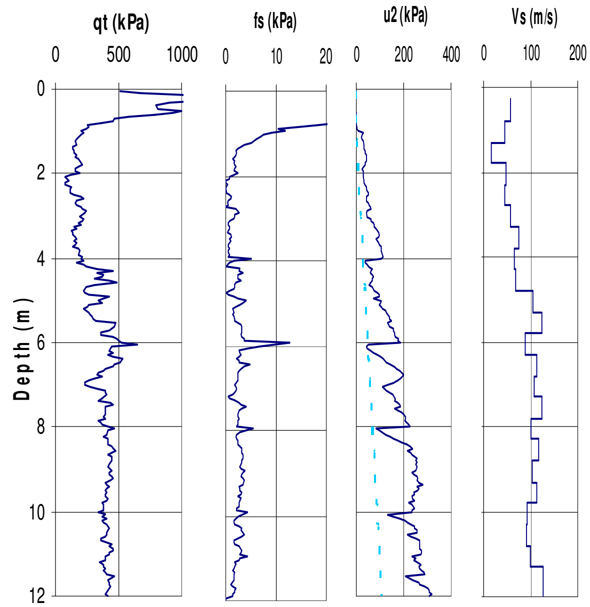

Figure 1 SCPTu results for Mud Bay Site

Pore pressure dissipation (PPD) testing was performed on 2 meter intervals during the CPTu and DMT testing. Once the dynamic testing was resumed at the conclusion of the PPD tests, the u_2 values for the CPT appeared to be reduced from the general trend of data. The values fell back in line with the general trend after the probe was pushed for approximately 0.5 meters. These dips should be noted if using correlations that use u_2 in the "recovery" interval and discretion must be used when using the values.

Lab testing performed on the samples taken from the layer of interest is summarized in Table 1. Due to the very soft consistency of the samples (all had a moisture content at or above the liquid limit), the interpretation of the results must consider that the lab test specimens may be far from "undisturbed" by the time the actual lab test was performed.

Table 1 Summary of soil properties- Mud Bay site

Sample Depth	*γ_T (kN/m3)*	*w (%)*	*LL*	*PL*	*s_u (kPa)*
3 m	15.4	50	40	27	3.9- 6.5
6 m	16.1-17.6	38	36	28	18.7- 35
11 m	14.7-16.8	63	49	29	6.8 - 10.0

3.2 Lesner Site; Virginia Beach, VA

This site is located near the mouth of the Chesapeake Bay in Virginia Beach, VA. The general soil profile consists of approximately 11 meters of primarily clean loose to dense sands (q_t = 10-40 MPa) overlying a soft, fine grained marine deposit (q_t = 0.7 to 0.9 MPa) to a depth of approximately 18 meters. The soil sensitivity of this deposit generally ranges from 5 to 10. The results presented herein are taken from the testing performed in the soft layer from a depth of approximately 11 to 18 meters. Shear wave velocities were between 130 m/s and 170 m/s. The depth to groundwater at the site is approximately 2.3 meters.

Lab testing performed on the samples taken from the layer of interest is summarized in Table 2.

Table 2 Summary of soil properties- Lesner site

Sample Depth	γ_T *(kN/m3)*	*w (%)*	*LL*	*PL*	σ_p' *(kPa)*	ϕ'	s_u *(kPa)*
12.5 m	16.9	44	54	24	129	33	67
15.5 m	17.0	46	61	25	158	-	-

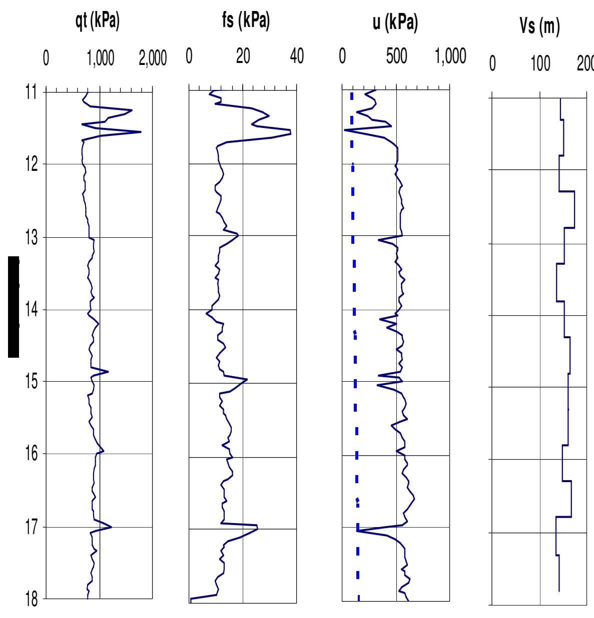

Figure 2 CPTu Results for Lesner Site

4 COMPARISON OF RESULTS

Total unit weight, stress history, undrained shear strength and published DMT/CPTu correlations were compared and matched to laboratory and field vane data.

4.1 Total Unit Weight, γ_T

Using equations [2], [11] and typical γ_T values for different SBT zones (as suggested by Lunne et al 1997), the total unit weight estimations are presented in figures 3 and 4.

The results indicate that the unit weight estimations using SBT zones overestimate the actual values at both sites. This will lead to compounding errors for in-situ stress calculations throughout the profile. For the Mud Bay site, there is a 20% difference between the DMT and SBT methods calculating total vertical stress at 12 meters depth. In the relatively firmer soils at the Lesner site, the difference is negligible (5%). The unit weight correlation based on Vs (equation [2]) compared well with the DMT estimates and lab values.

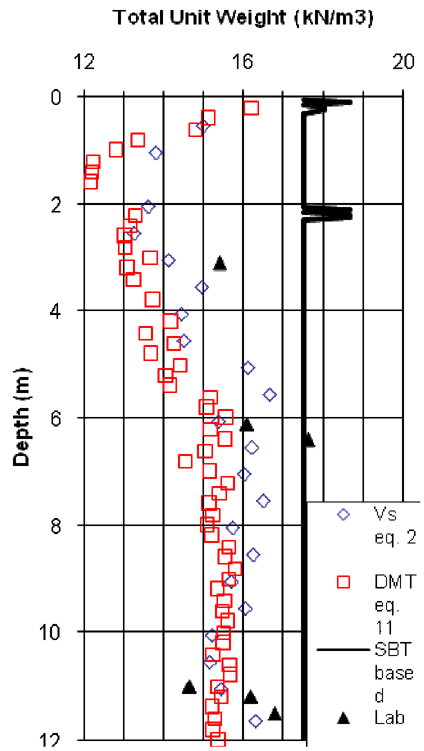

Figure 3 Total unit weight Mud Bay Site

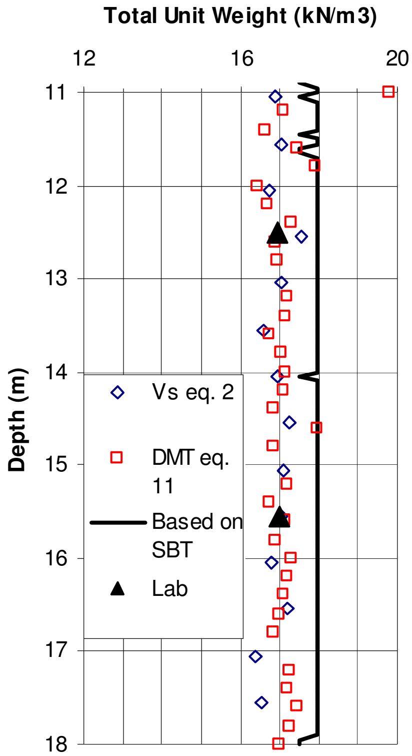

Figure 4 Total unit weight- Lesner Site

4.2 Undrained Shear Strength

Figures 5 and 6 illustrate the s_u estimations using equations [3], [4], [5], [14] and [15]. The correlations presented in equations [3] and [4] use N factors with a published typical ranges. The value of these N factors are typically chosen to match field vane or laboratory results. To illustrate each test as a "stand alone" method, the N values were not adjusted to match any strength values provided by other methods. We simply chose a mid-range value and used an N_{kt} and $N_{\Delta u}$ value of 15 and 8, respectively. The results presented using equation [5] used equation [8] for estimating σ_p'. As a comparison, s_u values determined by lab testing and field vane testing (FVT) are also presented.

The results at the Mud Bay site indicate that equations [4] (method using Δu) and [5] (method using σ_p' as a function of (q_t -u_2)) can be significantly effected by the "recovery" effects of the pore pressure dissipation tests. The s_u estimations using the CPTu correlations shows that equation [4] typically gave the lowest estimate while the estimations using equation [5] gave the highest values. The s_u estimations using the DMT data illustrate that equation [14] give slightly higher values than equation [15]. The field vane results tended to agree more with the higher range of s_u estimations while the laboratory data fell in the lower to mid-range values.

The results at the Lesner site generally follow two trends: The estimations with CPT data using equations [3] and [5] are in agreement with each other and typically lower than the estimations using the DMT data and the CPTu correlation presented in equation [4]. The s_u values from the field vane and laboratory testing fell along the upper bound of estimated strengths. Note the effect of the dissipation on the values at approximately 17 meters using equations [4] .

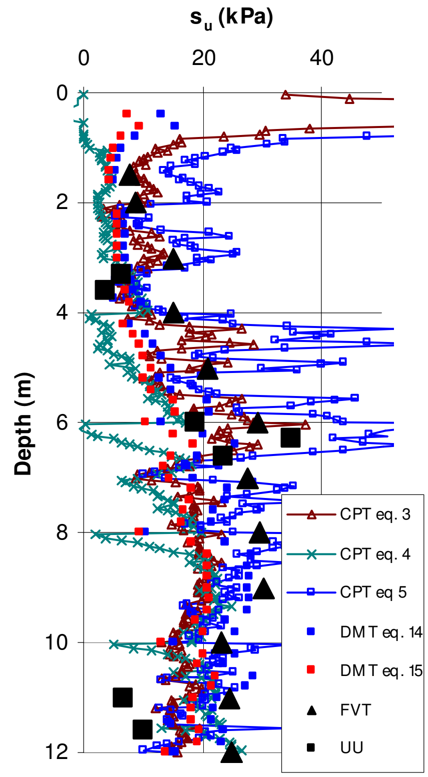

Figure 5 s_u predictions for Mud Bay Site

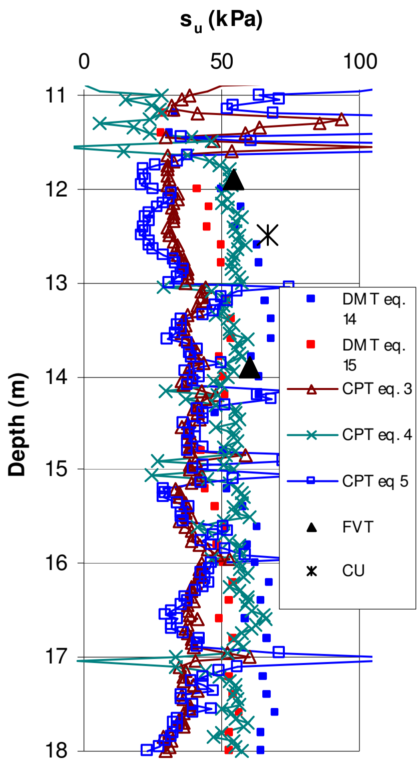

Figure 6 s_u predictions for Lesner Site

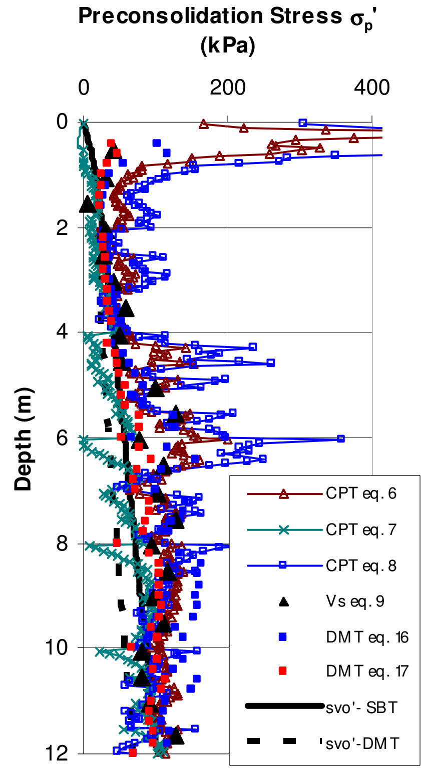

Figure 7 Preconsolidation stresses; Mud Bay Site

4.3 Stress History

Figures 7 and 8 compare calculated preconsolidation stresses for each of the given methods. For comparison purposes, the effective vertical stresses, based on SBT zones and DMT data, with depth are shown on the plots.

The estimated preconsolidation stresses for the Mud Bay site indicate OCR's of 1 to 2 for soils found at depths of 2 to 4 meters and 8 to 12 meters. In the zone from 4 to 8 meters, an OCR value of >5 would be estimated using equation [8]. This in turn also led to a higher shear strength estimations using equation [5] in Figure 5.

For the soils at the Lesner site, the preconsolidation estimates indicate a normally consolidated to slightly overconsolidated (OCR<1.3) soil. This agrees with the two consolidation tests performed.

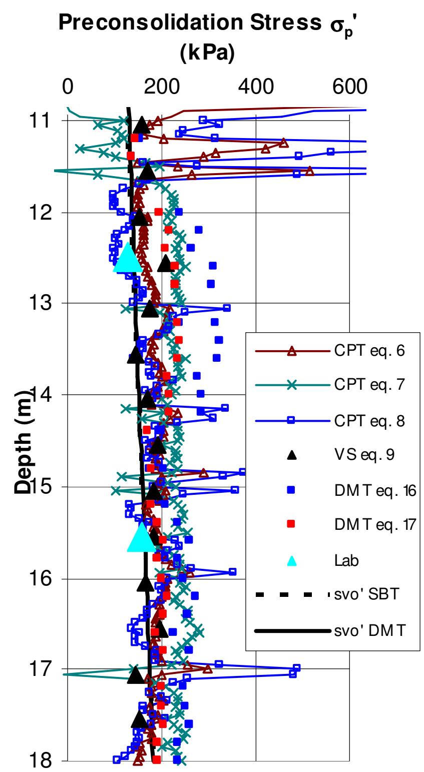

Figure 8 Preconsolidation stresses; Lesner Site

5 DISCUSSION

The correlations presented generally provide an accurate estimate of desired geotechnical parameters in soft soils. Particular attention should be given to the estimation of the total and effective stresses since these values are a part of many of the equations. In very soft soils, the use of the unit weights based on SBT zones over estimated the actual unit weights in the soils presented. Over estimated vertical stresses results in underestimated strengths and preconsolidation pressures. In firmer soils, this effect is less significant, however, in soft soils, the effect can be large.

The dynamic pore pressure can be a useful parameter when estimating s_u and σ_p', however, care must taken when using it in a continuous numerical manner if the material is not uniform and fine grained. One also must recognize the effect of allowing the pore pressures to dissipate significantly by stoppage of the probe during the test. In the extremely soft soils at the Mud Bay site, there was variation of the soils (especially from 4 to 8 meters) and there was a significant "recovery" distance before the generated pore pressures returned to a "normal" state after dissipation tests. This should be taken into account when judging the accuracy of the correlated values.

When characterizing soft soil deposits, there are many options available to the practicing engineer. Both the seismic piezocone and dilatometer can provide an accurate method for defining the required soil parameters. When analyzing data from these tests, or any other in-situ test, a number of correlations for the desired parameters should be compared. When a number of methods agree, the engineer can have confidence that the estimates are accurate.

A comprehensive test program ideally would consist of SCPTu, DMT and lab testing on high quality samples. When this is not possible, the dilatometer and seismic piezocone provide a viable stand-alone option for accurately characterizing soft soils. Based on the results given in this study, the seismic piezocone offers the most advantage due to the number of independent measurements and the ability to provide a near continuous profile.

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