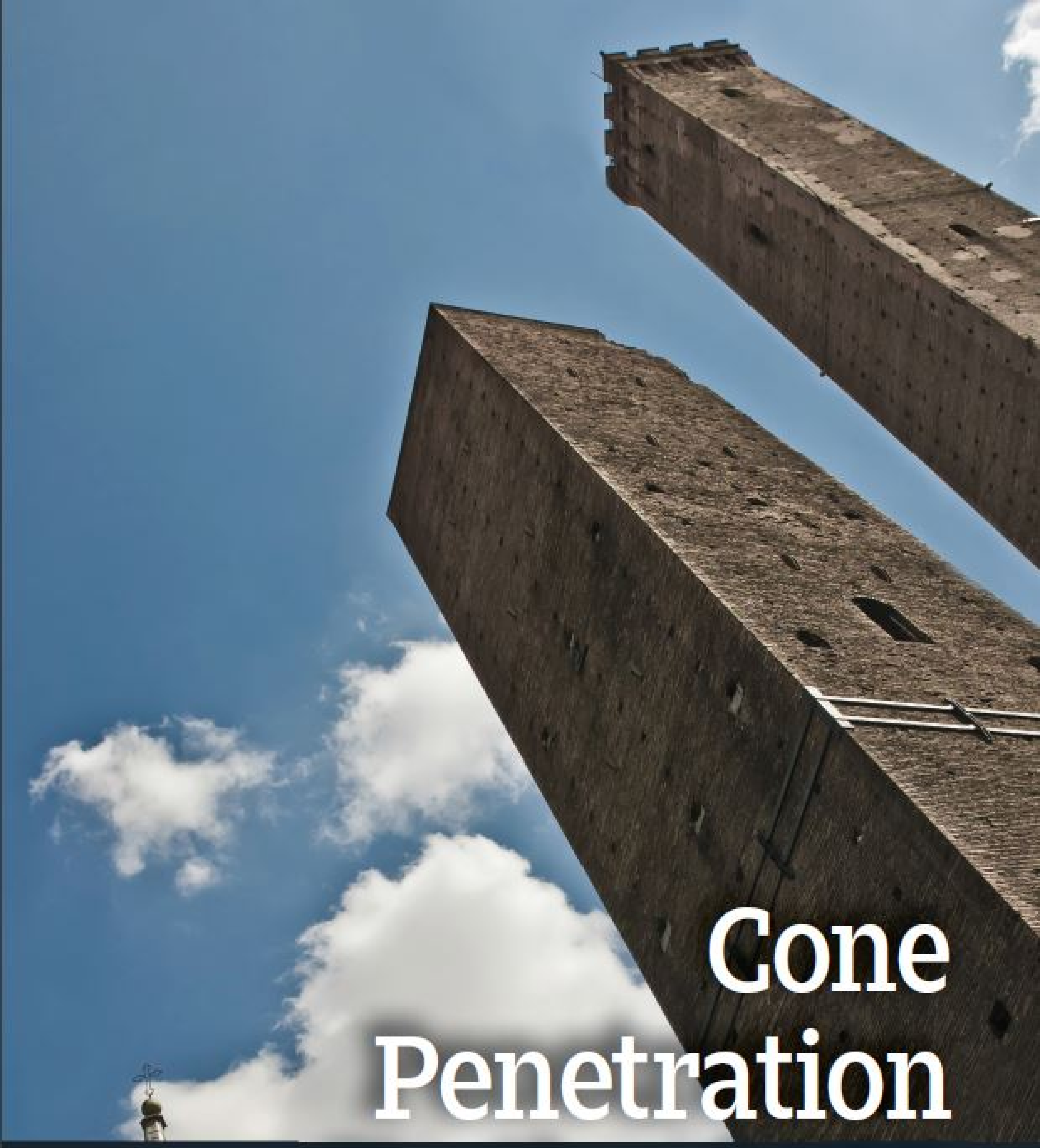

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Evaluating geoparameters of Maine sensitive clay by CPTU

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ABSTRACT: Using two sets of analytical solutions for CPTU in clays, a suite of theoretically-consistent geoparameters is shown to be in good agreement with independent laboratory and field test results obtained on sensitive Presumpscot clay in Portland, Maine, USA. Fall cone tests indicate a mean sensitivity of $S_t \approx 37$. Values of undrained rigidity index (I_R), undrained shear strength (s_u), and yield stress ratio (YSR) are provided by a modified spherical cavity expansion-critical state hybrid model while an effective stress limit plasticity solution is utilized to assess the effective friction angle of the sensitive clay at both peak strength [ϕ' at q_{max}] and also at maximum obliquity [ϕ' at $(\sigma_1'/\sigma_3')_{max}$]. A CPTU screening method that uses three simplified equations for YSR helps to identify that the clay is sensitive.

Mayne, P.W., Paniagua, P., DiBuò, B. and Agaiby, S.S. (2002). Evaluating geoparameters of Maine sensitive clay by CPTU. *Proceedings, 5th Intl. Symposium on Cone Penetration Testing* (CPT'22, Bologna), Paper ID 1110: 552-558.

1 INTRODUCTION

1.1 *Falmouth Bridge, Maine*

A geotechnical investigation for a new bridge along state highway route 26/100 at the northern portion of Portland, Maine, USA was performed by the University of Maine for the Maine Department of Transportation (Hardison & Landon 2015). The site is located to the east of Interstate I-95 and underlain by sensitive clays of the Presumpscot Formation and in close proximity to prior landslides along the Presumpscot River in the community of Westbrook (Devin & Sandford 2000), as shown by Figure 1.

The subsurface exploration for the Portland-Maine bridge included soil test borings, drive sampling, undisturbed sampling, seismic piezocone penetration test (SCPTU) soundings, and various series of laboratory tests (Langlais 2011). A detailed summary of the geotechnical data and results is provided by Hardison & Landon (2015).

1.2 *Interpretation of CPTU in clays*

The interpretation of CPTU in clays often relies on empirical correlations and simple statistical trends, although theoretical formulations also play a role. In this paper, two sets of analytical closed-form solutions are utilized so that a consistent and rational assessment is made for stress history and shear strength, both in terms of effective stress parameters (i.e., friction angle, ϕ') and total stress analysis (i.e. undrained shear strength, s_u).

The yield stress, or preconsolidation stress (σ_p') is presented in terms of the normalized yield stress ratio: YSR = σ_p'/σ_{vo}'. Conventionally, results from one-dimensional consolidation tests are taken at various elevations to develop the profile of YSR with depth in clays. Herein, a modified spherical cavity expansion-critical state soil mechanics (SCE-CSSM) hybrid model for CPTU in clays provides three YSR profiles, as well as a measure of undrained rigidity index (I_R = G/s_u), where G is the shear modulus.

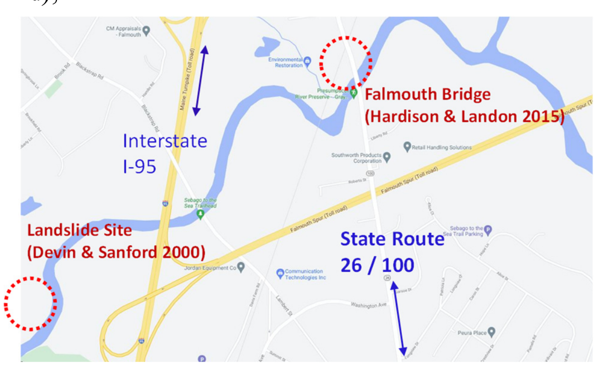

Figure 1. Locations of bridge site and landslide in soft sensitive Presumpscot clay, Portland, Maine, USA

2 CONE PENETRATION TESTS

2.1 *Piezocone soundings*

The CPTU provides three continuous readings with depth: (a) cone tip resistance, q_t; (b) sleeve friction,

f_s; and (c) penetration porewater pressure, u_2. The standard rate of advancement is 20 mm/s, although the use of variable rate CPTU soundings have been revealed to provide an effective method to characterize silts, mixed soils, and mine tailings that exhibit partially-drained behavior.

2.2 *Seismic piezocone testing*

The addition of a set of geophones to the standard penetrometer allows for downhole geophysical testing, most often at the 1-m rod breaks. A horizontal seismic source or autoseis unit is used to generate horizontally-polarized shear waves that are propagated vertically with depth. The profile of shear wave velocity (V_s) is used to obtain the small-strain shear modulus ($G_0 = G_{max}$) via elastic theory:

$$G_{max} = \rho_t \cdot V_s^2 \quad (1)$$

where $\rho_t = \gamma_t/g_a$ = total soil mass density, γ_t = total soil unit weight, and g_a = acceleration constant.

A representative SCPTU at the Portland-Maine Bridge site showing q_t, f_s, u_2, and V_s in sensitive soft clay is presented in Figure 2.

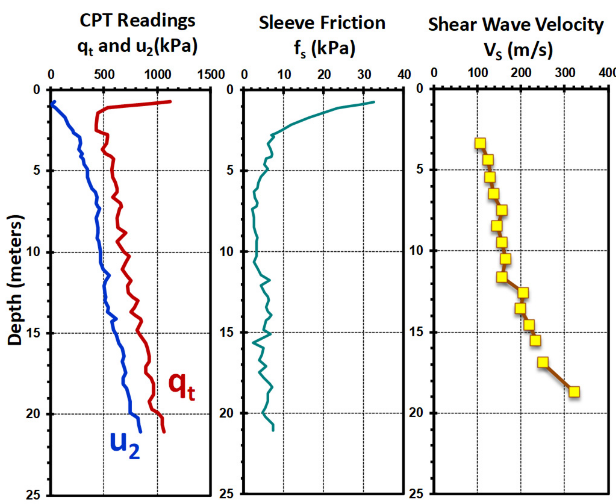

Figure 2. Profiles of q_t, f_s, u_2, and V_s from SCPTU at Route 26/100 bridge site in Portland, Maine (data from Hardison & Landon 2015)

2.3 *CPTU parameters*

It is convenient to express the CPTU results as net readings: (1) net cone resistance: $q_{net} = q_t - \sigma_{vo}$; (2) excess porewater pressure: $\Delta u = u_2 - u_0$; and (3) effective cone resistance: $q_E = q_t - u_2$; where σ_{vo} = total vertical overburden stress, u_0 = equilibrium porewater pressure; and $\sigma_{vo}' = \sigma_{vo} - u_0$ = effective overburden stress.

Furthermore, several normalized and dimensionless CPTU parameters can be defined: $Q = q_{net}/\sigma_{vo}'$, $B_q = \Delta u/q_{net}$, $U = \Delta u/\sigma_{vo}'$, and F_r (%) = $100 \cdot f_s/q_{net}$. Note that the first three of these parameters are inter-related via: $U = Q \cdot B_q$.

An update to the normalized Q is now provided with a variable exponent that depends upon soil type, termed Q_{tn}. Details are provided by Robertson (2009) and Robertson & Cabal (2015).

3 GEOPARAMETERS FROM CPTU

The evaluation of soil engineering parameters from CPTU is often addressed using empirical correlations and/or statistical trends derived from prior databases. As a result, some of the obtained values of the geoparameters are inconsistent with each other, or not well-matched well amongst each other, since they are assessed independently.

In this paper, the following geoparameters are assessed theoretically using two analytical solutions: (a) effective stress friction angle (ϕ') at both q_{max} and $(\sigma_1'/\sigma_3')_{max}$; (b) rigidity index, I_R; (c) undrained shear strength, s_u; and (d) yield stress ratio ($YSR = \sigma_p'/\sigma_{vo}'$). Thus, their values are obtained in a consistent and rational manner. Moreover, independent laboratory reference data on recovered soil samples are shown to be comparable with the CPTU evaluations.

3.1 *Index parameters of clay*

Laboratory index tests on the sensitive Presumpscot clay at the Portland-Maine site indicated: natural water content: $w_n = 43.6 \pm 7.3\%$, liquid limit: $LL = 42.1 \pm 6.9\%$, plasticity index: $PI = 17.5 \pm 5.6\%$, liquidity limit: $LI = 1.13 \pm 0.34$; and specific gravity of solids: $G_s = 2.78$. Measured unit weights gave a mean of $\gamma_t = 17.4$ kN/m^3, while natural water contents using G_s and $S = 1$ indicated a value of around 18.4 kN/m^3.

3.2 *Soil behavior type*

For soil classification by CPTU, it is common to utilize soil behavior type (SBT) charts. Hardison & Landon (2015) discuss the use of SBT that rely on Q-F and Q-B_q diagrams (Robertson & Cabal 2015). For the CPTU data at Portland-Maine Bridge, the Q-F charts primarily indicate a zone 3 soil type (clays to silty clays), with an intermingling of zone 1 (sensitive soils), as shown in Figure 3. For the Portland CPTU data, the Q-B_q chart fails to find sensitive clays, as presented in Figure 4.

A SBT chart by Schneider et al. (2012) uses Q and U to identify soil types. This approach seems to better recognize sensitive clays, as shown in Figure 5.

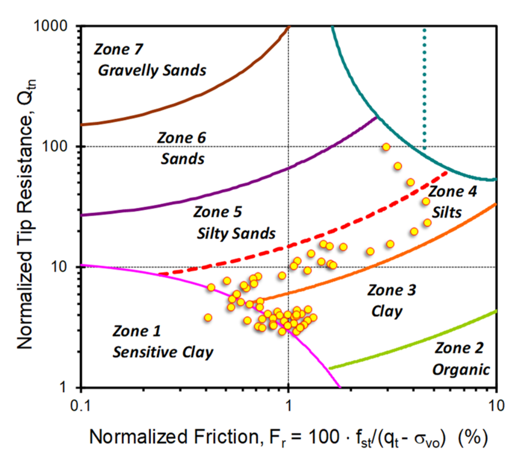

Figure 3. Portland-Maine CPTU data in Q-F soil behavior chart

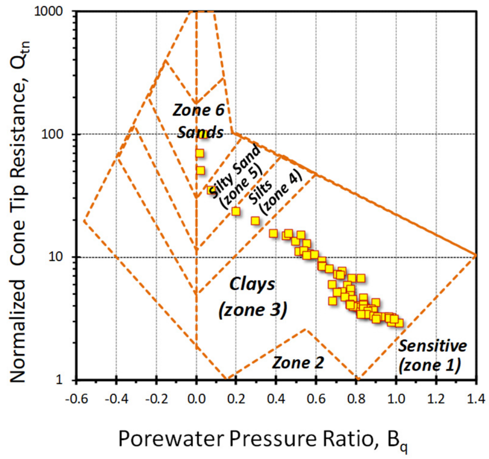

Figure 4. Portland-Maine CPTU data in Q-B_q soil behavior chart

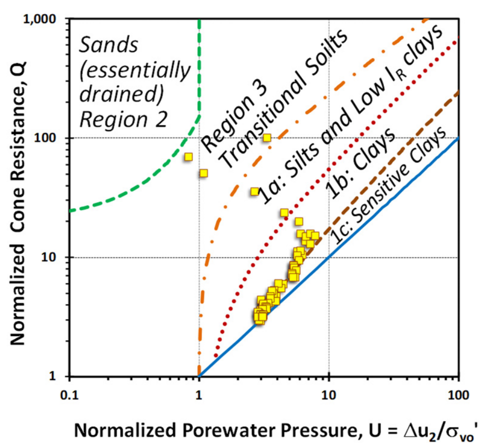

Figure 5. Portland-Maine CPTU data in Q-U soil behavior chart

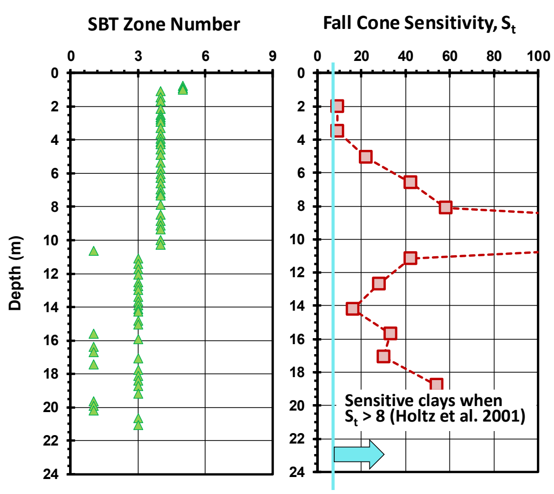

Figure 6. Profiles of SBT zone number from Q-F chart and clay sensitivity for Presumpscot clay at Portland-Maine site

In fact, clay sensitivities (S_t) measured by lab fall cone range from 9 to 268 at the Portland bridge site. Based on the guidelines discussed by Holtz et al. (2011), the clay classifies as medium to highly sensitive below depths of 6 m where $S_t > 8$, as evidenced by Figure 6.

3.3 *Screening for sensitive clays by CPTU*

In addition to SBT charts for identification of sensitive soils, a simple CPTU screening can be used, as detailed elsewhere (Agaiby & Mayne 2018, 2021; Mayne et al. 2019).

For "regular" clays that are inorganic and insensitive, the following applies:

$$0.60\ q_E \approx 0.33 q_{net} \approx 0.54\ \Delta u_2 \quad (2)$$

For sensitive clays, the following hierarchy applies:

$$0.60\ q_E < 0.33 q_{net} < 0.54\ \Delta u_2 \quad (3)$$

For the CPTU at Portland-Maine Bridge, Figure 7 shows that the hierarchy from (3) applies, thus identifying sensitive soft clay at depths below 6 m.

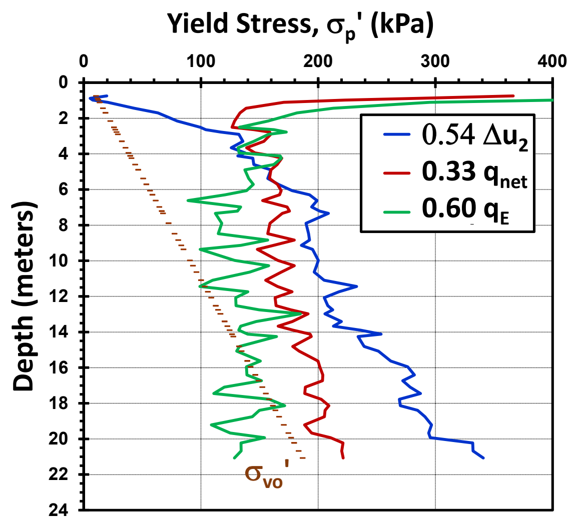

Figure 7. CPTU screening hierarchy to identify sensitive clays at Portland-Maine site

3.4 *Friction angle of sensitive Presumpscot clay*

The effective stress friction angle (ϕ') is a fundamental property of soil and an important parameter for stability analysis, foundation design, and numerical FEM simulations.

For the landslide investigation near Route 26/100, Devin & Sandford (2000) presented triaxial compression test data on soft sensitive Presumpscot clay. Figure 8 shows CK_0UC triaxial stress paths for a NC specimen, indicating a value of $\phi'_1 = 30°$ at peak strength, while at later stages of shearing, a value of $\phi'_2 = 33°$ is obtained at maximum obliquity (M.O.), defined when $(\sigma_1'/\sigma_3')_{max}$ occurs.

To obtain ϕ' in sensitive clays from CPTU, the Norwegian Institute of Technology (NTH, now NTNU) developed an effective stress limit plasticity solution for assessing ϕ' in all soil types (Janbu & Senneset 1974; Senneset et al. 1989; Sandven et al 2016). The expression for the case where $c' = 0$ and undrained

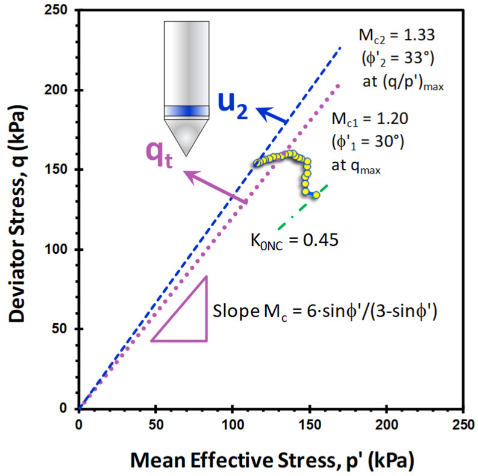

Figure 8. Triaxial stress path for Presumpscot clay at landslide site in Portland, Maine (data from Devin & Sandford 2000)

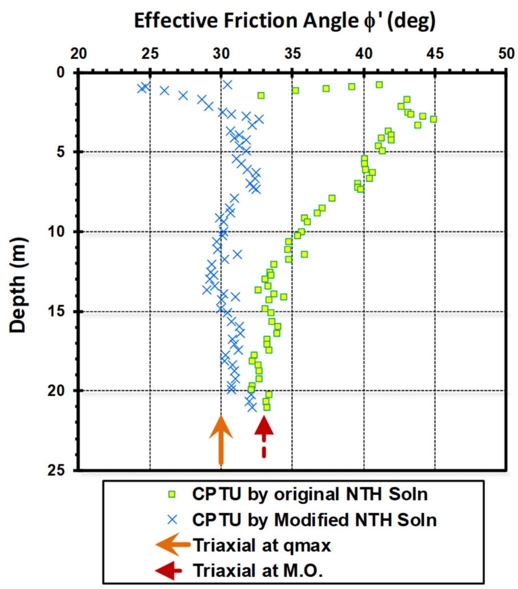

Figure 9. Profiles of ϕ'_1 and ϕ'_2 from CPTU using NTH solutions

penetration ($\beta = 0$) can be expressed:

$$Q = \frac{\tan^2(45° + \phi'/2)\cdot\exp(\pi\cdot\tan\phi') - 1}{1 + 6\cdot\tan\phi'(1+\tan\phi')\cdot B_q} \quad (4)$$

Since iteration is required, an approximate inversion to express ϕ' directly in terms of CPTU parameters Q and B_q has been devised for the following ranges: $0.05 \leq B_q < 1.0$ and $18° < \phi' < 45°$ (Mayne 2007):

$$\phi' = 29.5°\cdot B_q^{0.121}\cdot[0.256 + 0.336\cdot B_q + \log_{10}Q] \quad (5)$$

The value of ϕ' corresponding to large strains or maximum obliquity is obtained with this original NTH solution. Figure 9 shows the profile of ϕ'_2 with depth and that a value of $\phi'_2 \approx 33°$.

To obtain the value of ϕ'_1 at q_{max}, a modified NTH solution is implemented (Sandven et al. 2016; Ouyang & Mayne 2019). In this case, Q in (4) is replaced with Q' that includes stress history:

$$Q' = Q/YSR^{\Lambda} \quad (6)$$

where YSR = σ_p'/σ_{vo}' = yield stress ratio. The exponent Λ can be theoretically calculated as $\Lambda = 1 - C_s/C_c$ where C_c = compression index and C_s = swelling or recompression index, however, more often is assigned as a value $\Lambda \approx 0.7$ to 0.8 for insensitive clays and $\Lambda \approx 0.95$ to 1.0 for sensitive and quick clays (Ouyang & Mayne 2019).

At the Route 26/100 Falmouth Bridge site in Portland, Maine, the trend of YSR with depth from CRS consolidation tests indicates:

$$YSR \approx 5.12\cdot z^{-0.508} \quad (7)$$

where z = depth (meters). The CRS results will be presented later in the paper.

Adopting $\Lambda = 0.95$ for soft sensitive Presumpscot clay and using Q' from (6) in (5), the profile of ϕ'_1 with depth is shown in Figure 9. From depths between 8 to 20 m, the CPTU value more or less agrees with the CK_0UC value $\phi'_1 = 30°$.

3.5 *Rigidity index*

The undrained rigidity index is defined as $I_R = G/s_u$ where G = shear modulus and s_u = undrained shear strength. The difficulty here is that the magnitude of G ranges greatly, from a very high value at the non-destructive range at G_{max} to a low value at failure (G_f) corresponding to peak strength.

In many instances, empirical correlations for estimating I_R are used. A UC-Berkeley method developed from triaxial tests on clays by Keaveny & Mitchell (1986) relates I_R with plasticity index (PI) and YSR, for which an approximate expression is available (Mayne 2007).

$$I_{R50} \approx \frac{\exp[(137 - PI)/23]}{1 + \ln_e\left[(YSR+1)^{3.2}/26 + 1\right]^{0.8}} \quad (8)$$

which applies when: $10 < PI < 50$ and $YSR < 10$. For the range of PI at Portland-Maine Bridge ($9 < PI < 28$), this indicates an $120 < I_R < 250$.

A UC-Davis approach uses results from SCPTU to obtain a mobilized stress level based value of I_R at 50% strength. In this case, I_{R50} is obtained from:

$$I_{R50} = \frac{1.81\cdot G_{max}}{(q_{net})^{0.75}(\sigma_{vo}')^{0.25}} \quad (9)$$

where G_{max}, q_{net}, and σ_{vo}' are all in same units. At the Portland, Maine site, this approach gives a range: $148 < I_R < 260$.

A spherical cavity expansion - critical state soil mechanics (SCE-CSSM) model for CPTU in sensitive clays provides the direct assessment of I_R (Agaiby & Mayne 2018):

$$I_R = \exp\left[\frac{1.5 + 2.925\cdot M_{c1}\cdot a_q}{M_{c2} - M_{c1}\cdot a_q}\right] \quad (10)$$

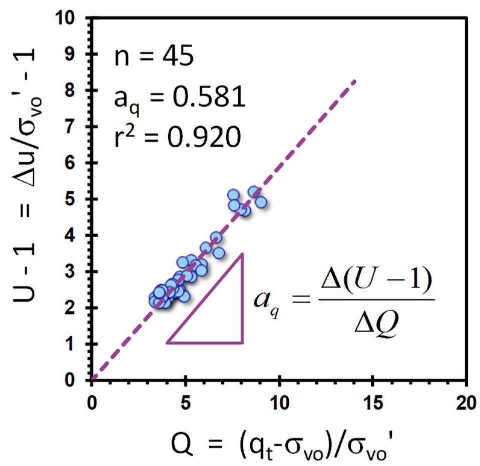

Figure 10. Plot of U-1 versus Q to obtain slope parameter a_q for evaluating rigidity index of Presumpscot clay

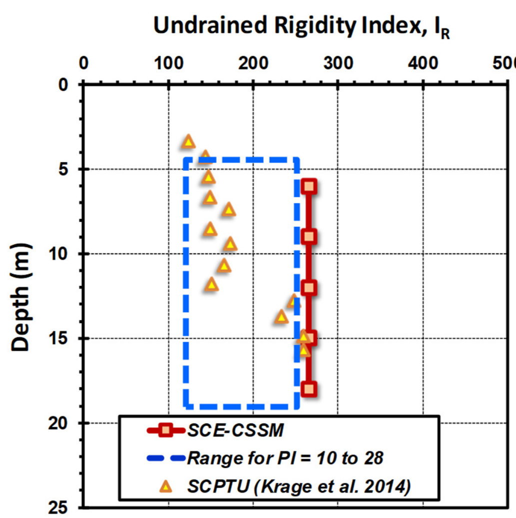

Figure 11. Undrained rigidity index profiles at Portland site

where $M_c = 6\cdot\sin\phi'/(3-\sin\phi')$ is the frictional parameter in Cambridge q-p' space and a_q is obtained as the slope of (U-1) versus Q. The value of M_{c1} corresponds to ϕ'_1 at q_{max} while M_{c2} is associated with ϕ'_2 at large strains.

For the Portland site, a slope parameter $a_q = 0.581$ is obtained, as shown in Figure 10. Using the corresponding values of $M_{c1} = 1.20$ and $M_{c2} = 1.33$ gives a calculated $I_R = 266$.

The three approaches for I_R are presented in Figure 11. The upper bounds for the UCB and UCD methods imply that $I_R = 266$ from SCE-CSSM solution is reasonable.

3.6 *Yield stress ratio at Portland-Maine site*

The SCE-CSSM provides three expressions for YSR in clays that are functions of either Q or U, as well as both Q and U (Di Buò, et al. 2019):

$$YSR = 2\cdot\left[\frac{Q/M_{c1}}{1.95+0.667\cdot\ln_e I_R}\right]^{1/\Lambda} \tag{11}$$

$$YSR = 2\cdot\left[\frac{U-1}{0.667\cdot M_{c2}\cdot\ln_e I_R - 1}\right]^{1/\Lambda} \tag{12}$$

$$YSR = 2\cdot\left[\frac{Q-(M_{c1}/M_{c2})\cdot(U-1)}{1.95\cdot M_{c1}+(M_{c1}/M_{c2})}\right]^{1/\Lambda} \tag{13}$$

The input parameters of Q and U, together with $M_{c1} = 1.20$, $M_{c2} = 133$, $\Lambda = 0.95$, and $I_R = 266$ are used to generate three profiles of YSR. The associated yield stresses (σ_p') are consistent and shown to be in good agreement with CRS consolidation tests on undisturbed samples from the site, as seen in Figure 12a.

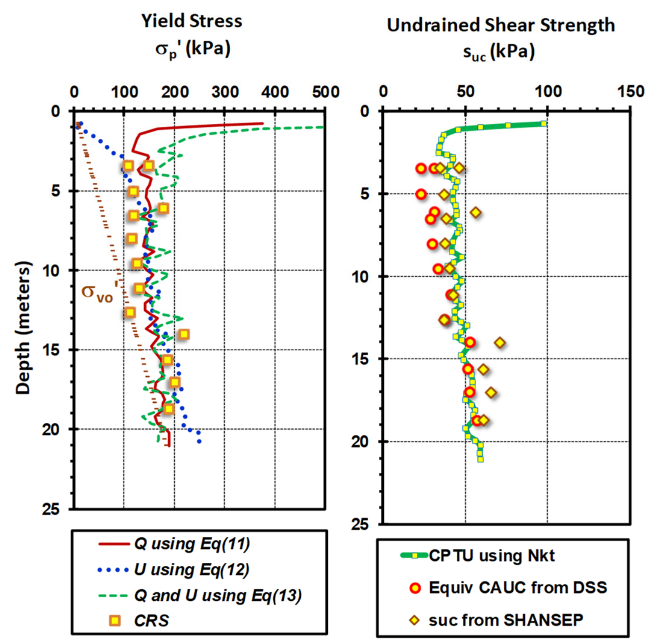

Figure 12. Profiles of yield stress and undrained shear strength for sensitive Presumpscot clay in Portland, Maine (reference lab data from Hardison & Landon 2015).

3.7 *Undrained shear strength*

For the Portland, Maine site, a series of constant volume direct simple shear (DSS) tests were performed (Langlais 2011). These results can be converted to an equivalent triaxial compression mode (CK_0UC) via a simple relationship that depends on ϕ' (Mayne 2008):

$$s_{uDSS}/s_{uc} = 0.65 \tag{14}$$

Alternatively, a SHANSEP type approach can be used where (Hardison & Landon 2015):

$$s_{uc} = S\cdot YSR^{\Lambda}\cdot\sigma_{vo}' \tag{15}$$

where S = 0.33 and $\Lambda = 0.95$. Here, the results of the CRS consolidation tests provide the YSR.

For the CPTU, the undrained shear strength is a triaxial compression mode such that:

$$s_{uc} = q_{net}/N_{kt} \tag{16}$$

where N_{kt} is a cone bearing factor obtained from SCE theory (Vesić 1977):

$$N_{kt} = 4/3\cdot(\ln_e I_R+1) + \pi/2 + 1 \quad (17)$$

For a value of I_R = 266, the calculated bearing factor is N_{kt} = 11.35.

All three evaluations of s_{uc} for soft sensitive clay at Portland are presented in Figure 12b with reasonable agreement shown for all profiles.

4 ADDITIONAL CASE STUDIES

These analytical CPTU solutions have also been successfully applied to a number of other sensitive clays, including Finland (Di Buò, et al. 2019), Norway (D'Ignazio, et al. 2019; Mayne et al. 2019), USA (Mayne & Benoît 2020), and Canada (Agaiby & Mayne 2018; Agaiby et al. 2021).

5 CONCLUSIONS

Two sets of analytical solutions for CPTU in soft sensitive clays are applied to a case study involving the Presumpscot Formation in Portland, Maine. A consistent and theoretical assessment is made for the geoparameters of the clay, including effective stress friction angle (ϕ' at q_{max} and ϕ' at M.O.), rigidity index (I_R), undrained shear strength (s_{uc}), and yield stress ratio (YSR). The values from these closed-form solutions are in general agreement with results from laboratory testing on undisturbed samples, including CRS-type consolidation tests, triaxial compression tests, and direct simple shear. A simple means for screening to identify sensitive clays from "regular" insensitive and inorganic clays is also presented.

6 ACKNOWLEDGMENTS

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