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Piezocone evaluation of geoparameters for sensitive clays of New England

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ABSTRACT: Piezocone soundings in sensitive clay in the New England portion of the United States, specifically Maine, New Hampshire, and Massachusetts, have been utilized to provide a suite of soil parameters. The data allow for the application of two closed-form analytical solutions based on: (1) effective stress limit plasticity; and (2) hybrid cavity expansion-critical state methods. The interpreted geoparameters include: effective stress friction angle (ϕ'), undrained shear strength (s_u), Skempton's pore pressure parameter (A_f), yield stress ratio (YSR), and undrained rigidity index (I_R). Profiles of these soil parameters are compared with benchmark values obtained from series of available laboratory testing programs, including index testing, triaxial compression, and one-dimensional consolidation. If piezocone dissipation tests are conducted in the field, the resulting data can be used to assess the coefficient of consolidation and soil permeability. A soil behavioral chart of Q versus U is used to identify sensitive clays.

KEYWORDS: clay, cone penetration, friction angle, piezocone, sensitive soils, undrained shear strength, yield stress ratio.

1 INTRODUCTION

Sensitive clays are well-associated with ground instability, particularly landslides, foundation performance issues, and related construction difficulties; thus, they can be considered a type of problematic geomaterial (L'Heureux et al., 2014). Consequently, it is paramount that sensitive clays are identified adequately during the geotechnical site investigation phase.

With the increasing use of piezocone penetration testing (CPTU), a reliable means of assessing whether a clay is of low sensitivity versus a clay that is highly sensitive to quick must be established and verified. Current conventional CPTU practice for this purpose has relied on empirical soil behavioral type (SBT) charts (Lunne et al. 1997; Niazi 2021). However, it has been well-documented that this method can often falsely misclassify high-sensitivity clays as either regular clay, silt, or organic soil (e.g., Shahri et al., 2015; Sandven et al., 2016; DeGroot et al., 2019; Agaiby et al., 2021; Mayne et al., 2024).

Once properly identified as sensitive clay, it is also essential to have a systematic procedure for interpreting key geoparameters necessary for analyzing slope stability, foundation bearing capacity, and excavations, as well as for numerical finite element studies of the planned project.

Towards these purposes, two sets of analytical solutions are available: (1) effective stress limit plasticity theory that provides the values of effective friction angle at both the peak strength and at large strains (Sandven et al. 2016); (2) hybrid spherical cavity expansion - critical state soil mechanics (SCE-CSSM) framework that supplies the undrained shear strength, stress history, and rigidity index (Agaiby & Mayne 2021).

The interpretation of geoparameters is illustrated through examples from three deposits of soft, sensitive clays in New England, including sites in Portland, Maine; Dover, New Hampshire; and Saugus, Massachusetts.

2 CONE PENETRATION TESTING

A representative cone penetrometer sounding obtains three continuous readings with depth: (a) cone tip resistance, q_t; (b) sleeve friction, f_s; and (c) penetration porewater pressure, u_2; thus, it is termed a piezocone penetration test (CPTU).

In addition to the direct CPTU readings, post-processing of the digital data includes: net cone resistance: (a) $q_n = q_t - \sigma_{vo}$, (b) excess porewater pressure: $\Delta u = u_2 - u_0$, and (c) effective cone resistance: $q_E = q_t - u_2$. Normalized and dimensionless piezocone parameters are also utilized, including: $Q = q_n/\sigma_{vo}'$, $U = \Delta u/\sigma_{vo}'$, $F = 100\ f_s/q_n$ (%), $Q_E = q_E/\sigma_{vo}'$, and the pore pressure ratio, $B_q = \Delta u/q_n$ which is also found as $B_q = U/Q$. The parameter Q is also designated as Q_t and Q_{t1} in the open geotechnical literature. Details on these various parameters are found in Mayne et al. (2023).

3 GEOPARAMETER EVALUATION

The assessment of reasonable geotechnical engineering parameters is critical and should proceed in a proper manner. The geoparameters include: rigidity index (I_R), undrained shear strength (s_{uc}), effective stress friction angle (ϕ'), and yield stress ratio ($YSR = \sigma_p'/\sigma_{vo}'$), where σ_p' = effective preconsolidation or yield stress. In addition, the results of CPTU dissipation tests enable the evaluation of flow parameters, including in-situ values of the coefficient of consolidation (c_v) and hydraulic conductivity (k) at selected test depths (Mayne et al., 2023).

3.1 *Undrained rigidity index*

The undrained rigidity index (I_R) is defined as the ratio of shear modulus to undrained shear strength; thus, $I_R = G/s_{uc}$. From the hybrid SCE-CSSM analytical CPTU model, the value of I_R mobilized at peak strength (i.e., $I_{R\text{-}100}$) in sensitive clay deposits is determined from:

$$I_R = exp\left[\frac{1.5+2.925M_{c1}\cdot a_q}{M_{c2}-M_{c1}\cdot a_q}\right] \tag{1}$$

where the parameter a_q is found as the slope of the difference ($u_2 - \sigma_{vo}$) versus q_n (Agaiby & Mayne 2021). Alternatively, a_q is found as the slope of (U-1) versus Q, or if the mean values of both Q and B_q are already determined for a clay layer, then $a_q = B_q - 1/Q$. The frictional parameter M_c corresponds to triaxial compression and relates to the effective friction angle: M_c =

6·sinϕ'/(3-sinϕ'). Specifically, M_{c1} = frictional parameter defined at peak strength (i.e., q_{max}) while M_{c2} = frictional parameter either at large strains or for a definition at maximum obliquity (M.O.), i.e. $(\sigma_1'/\sigma_3')_{max}$.

The parameter a_q also serves as an indication of the level of clay sensitivity that can be used independently of the common SBT charts in practices, such as the zones of the Q-F and Q-B_q series (e.g., Lunne et al., 1997). As such, clays of high sensitivity occur when $a_q > 0.5$, while clay soils of low to medium sensitivity are found when $a_q \leq 0.5$ (Di Buö et al., 2019; Paniagua et al., 2024; Mayne et al., 2024).

3.2 *Undrained shear strength*

For CPTU in clays, the undrained shear strength relates to the net cone tip resistance: $q_n = (q_t - \sigma_{vo})$ using the classic bearing capacity equation:

$$s_{uc} = \frac{q_n}{N_{kt}} \quad (2)$$

where N_{kt} is the cone-bearing factor. Here, the undrained shear strength corresponds to a triaxial compression mode, and the factor is simply a function of the rigidity index (Vesic 1977):

$$N_{kt} = 1.33\ln(I_R) + 3.90 \quad (3)$$

3.3 *Yield stress ratio*

The SCE-CSSM solution for CPTU in clay provides three separate expressions for yield stress ratio (Agaiby & Mayne 2018; Di Buö et al. 2019; Mayne et al. 2022):

$$YSR = 2\left[\frac{Q/M_{c1}}{0.667\cdot\ln(I_R)+1.95}\right]^{\left(\frac{1}{\Lambda}\right)} \quad (4)$$

$$YSR = 2\left[\frac{U-1}{0.667\cdot M_{c2}\cdot\ln(I_R)-1}\right]^{\left(\frac{1}{\Lambda}\right)} \quad (5)$$

$$YSR = 2\left[\frac{Q-\left(M_{c1}/M_{c2}\right)(U-1)}{1.95\cdot M_{c1}+\left(M_{c1}/M_{c2}\right)}\right]^{\left(\frac{1}{\Lambda}\right)} \quad (6)$$

where Λ = 1 for sensitive clays and assumes values of about 0.7 to 0.8 for triaxial compression in clays of low to medium sensitivity. If all parameters are properly evaluated, the three derived profiles of YSR will agree with each other and with profiles developed from laboratory consolidation tests.

3.4 *Effective friction angle*

An effective stress limit plasticity theory for CPTU allows the determination of the effective stress friction angle for all soil types (Senneset et al. 1989). The method was developed at the Norwegian Institute of Technology (NTH). For undrained penetration in clays that are uncemented (c' = 0), the rigorous solution is given by:

$$Q = \frac{\left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)[\exp(\pi\cdot\tan\phi')]-1}{1+(6\cdot\tan\phi')(1+\tan\phi')\cdot B_q} \quad (7)$$

which can be solved by iteration. Alternatively, an approximate direct solution can be used:

$$\phi' \approx 29.5°(B_q)^{0.121}\cdot\left[0.256+0.336B_q+\log Q\right] \quad (8)$$

which applies for the following ranges of friction angle: 18° ≤ ϕ' ≤ 45° and pore pressure ratio: $0.05 < B_q < 1.0$.

A modified NTH solution has been recommended for overconsolidated clays (Sandven et al. 2016) and simply replaces the Q in the above two equations by use of Q' that is defined by:

$$Q' = Q/YSR^{\Lambda} \quad (9)$$

The friction angle from the modified NTH solution corresponds to the value of ϕ_1' defined at peak strength (i.e., q_{max}), whereas the original NTH solution provides ϕ_2' at large strains.

4 SENSITIVE CLAYS OF NEW ENGLAND

Deposits of sensitive marine clays are found in New England in the northeastern USA. Herein, the noted analytical solutions are applied to CPTUs in the sensitive Presumpscot Formation (DeGroot et al. 2019) at three locations: (a) Portland, ME; (b) Dover, NH and (c) Saugus, MA.

4.1 *Portland, Maine*

A representative CPTU sounding (No. 305) for the Portland, Maine site is presented in Figure 1 (Hardison & Landon 2015).

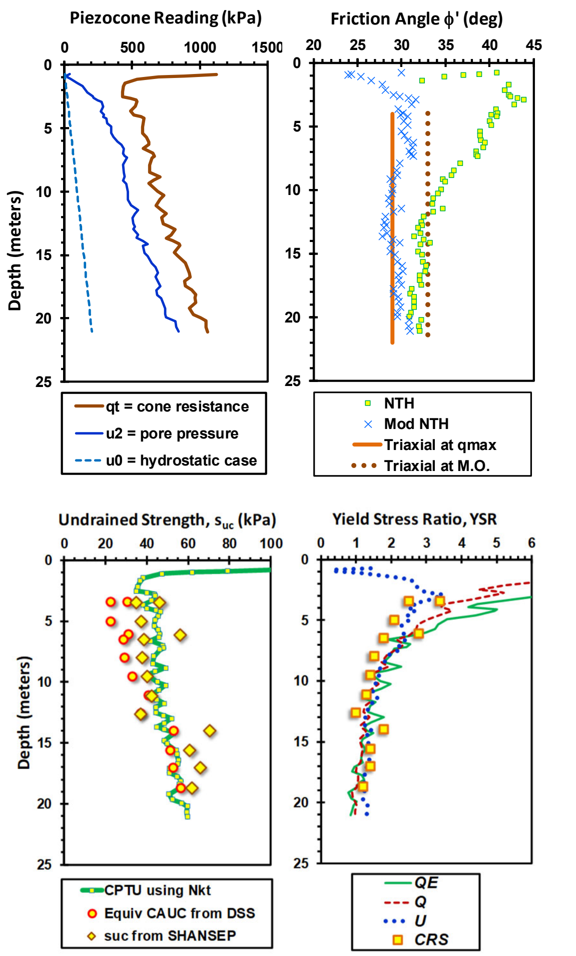

Figure 1. Profiles in soft sensitive clay at the Portland, Maine site.

The geotechnical investigation for a new bridge was located along State Highway Route 26/100 and east of interstate I-95 in Portland, Maine and conducted by the University of Maine.

Laboratory index tests on the sensitive Presumpscot clay at the Portland-Maine site indicated: natural water content: w_n = 43.6 ± 7.3%, liquid limit: LL = 42.1 ± 6.9%, plasticity index: PI = 17.5 ± 5.6%, liquidity limit: LI = 1.13 ± 0.34; and specific gravity of solids: G_s = 2.78. Measured unit weights gave a mean of γ_t = 17.4 kN/m^3. Sensitivities at the Portland site were evaluated using lab fall cone tests with S_t ranging from 16 to 100+ and averaging 59.

Post-processing of the CPTU data yielded a value of a_q = 0.58 and a corresponding I_R = 192, using ϕ_1' = 29° and ϕ_2' = 33° from triaxial compression tests (Mayne et al. 2022). These in turn provided profiles of s_{uc} and YSR that agreed well with advanced lab testing, as shown in Figure 1.

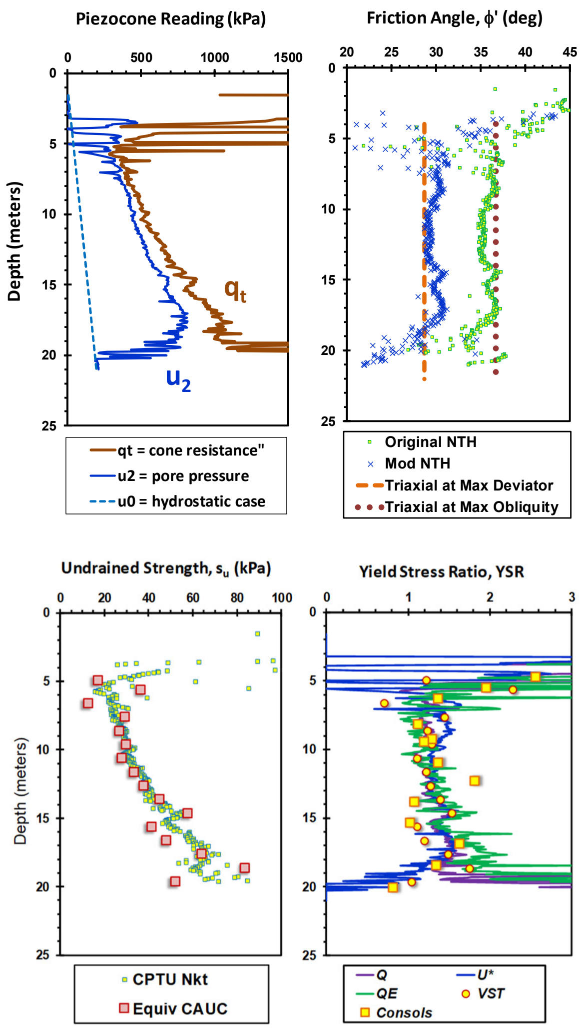

Figure 2. Profiles in sensitive clay at the Dover, New Hampshire site.

4.2 *Dover, New Hampshire*

A test embankment for the New Hampshire Department of Transportation required site investigations performed along State Route 4 to the Spaulding Turnpike near Dover, NH. The subsurface conditions were explored using flat dilatometer tests (DMT), piezocone soundings (CPTU), vane shear tests (VST), and rotary soil borings with sampling and complementary laboratory testing (Getchell et al. 2014).

The subsurface conditions consist of an upper 6 m of variable alluvium, comprising sands, silts, and clay, underlain by soft marine clays from 6 to 18 m, and further underlain by silty to sandy glacial sediments. Groundwater lies about 1 m deep at the site. Laboratory testing on the recovered clay samples at Dover included index tests and one-dimensional consolidation tests. The laboratory index parameters yielded the following mean values: void ratio e_0 = 1.2, natural water content w_n = 41%, liquid limit LL = 38%, plastic limit PL = 13%, plasticity index PI = 14%, and liquidity index LI = 1.5. The total soil unit weight averages 17.4 kN/m³ in the clay.

In the depth range from 7 m to 18 m, the measured VST sensitivities ($S_t = s_{uv}/s_{ur}$) range from 8 to 36 at the site, with a mean value of $S_t \approx 25$, indicating sensitive clay.

Post-processing of the CPTU data indicated a_q = 0.70 which determined I_R = 260 using triaxial compression test friction angles: ϕ_1' = 28.7° and ϕ_2' = 36.7° (Mayne & Benoît 2020). The derived profiles of s_{uc} and YSR are presented in Figure 2, showing consistent results in agreement with available field and laboratory benchmark values.

4.3 *Saugus, Massachusetts*

Boston Blue Clay (BBC) is a marine deposit that has undergone numerous environmental and geological processes, resulting in a sensitive clay structure (DeGroot et al. 2019). Based on comprehensive laboratory testing at the Saugus site reported by Whittle et al. (2001), mean indices in the clay gave: w_n = 40%, LL = 45%, PI = 23%, γ_t = 18 kN/m³, and G_s = 2.81. Liquidity indices ranged from 0.7 to 1.7. A series of CRS consolidation tests showed the clay layers below 24.5 m are normally consolidated (NC) to very lightly overconsolidated (LOC).

Mechanical vane tests indicated that S_t values range from 5 to 9 in the soft NC clay portion, thus classified as "medium sensitive" per Baligh & Levadoux (1981). However, several soil samples showed liquidity indices LI > 1.0 which is generally an indicator of clays of high sensitivity. Note that at the nearby Newbury MA test site, located about 45 km north of Saugus, the soft sensitive BBC is much shallower (z < 10 m) with S_t in the range of 20 to 30, as measured by fall cone.

The CPTU data determined the parameter value a_q = 0.64, which together with the CAUC data with mobilized effective friction angle at q_{max}: ϕ_1' = 24° and at $(\sigma_1'/\sigma_3')_{max}$: ϕ_2' = 32°, determined an operational value I_R = 170 (Agaiby & Mayne 2022). Figure 3 shows the interpreted profiles at the Saugus site, including (a) CPTU q_t and u_2; (b) effective friction angles from CAUC and NTH solutions; (c) undrained shear strength, s_{uc}; and (d) YSR with depth.

5 CPTU SOIL BEHAVIORAL CHARTS

There are over 30 different methods for assessing soil classification using the CPTU, mainly presented in the form of soil behavioral type (SBT) charts (Niazi 2021). Several of these have zones intended to identify sensitive soils, having varied degrees of success (e.g., Shahri, et al. 2015; Hardison & Landon 2015; DeGroot et al. 2019; Agaiby & Mayne 2021).

Of particular interest, the SBT chart using Q versus U developed by Schneider et al. (2008) distinguishes CPTU data into five separate zones of soil types. Figure 4 shows the data from the three Presumpscot clay sites of New England, all of which fall within the "sensitive clay" zone.

6 CONCLUSIONS

The proper and rational interpretation of CPTU in sensitive clays is crucial, as these soils often exhibit instability, landslides, and potential collapse during construction. Herein, two analytical closed-form solutions (NTH and SCE-CSSM) are utilized for three case studies involving sensitive clays in New England. The careful evaluation of select geoparameters includes: rigidity index (I_R), undrained shear strength (s_{uc}), yield stress (σ_p'), and effective stress friction angle (ϕ'). Results compare favorably with available benchmark values from companion field and lab tests on these sensitive clays.

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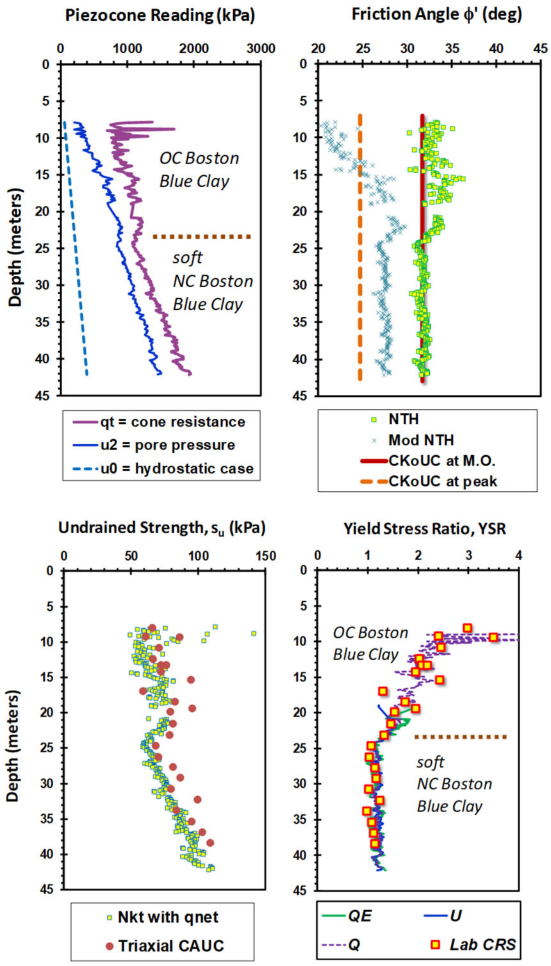

Figure 3. Profiles in Boston Blue clay at Saugus, Massachusetts site.

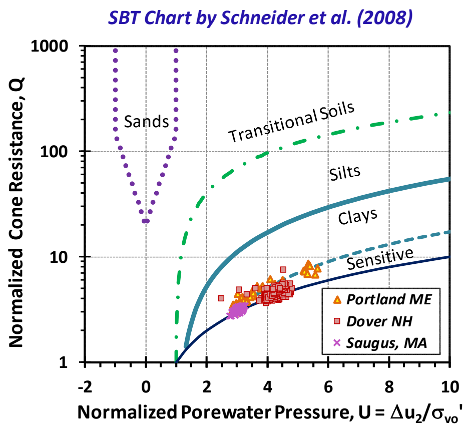

Figure 4. The SBT Q-U chart with CPTU data from New England clays.

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