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Application of two analytical CPTU solutions to sensitive clay in Québec

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ABSTRACT

A recent series of CPTU soundings in sensitive clay at the Louiseville Quebec test site has been published, allowing for the application of two closed-form analytical solutions based on: (1) effective stress limit plasticity and (2) hybrid cavity expansion-critical state methods. These are used for geoparameter interpretations including: effective stress friction angle, undrained shear strength, yield stress ratio, undrained rigidity index, and the coefficient of consolidation. Profiles of these soil parameters are compared with the benchmark values obtained from available series of previous and present laboratory testing programs at this site, including index testing, triaxial compression, and one-dimensional consolidation. Piezocone dissipation tests are used to assess the coefficient of consolidation which are validated by pressuremeter holding tests and lab oedometer tests. An empirical CPTU screening method identifies the Louiseville site as underlain by sensitive clay, versus a regular insensitive clay deposit, whereas soil behavior type charts using Q-F-B_q indicate the soils are either silt (zone 4) or regular clay (zone 3). The alternate screening method is verified by field vane and laboratory fall cone that show the clay is in fact sensitive (average $S_t \approx 22$).

Keywords: clay, cone penetration, friction angle, piezocone, sensitivity, undrained strength, yield stress

1. Introduction

Sensitive clays are problematic geomaterials as they are well-associated with ground instability, landslides, foundation performance issues, and related construction difficulties. Therefore, it is important and paramount that sensitive clays are properly identified 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-medium 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). However, it has been observed and well-docmented that this approach can often mis-classify sensitive 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. 2022).

Once properly identified as sensitive clay, it is also essential to have a systematic methodology for the interpretation of the key geoparameters needed in the analyses of slope stability, foundation bearing capacity, and excavations, as well as numerical simulations of the planned project.

Herein, a recent series of CPTU soundings at the well-known sensitive clay site in Louiseville, Québec (Dourlet 2020) are examined as a case study to detail the aforementioned issues and present the applications of two analytical solutions in evaluating stress history, rigidity index, and both total and effective soil strengths.

In addition, alternative means to common SBT charts are described for assessing whether the clay is of low or high sensitivity using CPTU results.

2. Case study at Louiseville, Québec

Louiseville is a town located on the north side of the Saint Laurence River about mid-way between Montreal and Québec City. For over four decades, the site at Louiseville has been utilized as a geotechnical testing site for improved understanding and research on soil behavior (Hamouche et al. 1990; Leroueil et al. 2003). The site is underlain by sensitive clay as belonging to the Champlain Sea deposits, or Leda clay formation of eastern Canada.

Laboratory testing of the clay at Louiseville include the following summary: specific gravity of solids (G_s) = 2.78, natural water contents (w_n) decreasing from 90% to 64% with depth, plastic limits (w_p) from 23% to 26%, liquid limits (w_L) from 72% to 62%, average plasticity index $I_p \approx 45\%$, average clay fraction (CF < 0.002 mm) of 80%, liquidity index (I_L) from 1.2 to 1.4, and calcium carbonate content ≈ 4%. Fall cone tests (FCT) indicate a sensitivity $S_t \approx 22$. Results of one dimensional consolidation tests on undisturbed samples show an overconsolidation difference (OCD = $\sigma_p' - \sigma_{vo}'$) that is nearly constant with depth at approximately 100 kPa. Corresponding profiles of yield stress ratio (YSR), also termed the overconsolidation ratio (OCR = σ_p'/σ_{vo}'), decrease from about 5 to 3 in the upper 15 meters.

Much of the geotechnical research has been conducted by Laval University and the Ministère des Transport du Québec (MTQ). Additional details on the results of field and laboratory testing of Louiseville clay are published elsewhere (Hamouche et al. 1995a, 1995b), Tanaka et al. (1998, 2001), including a summary paper by Leroueil et al. (2003). Specific to the soundings and lab data on this program, please refer to Dourlet (2020).

2.1 Piezocone soundings

Lately, a series of 7 CPTU soundings were advanced at the site for a research project on clay excavations conducted by Dourlet (2020). Figure 1 presents a representative sounding showing the respective piezocone reading profiles to 15 m depth, including: (a) cone tip resistance, q_t; (b) sleeve friction, f_s; and (c) penetration porewater pressure, u_2; which will be used in the subsequent methods of analysis.

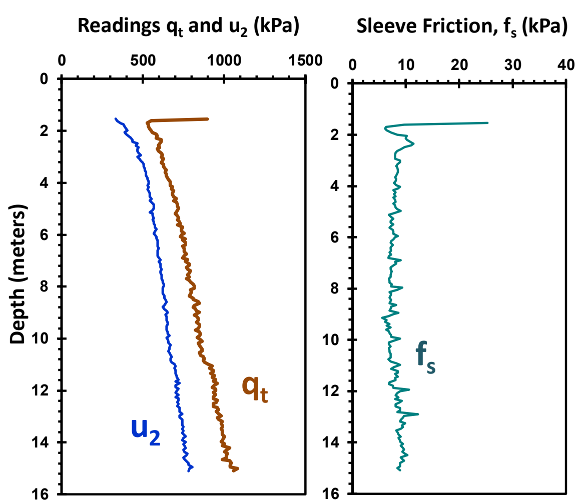

Figure 1. Representative CPTU C01 at Louiseville, Québec
Note: piezocone data from Dourlet (2020)

2.2 Soil behavior type

Since soil samples are not routinely obtained during CPTU, indirect methods have been developed to ascertain the soil types with depths from the individual readings. A common method is the use of empirical soil behavior type (SBT) charts and an extensive review of these is given by Niazi (2021).

In addition to use of the direct CPTU readings of q_t, f_s, and u_2 in the charts are net readings, including: 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: $q_E = q_t - u_2$. Furthermore, normalized piezocone readings 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 found as Q_t and Q_{t1} in the open geotechnical literature. Details on these various parameters are found in Mayne et al. (2023).

Certainly one of the most common SBT systems is that presented by Lunne et al. (1997) in terms of two paired charts with nine soil zones showing: (a) Q versus F and (b) Q versus B_q, An updated version of the Q-F chart uses a modified cone tip resistance (Q_{tn}), as discussed by Robertson & Cabal (2022).

Results from the CPTU sounding at Louiseville are plotted on the Q_{tn}-F chart in Figure 2, indicating a silty type soil (zone 4), while Figure 3 presents these data on the Q-B_q chart which classifies the on-site soils as clay (zone 3). Based on laboratory index, fall cone, and field vane, however, the data should fall into the sensitive clay region delineated as zone 1 in both graphs. Other independent studies have also found issues with the Q-F-B_q system in properly identifying sensitive and quick clays (e.g. Shahri et al. 2015; Sandven et al. 2016; DeGroot et al. 2019).

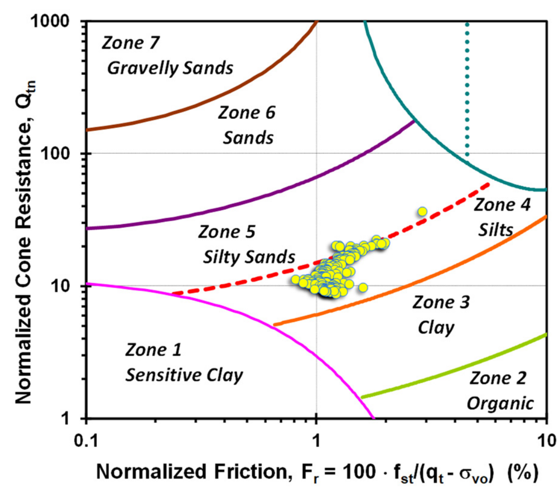

Figure 2. Soil behavior type from Q_{tn}-F_r chart for Louiseville

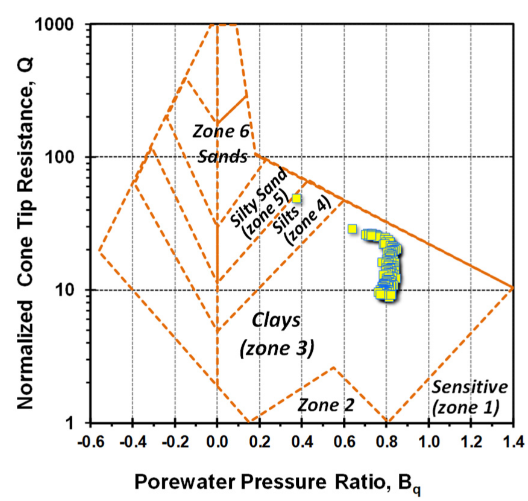

Figure 3. Soil behavior type from Q-B_q chart for Louiseville

2.3 Alternate CPTU screening approach

An alternative means to utilize CPTU data in separating out normal or regular type clays from sensitive clays and organic soils relies on a comparison of three simple expressions for the yield stress: (a) $\sigma_p' \approx 0.33q_n$; (b) $\sigma_p' \approx 0.54\Delta u$; and (c) $\sigma_p' \approx 0.60q_E$. Using the derived profiles of yield stress, it has been shown that the following hierarchy can be used to distinguish the clay type (Agaiby & Mayne 2021; Agaiby et al. 2021):

Regular clay: $0.33q_n \approx 0.54\Delta u \approx 0.60q_E$ (1)

Sensitive clay: $0.60q_E < 0.33q_n < 0.54\Delta u$ (2)

Organic clay: $0.54\Delta u < 0.33q_n < 0.60q_E$ (3)

For the CPTU data at Louiseville, the three estimated profiles of yield stress are presented in Figure 4 showing a hierarchy that clearly indicates a sensitive clay deposit. Also shown are the σ_p' values obtained from lab consolidation testing programs with non-agreement evident amongst the profiles.

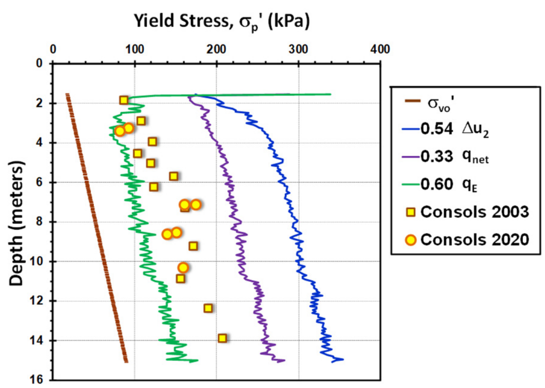

Figure 4. Screening by CPTU to illustrate Louiseville soil is sensitive clay formation.

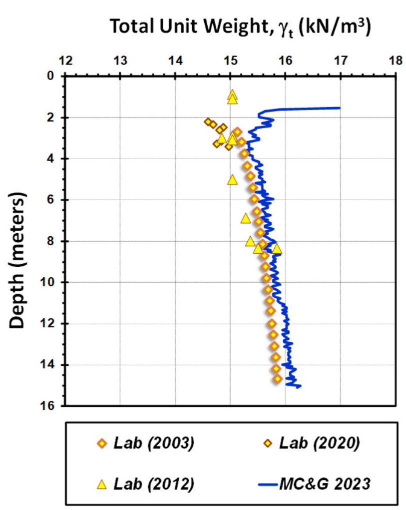

Figure 5. Soil unit weight estimate by CPTU for Louiseville.

3. Geoparameter evaluation by CPTU

As the clay at the Louiseville site is now recognized as sensitive, the assessment of selected geotechnical engineering parameters can proceed in a proper manner. The geoparameters include: unit weight (γ_t), rigidity index (I_R), undrained shear strength (s_u), effective stress friction angle (ϕ'), and yield stress ratio ($YSR = \sigma_p'/\sigma_{vo}'$), where σ_p' = effective preconsolidation or yield stress. In addition, with the results of CPTU dissipation tests, it is possible to evaluate the flow parameters, including in-situ values of coefficient of consolidation (c_v) and hydraulic conductivity (k) at selected test depths.

3.1 Soil unit weight

The total soil unit weight (γ_t) can be evaluated from an empirical expression using the effective cone tip resistance, q_E (Mayne, Cargill, & Greig 2023):

$$\gamma_t/\gamma_w = 1.54 + 0.254 \cdot \log\left(q_E/\sigma_{atm}\right) \tag{4}$$

where γ_w = unit weight of water and σ_{atm} = atmospheric pressure (≈ 1 bar = 100 kPa). Figure 5 shows the results of estimated unit weight from the CPTU in good agreement with the measured lab values. Additional methods for estimating soil unit weight are also available and provide good estimates at the Louiseville site (e.g., Robertson & Cabal 2010; Mayne 2014; Lengkeek & Brinkgreve 2022).

3.2 Undrained rigidity index

The undrained rigidity index (I_R) is defined as the ratio of shear modulus to undrained shear strength, thus $I_R = G/s_u$. From a hybrid analytical CPTU model based on spherical cavity expansion (SCE) theory and crtical-state soil mechanics (CSSM), the value of I_R in sensitive clay deposits is determined from:

$$I_R = exp\left[\frac{1.5+2.925M_{c1}\cdot a_q}{M_{c2}-M_{c1}\cdot a_q}\right] \tag{5}$$

where M_{c1} = frictional parameter defined at peak strength (i.e., q_{max}); M_{c2} = frictional parameter at maximum obliquity (i.e. maximum σ_1'/σ_3'), and a_q is found as the slope of the difference $(u_2 - \sigma_{vo})$ versus q_n. Alternatively, the parameter a_q is found as the slope of (U-1) versus Q. Values of the parameter M_c are derived from effective stress paths in triaxial compression and relate to the effective friction angle: $M_c = 6\cdot\sin\phi'/(3-\sin\phi')$.

For the Louiseville clay, results from several triaxial test programs have been reported from both isotropically consolidated (CIUC) and anisotropically consolidated (CAUC) series (Hamouche et al. 1995; Tanaka et al. 1998, 2001; Leroueil et al. 2003), as well as CIDC drained triaxial series (Oka et al. 1989). Figure 6 shows

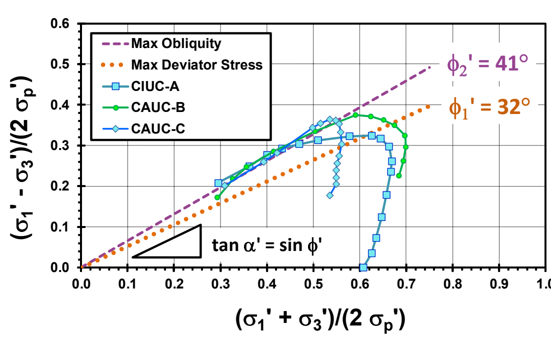

Figure 6. Selected triaxial stress paths for Louiseville clay.

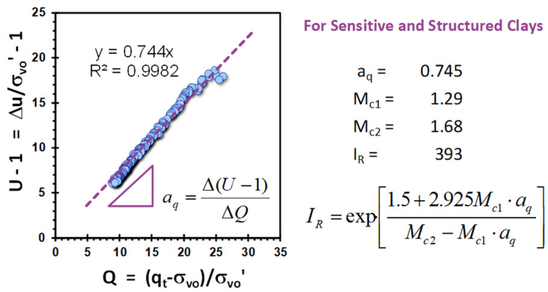

Figure 7. Evaluation of the slope parameter a_q and undrained rigidity index (I_R) of Louiseville sensitive clay.

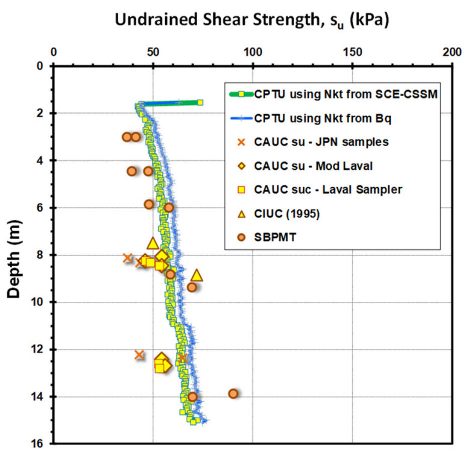

Figure 8. Evaluation of the undrained shear strength from CPTU in Louiseville clay.

a selection of representative triaxial test results and the interpreted values $\phi_1' = 32°$ at peak and $\phi_2' = 41°$ at maximum obliquity that correspond respectively to $M_{c1} = 1.29$ and $M_{c2} = 1.68$.

The plot of (U-1) vs. Q is presented in Figure 7 and indicates the slope parameter $a_q = 0.744$. Taking the two values of M_c provides the assessed value of rigidity index for Louiseville clay with $I_R = 393$.

3.3 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:

$$s_{uc} = \frac{q_n}{N_{kt}} \tag{6}$$

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

$$N_{kt} = 1.33\ln(I_R) + 3.90 \tag{7}$$

where a value $N_{kt} = 11.8$ is obtained from $I_R = 393$.

Figure 8 presents the derived profile of undrained strength at Louiseville from the CPTU in comparison with four series of triaxial compression tests. Also shown are results from self-boring pressuremeter testing (SBPMT) at the site (Hamouche et al. 1995a). Relatively good agreement is observed for this dataset.

Of additional interest, an empirical methodology for evaluating the value of N_{kt} directly from the pore pressure ratio (B_q) was developed from a statistical analysis of a large database of 62 clays (Mayne & Peuchen 2018, 2022) which showed:

$$N_{kt} \approx 10.5 - 4.6\ln(0.1 + B_q) \tag{8}$$

This too showed comparable results between the individual CAUC and CIUC triaxial tests, the SBPMT, and CPTU profile from the SCE-CSSM solution, as evident in Figure 8.

3.4 Yield stress ratio

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

$$YSR = 2\left[\frac{Q/M_{c1}}{0.667\cdot\ln(I_R)+1.95}\right]^{\left(\frac{1}{\Lambda}\right)} \tag{9}$$

$$YSR = 2\left[\frac{U-1}{0.667\cdot M_{c2}\cdot\ln(I_R)-1}\right]^{\left(\frac{1}{\Lambda}\right)} \tag{10}$$

$$YSR = 2\left[\frac{Q-\left({}^{M_{c1}}/_{M_{c2}}\right)(U-1)}{1.95\cdot M_{c1}+\left({}^{M_{c1}}/_{M_{c2}}\right)}\right]^{\left(\frac{1}{\Lambda}\right)} \tag{11}$$

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. The three derived profiles of YSR are shown in Figure 9 along with the benchmark results from one-dimensional consolidation testing. Rather good agreement is noted amongst the three expressions and with the laboratory results.

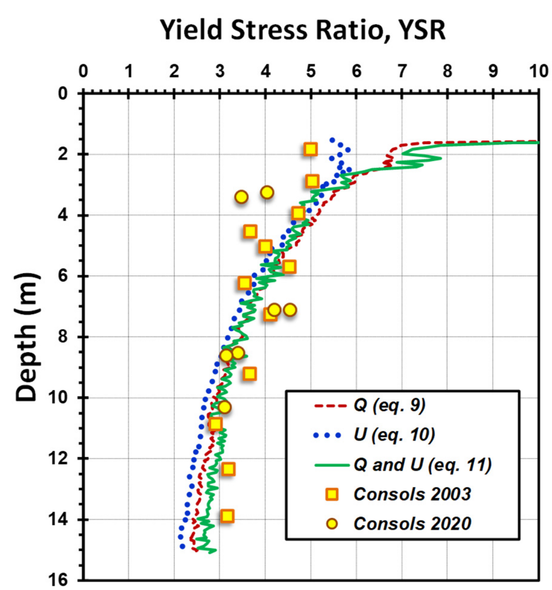

Figure 9. Profiles of yield stress ratio from in-situ CPTU and laboratory consolidation tests at Louiseville site.

3.5 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:

$$Q = \frac{\left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)[\exp(\pi\cdot\tan\phi')]-1}{1+(6\cdot\tan\phi')(1+\tan\phi')\cdot B_q} \quad (12)$$

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

$$\phi' \approx 29.5°(B_q)^{0.121}\cdot[0.256+0.336B_q+\log Q] \quad (13)$$

which applies for the following ranges: $18° \le \phi' \le 45°$ and $0.05 < B_q < 1.0$.

A modified NTH solution was 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:

$$Q' = Q/YSR^{\Lambda} \quad (14)$$

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 corresponds to the value of ϕ_2' at maximum obliquity (M.O.), defined when $(\sigma_1'/\sigma_3')_{max}$.

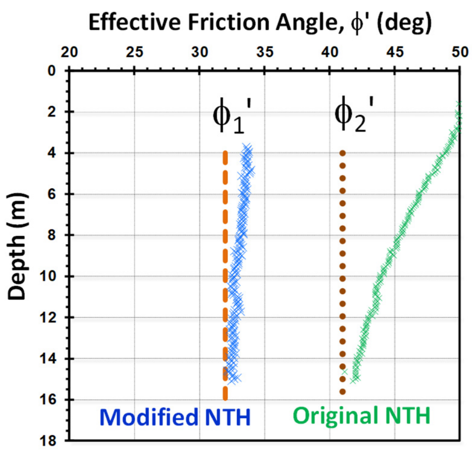

Figure 10. Profiles of effective friction angle for Louiseville clay using original and modified NTH solutions.

For the CPTU at Louiseville, Figure 10 presents the applications of the both the original and modified NTH solutions in evaluating the profiles of effective friction angles with depth, defined at $(\sigma_1'/\sigma_3')_{max}$ and q_{max} respectively. Here, the original NTH solution is seen to match the triaxial value at maximum obliquity of $\phi_2' = 41°$ at a depth of around 15 m, while the modified NTH solution agrees with the triaxial value of $\phi_1' = 32°$ over all depths. At this time, it remains unclear why the friction angle profile for ϕ_2' decreases with depth, thus offering impetus for continued research in this arena.

The ratio of the two friction angles (ϕ_1' / ϕ_2') has been noted to track with the CPTU parameter a_q, as shown in Figure 11 (Mayne, Cargill, & Greig 2023). For natural insensitive clays, including the common kaolin deposits used extensively in laboratory chamber and centrifuge testing, the ratio (ϕ_1' / ϕ_2') = 1, due mainly to the stress-strain curve and the pore pressure-strain curve reaching maximum values at the same time during triaxial shear. Note that these clays are found when $a_q < 0.5$.

For Louiseville, the triaxial values of (ϕ_1' / ϕ_2') = 0.78 and $a_q = 0.74$ are shown to follow the general trend and associate with the sensitive clay grouping. Thus, another method to properly identify sensitive clays from CPTU readings is when $a_q > 0.5$.

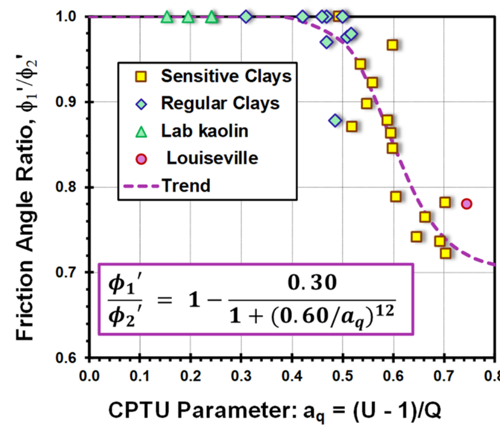

Figure 11. Ratio of friction angles at peak and maximum obliquity with CPTU slope parameter a_q.

3.6 Coefficient of consolidation

The field test program by Dourlet (2020) did not report any piezocone dissipation testing. However, some limited results of CPTU dissipations at Louiseville are reported by Leroueil et al. (2003).

The SCE-CSSM solution also addresses the interpretation of the coefficient of consolidation (c_{vh}) from porewater pressure dissipation tests (Burns & Mayne 2002). In homogeneous marine clays, there is little difference between the horizontal and vertical permeabilities (Jamiolkowski et al. 1985; Leroueil & Hight 2003). Therefore, permeability anisotropy is not of a major concern and the symbol cvh implies that the magnitude of the parameter is the same in both vertical and horizontal directions of flow.

A simplified solution is presented by Mayne, Cargill, and Greig (2023):

$$c_{vh} = \frac{T_{50}\cdot(a_c)^2(I_R)^{0.75}}{t_{50}} \quad (15)$$

where T_{50} = 0.028 is the theoretical time factor corresponding to 50% degree of consolidation, a_c = radius of the penetrometer, I_R = rigidity index, and t_{50} is the measured time to reach 50% consolidation.

Figure 12 presents the u_2 dissipation at a depth z = 9.67 m where the excess pore pressure (Δu) is normalized to the initial value at the start of the test. The corresponding time to 50% is measured at t_{50} = 12 minutes, and with I_R = 393, determines a value c_{vh} = 1.09 E-06 m^2/s.

This value compares favorably with laboratory values from oedometer tests in the OC region (Leroueil et al. 2003), as well as self-boring pressure-meter holding tests (Hamouche et al. 1995a), as seen in Figure 13.

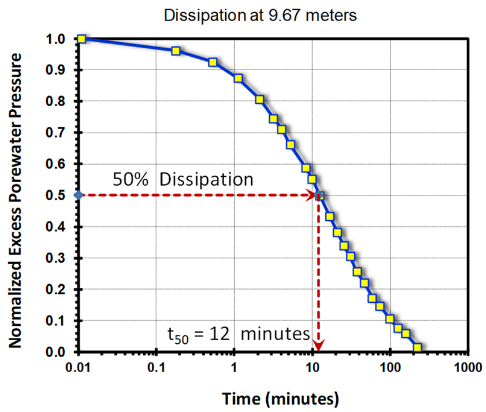

Figure 12. Results of dissipation test at 9.67 m in Louiseville reported by Leroueil et al. (2003).

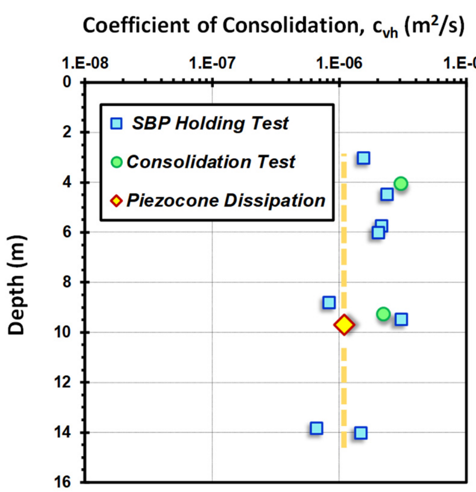

Figure 13. Coefficient of consolidation from piezocone dissipation, self-boring pressuremeter, and laboratory tests

3.7 Remolded strength and clay sensitivity

The clay sensitivity can be measured by field vane tests, laboratory fall cone, or other methods, such as miniature lab vane, or in-situ T-bar or ball penetrometer (DeGroot et al. 2012). All devices give different values of the sensitivity which is defined as the ratio of the peak undrained strength to the remolded strength at the same water content: $S_t = s_u/s_{ur}$.

It has long been postulated that the remolded undrained shear strength can be taken equal to the measured sleeve friction during CPTU, or $s_{ur} = f_s$ (e.g., Lunne et al. 1997). Consequently, a common means of using CPTU to assess clay sensitivity is given by:

$$S_t \approx 7/R_f \quad (16)$$

where $R_f(\%) = 100 \cdot f_s/q_t$ is the friction ratio (Robertson & Cabal 2022).

However, DiBuò et al. (2024) show that, for clays of low-medium sensitivity (S_t < 10), the aforementioned appears valid for many clays, while in contrast, for sensitive to quick clays (S_t > 10), it significantly underpredicts the sensitivity. This may be due to the accuracy and poor resolution of the sleeve friction at low readings, as well as need for correcting f_s for porewater pressures (Jamiolkowski et al. 1985; Lunne et al. 1997). Thus, for high sensitivity clays, it is insufficient to capture the remolded strength, and consequently, the measured f_s is 2 to 10 times higher than the measured s_{ur} (Mayne et al. 2023).

For the Louiseville test site, Figure 14 shows the application of eq. (16) that supplies an average S_t = 7, thereby underestimating the clay sensitivity when compared to lab fall cone results.

To resolve these issues, work by Yafrate & DeJong (2006), DeGroot et al. (2012), and Paniagua et al. (2024) have suggested alternate paths to assessing the remolded strength and sensitivity using CPTU and in-situ probes.

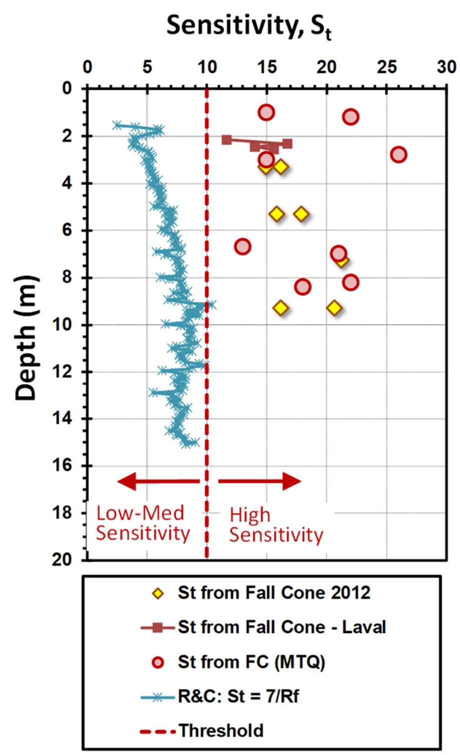

Figure 14. Measured fall cone sensitivity at Louiseville in comparison with conventional CPTU method (eq. 16).

4. Discussion

4.1 Alternate SBT charts

While the aforementioned section 2.2 presented commonly-used soil behavioral type charts that improperly categorized the soils at Louiseville as either silts or regular clay, further examination and post-processing of the CPTU data indicated that newer charts developed by Schneider et al. (2008, 2012) in fact are capable of assessing that the on-site soils are deemed sensitive. This is shown in the plot of Q versus U presented as Figure 15 where the Louiseville data fall within the zone designated for “sensitive soils”.

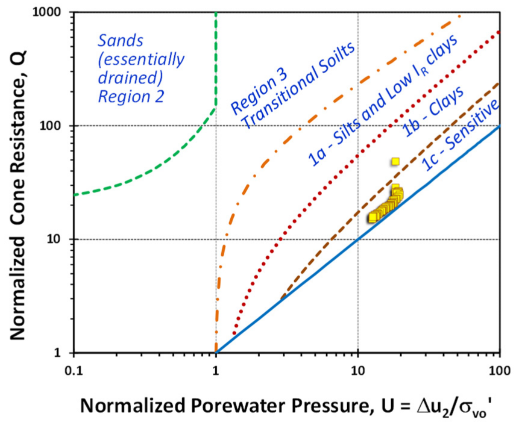

Figure 14. Alternate soil behavioral type chart with Q versus U by Schneider et al. (2008, 2012) with yellow symbol dots indicating CPTU data from Louiseville, Québec

4.2 YSR for modified NTH approach

According to Leroueil et al. (2003), the Louiseville clay has become overconsolidated due to 10 to 12 m of overburden erosion, thus a value of yield stress difference (YSD = $\sigma_p' - \sigma_{vo}'$) can be taken as YSD = 110 kPa. This agrees well with the profile of preconsolidation stress from the consolidation tests shown in Figure 4. In fact, the modified NTH profile of ϕ_1' with depth in Figure 10 was obtained from YSR = (YSD + σ_{vo}')/σ_{vo}'.

Figure 15 shows the results of the modified NTH approach using three separate evaluations of the YSR profile: (a) YSD = 110 kPa; (b) average of the SCE-CSSM solutions which requires iterative calculations; (c) empirical estimate based on correlative studies by Demers & Leroueil (2002) involving 22 case studies of CPTU in Québec where the preconsolidation stress is taken proportional to the net cone resistance:

$$\sigma_p' \approx 0.294\, q_n \tag{17}$$

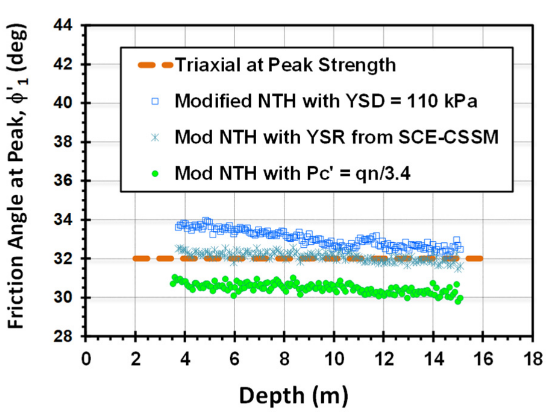

Figure 15. Influence of method for assessment of yield stress ratio on the effective friction angle at peak (ϕ_1') from modified NTH solution.

As seen by Figure 15, the YSD method provides a value of ϕ_1' in the range of 33.5° to 32.5°, whereas the SCE-CSSM solutions give a value ϕ_1' = 32° that matches exactly the triaxial value, and the empirical approach from eq (17) provides slightly lower values between 31° and 30.5°. Therefore the latter approach could be used initially to hone in on a working value of ϕ_1' until the SCE-CSSM profiles are more fully developed.

5. Conclusions

A case study involving piezocone tests in sensitive Leda clay at Louiseville test site in Québec has been used to illustrate the application of an analytical closed-form SCE-CSSM solution that delivers parametric values of the undrained shear strength (s_u), rigidity index (I_R), and yield stress (σ_p') profiles with depth, as well as the interpretation of the coefficient of consolidation (c_{vh}) from dissipation tests at selected depths.

A separate NTH analytical solution from limit plasticity theory is utilized to assess the effective friction angle of the clay from CPTU at two definitions: (a) peak strength, or ϕ_1' at q_{max}; and (b) maximum obliquity, or ϕ_2' at $(\sigma_1'/\sigma_3')_{max}$.

In addition, discussions are given on the assessment of soil unit weight (γ_t), soil behavior type (SBT), remoulded strength (s_{ur}), and clay sensitivity (S_t) by CPTU are provided. Charts of SBT in terms of Q-F-B_q have been known to miss the sensitive clay zone category, while other charts using Q-U are seen to be successful for the Louiseville site.

As an alternative or supplement to SBT charts, a sensitive clay can be properly identified by CPTU using two approaches: (a) a specified hierarchy of three σ_p'-based expressions given by eq. (1, 2, 3), or (b) recognized as “sensitive or quick” when the slope parameter $a_q > 0.5$, where a_q is defined as the slope of ($u_2 - \sigma_{vo}$) versus q_n = ($q_t - \sigma_{vo}$). Alternatively, a_q is found as the ratio of (U-1) to Q, which can be rearranged such that $a_q = B_q - 1/Q$.

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