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Case report

Geotechnical characteristics of sensitive Leda clay at Canada test site in Gloucester, Ontario

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Abstract: In 1954, the Canadian Geotechnical Research Site No. 1 was established as a testing and experimentation site for the soft and highly sensitive Leda clays that were deposited within the now-drained Champlain Sea. The Gloucester test site has a shallow groundwater table taken as hydrostatic at 0.8 m and is underlain by a sequence of leached marine sediments that make-up the Leda clays and extend about 22 m below grade. Below that are dense glacial tills and limestone bedrock. Previous research studies at the site have included full-scale construction, monitoring, and instrumentation of shallow foundations, embankments, drilled shafts, and driven pilings. Recently, a series of in-situ tests in 2012 and 2014 included type 1 and 2 piezocone soundings, porewater dissipations, and both conventional downhole tests and continuous-interval shear wave velocity measurements have been undertaken. Selected results obtained from laboratory, in-situ testing, and geophysical tests over the past six decades are presented.

Keywords: clays; cone penetration; field testing, in-situ tests; piezocone; porewater pressures; sensitivity; shear wave velocity; triaxial tests; vane shear

1. Introduction

A significant formation of soft sensitive clays known as Leda clays lies in eastern Canada including portions of the major cities of Ottawa, Montreal, and Quebec City. These complex marine deposits were deposited around 12,000 years ago in the former Champlain Sea. Figure 1 shows the approximate extent of the Leda clay deposits and includes terrain in and around Ontario, Quebec, New York and Vermont. Of special interest, these soft and very sensitive to quick clays are subject to destructuring and dramatic loss of strength, as evidenced by hundreds of recorded landslides in the region [1,2].

In an effort to improve our understanding on the behavior and engineering properties of Leda clays, the National Research Council of Canada (NRCC) established the Canadian Geotechnical Research Site No. 1 at South Gloucester, Ontario, about 24 km south of the capital city of Ottawa. Early geotechnical interests focused on problems related to excessive settlements and poor shallow foundation performance of existing dormitory buildings at the site circa 1954 [3]. Then, an impressive and extensive geotechnical study was conducted by Bozozuk [4] that involved careful borehole sampling and laboratory testing, limited field test methods, and the construction of a full-scale instrumented earthen embankment at the Gloucester site. Since then, many supplementary studies involving in-situ penetrometer and probe tests, laboratory testing, geophysics, and pile foundations have been completed.

Figure 1. Extent of sensitive Leda clays deposited by the Champlain Sea in Canada and USA.

2. Laboratory testing at Gloucester

2.1. Sampling

During the initial subsurface explorations, undisturbed piston tube type samples of the soft Leda clay were obtained using both NGI type and Osterberg samplers [4]. These were utilized for procurement of high-quality specimens used in the original set of consolidation and triaxial test programs. Later, a series of consolidation tests were performed on specimens taken with a Geonor piston sampler [7]. In a more recent set of triaxial and consolidation tests conducted in 2007, a Sherbrooke block sampling device was used [8].

2.2. Index tests

A profile of the various interpreted soil layers with selected index properties for the Gloucester test site is presented as Figure 2, including natural water content (w_n), liquid limit (w_L), and plastic limit (w_p), as summarized by Bozozuk and Leonards [5]. Overall, the mean index readings in the clay layers include: w_n = 64%, w_L = 52%, and plasticity index, I_P = 26%, thus classifying the soft soils as high plasticity clay (CH) per the Unified Soils Classification System (USCS), as detailed by ASTM D 2487. Sensitivities of the clay based on fall cone lab tests are reported in the range of 65 to 85 [6] and over 100+ based on earlier studies [4,5].

From the examination of recovered soil samples, the initial subsurface exploration defined 9 different soil layers at the site, as indicated in the soil profile presented as Figure 2 [4]. Later, based on the consolidation test results and piezocone dissipation responses, a simplified profile was evaluated as a 2-m crust underlain by three soft clay layers extending to 18 m.

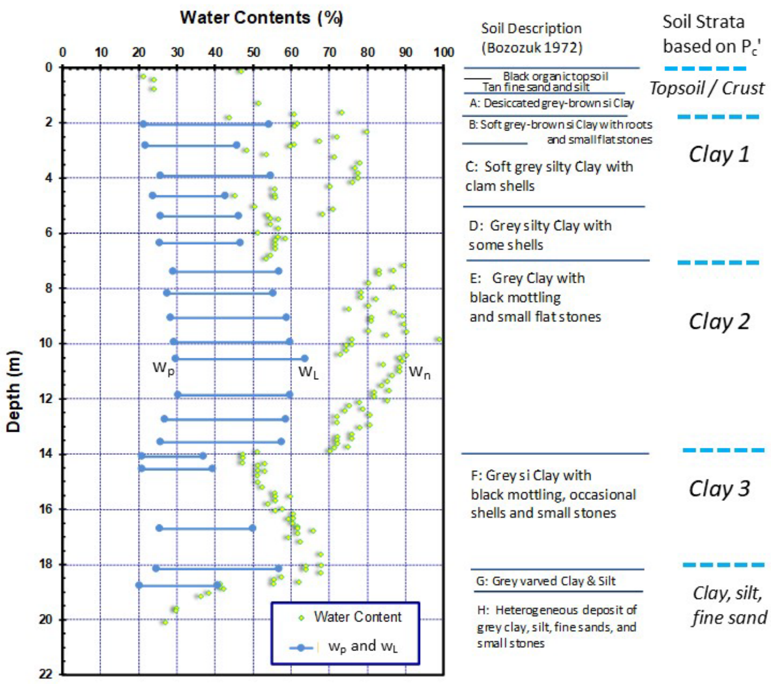

Figure 2. Subsurface profile at Gloucester NRC-1 site with plasticity information. Data from [4,5].

2.3. Yield stresses

The soft clay layers at Gloucester have a slight overconsolidation or quasi-preconsolidation probably due to a combination of ageing and secondary compression, some minor erosion, and bonding due to geochemical processes. The interpreted profile of preconsolidation stress (σ_p'), or effective yield stress, is shown in Figure 3 based on three sets of consolidation test series [4,7,8]. The interpretation of three main clay strata have been grouped by their minimum and maximum σ_p'

values obtained from the Casagrande procedure, as indicated [4]. Most probable values of σ_p' using this procedure are also discussed later (when Figure 19 is introduced). The consolidation test data show breaks in the σ_p' profiles at depths of 2, 7, 14, and 18 m. The upper 2 m is a desiccated crustal layer and the subsequent underlying soft clay layers appear to be different successions of deposition by the ancient Champlain Sea. Trends of σ_p' with depth that are indicated are those given by the interpretation provided by Bozozuk [4].

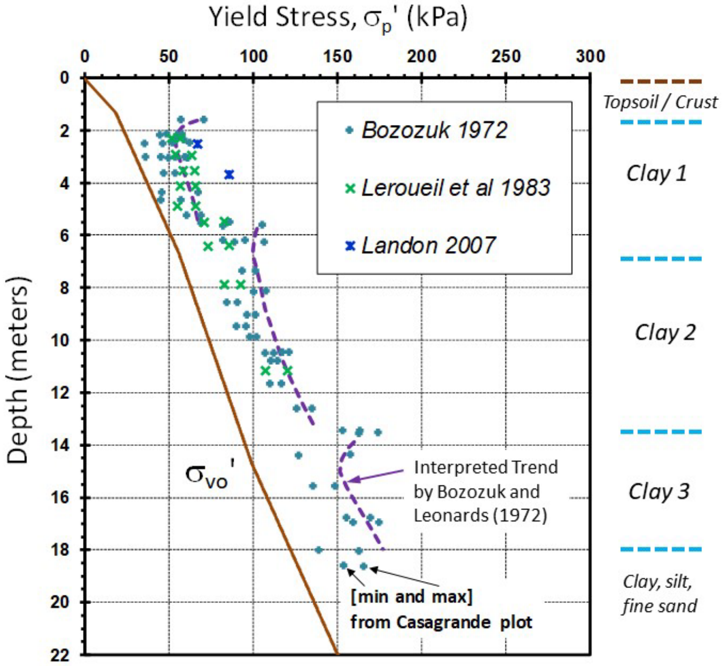

Figure 3. Profile of yield stresses in soft sensitive clay at Gloucester, Ontario.

2.4. Effective strength envelopes

Several series of laboratory strength tests were performed on undisturbed samples of Leda clay from the NRC-1 site by Bozozuk [4] including: (a) CIUC triaxials (consolidated isotropically undrained compression), (b) CAUC triaxials (consolidated anisotropically undrained compression) , and (c) plane strain compression (PSC) tests. Figure 4 shows a representative set of stress-strain and porewater pressure versus strain measurements on a clay specimen from the site [8]. A characteristic of many sensitive clays, the shear stress peaks early on at about 1% strain, while the excess pore pressure continues to increase, reaching a maximum value much later on at around 11% strain. Consequently, the effective stress strength parameters (assuming c' = 0) can be defined using two criteria. For the q-ε curve, an effective stress friction angle at peak is represented by $\phi' = 28°$ at q_{max}, while the Δu-ε curve indicates a higher value of $\phi' = 39°$ at maximum obliquity (M.O.), or $(\sigma_1'/\sigma_3')_{max}$. The value of ϕ' at q_{max} is characteristic of the destructured clay (or normally-consolidated) while ϕ' at M.O. corresponds to the intact clay that is lightly-overconsolidated.

These two envelopes are shown in Figure 5 for samples tested at confining stresses high enough to reach the normally-consolidated (NC) region, while Figure 6 shows the envelope for naturally lightly-overconsolidated (LOC) specimens tested at stresses close to the in-situ values. More recent triaxial tests by Landon [8] using the SHANSEP procedure show similar envelopes for specimens tested at OCRs of 1 and 2, as indicated by Figure 7.

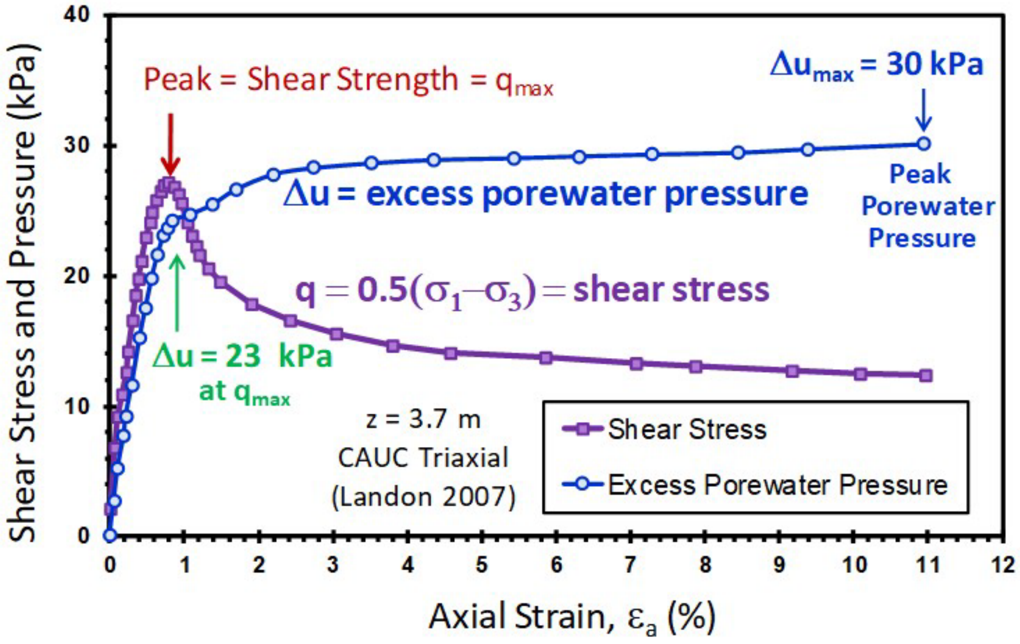

Figure 4. Representative triaxial data on soft clay from Gloucester NRC-1 site. Data taken from [8].

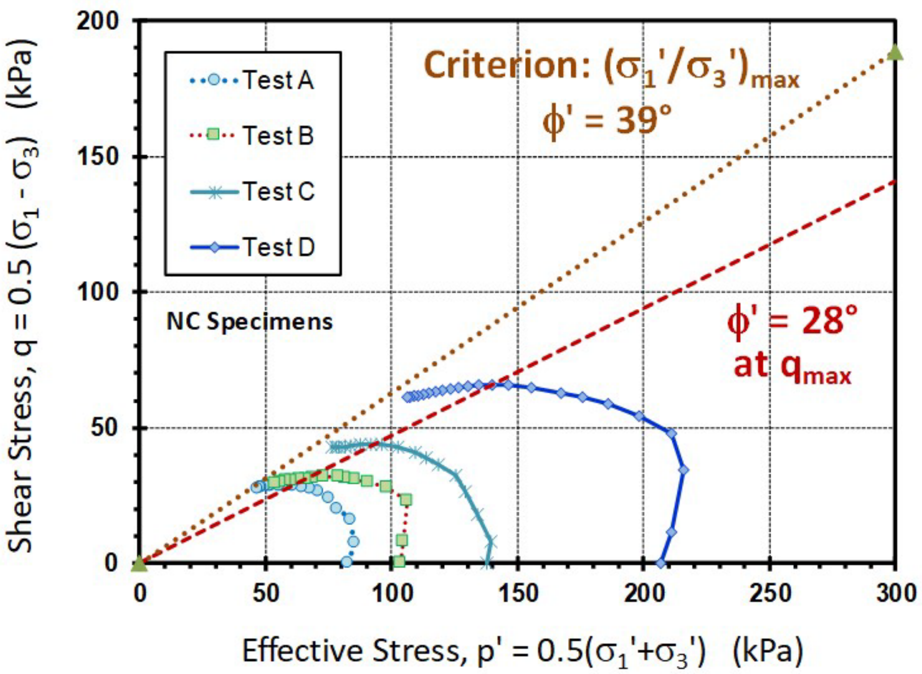

Figure 5. CIUC triaxial tests on NC specimens of normally-consolidated Leda clay from NRC-1 where $\sigma_{vc}' > \sigma_p'$. Data reported by [4].

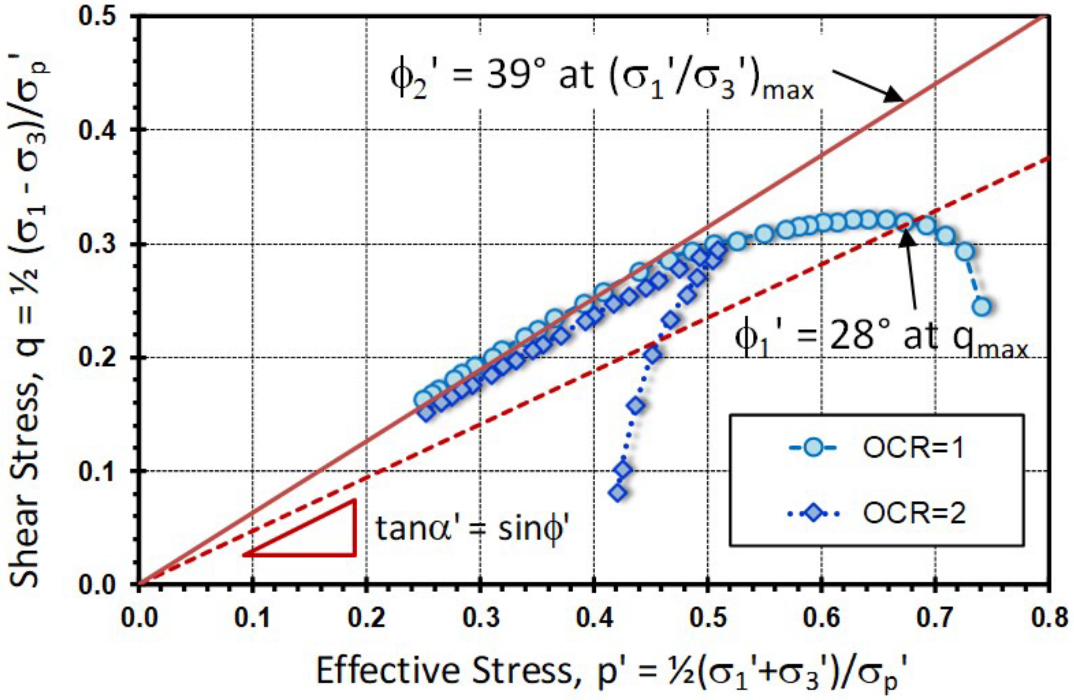

Figure 6. Effective stress paths from CAUC and CIUC tests from LOC range of soft clay at Gloucester where $\sigma_{vc}' < \sigma_p'$ and 1.2 < OCRs < 1.8. Data taken from [4].

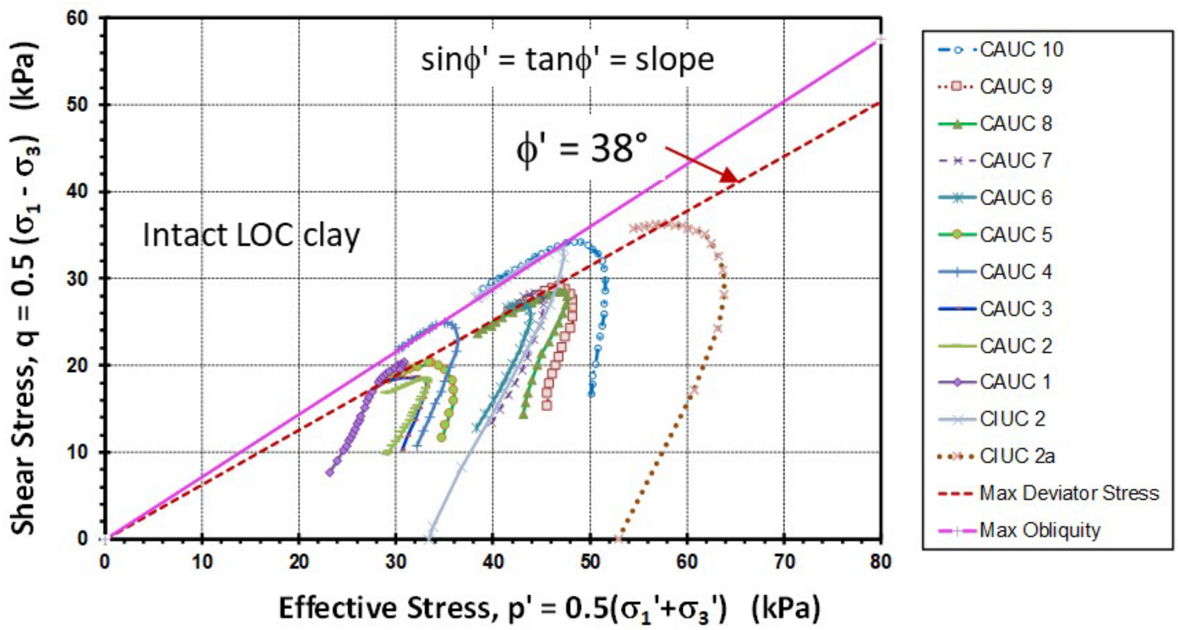

Figure 7. Results of CAUC triaxial tests on NC and OC clay specimens at Gloucester using SHANSEP procedure [8]. Note: sample depth z = 3.7 m.

3. In-situ testing

Two recent field testing programs were completed by the authors in 2012 and 2014. Figure 8 shows the setup for the rig at Gloucester NRC-1 that was used to conduct piezocone tests (CPTu), vane shear tests (VST), shear wave velocity (V_s) measurements, pressuremeter tests (PMT), and sampling at the site.

Figure 8. Setup and rig used for in-situ testing at Gloucester NRC-1 site, Ontario in 2014.

3.1. Field vanes

Three separate series of vane shear tests (VST) were reported by [5,7,9] using mechanical Geonor and Nilcon vane equipment. The newer 2014 series of VST were conducted using an electrovane built in-house by ConeTec (unpublished). A compilation of the vane strengths s_{uv} are seen in Figure 9 and indicate a general trend of strength increase with depth, as commonly observed in soft clays. For the limited data provided by the e-VST data, a general trend can be approximated from the ratio of vane strength to yield stress, giving $s_{uv}/\sigma_p' = 0.26$ at the site which appears in line with the general plasticity characteristics of the clay. Alternatively, separate trends could be evaluated for each of the three sublayers of soft clay.

The observed scatter in the older mechanical VST data hinders a more detailed evaluation of the three clay strata, mainly because the torque and rotation are measured uphold at the ground surface, where in fact the vane shearing occurs at some depth. While insufficient data are provided here, the authors believe that in general the results of e-VST are much improved over the older mechanical VST equipment, since the torque and rotation are measured downhole with e-VST at the same elevation as shearing, and then the data are transmitted up to the surface for recording.

3.2. Piezocone soundings

Several series of piezocone penetration tests (CPTu) have been performed at the Gloucester test site, including an early series [10]. Figure 10 shows a comparison of six CPTu soundings advanced at the site, including soundings 1 and 2 advanced by the authors in 2012 using an Adara CPT system and sounding 3 commissioned by Geological Survey of Canada (GSC) in 2012 that were performed using a Hogentogler system [9]. These CPTu data were collected by separate teams and equipment [11] about 4 months apart, both systems having a_{net} = 0.80 and b_{net} = 0.

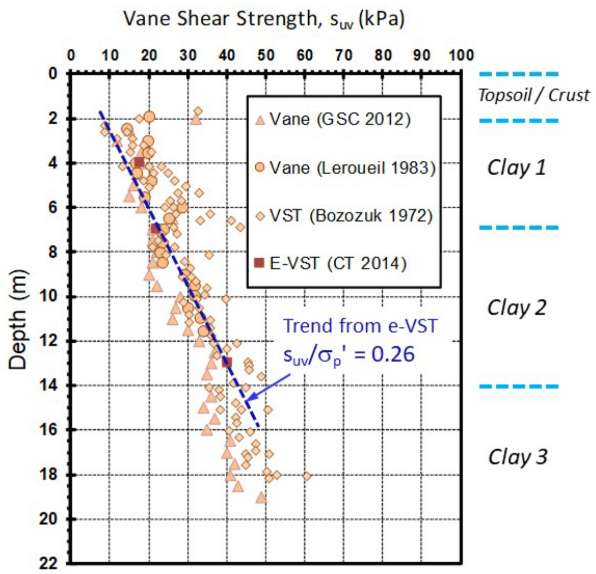

Figure 9. Compilation of 4 series of vane shear tests at Canada Test Site 1, Ontario.

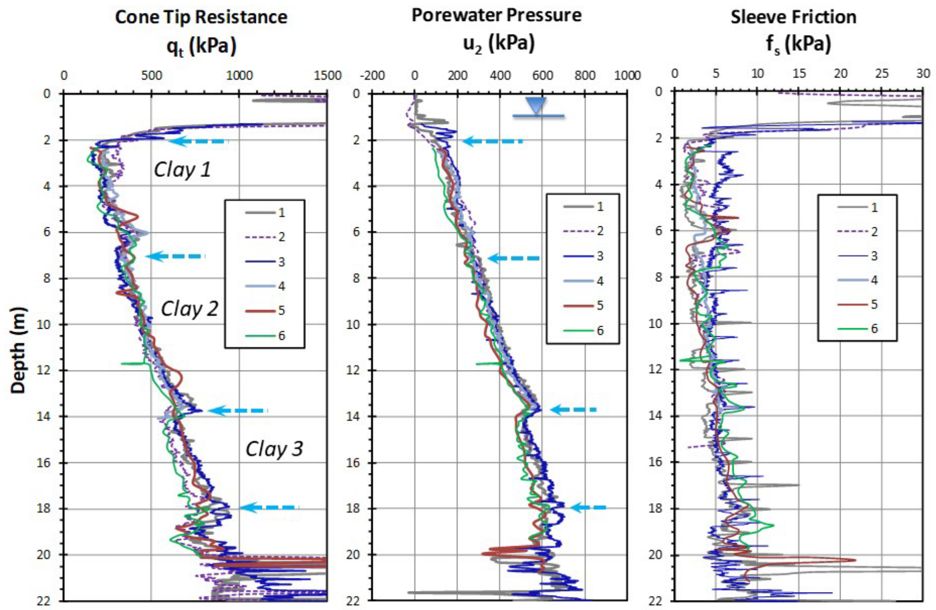

Figure 10. Comparison of 6 CPTu soundings at Gloucester Test Site by four organizations.

Sounding 4 was conducted by researchers at the University of Massachusetts—Amherst in 2007 using a Vertek penetrometer [8]. In order to match the aforementioned CPTu data, a correction of the sleeve reading using an assumed b_{net} value of −0.006 was made. Finally the last two soundings were obtained recently by an independent team [12] using a GeoTech CPT system that has a documented unequal end area of the sleeve, taken here as $b_{net} = 0.010$.

The soundings initially penetrated a crustal layer approximately 2 m thick. The maximum recorded cone resistance from the 6 soundings was $q_t = 6.5$ MPa (average about 4 MPa) and maximum sleeve friction was $f_s = 92$ kPa (average about 50 kPa) in the crust. The scales of q_t and f_s have been clipped to better show the comparisons in the soft Leda clay layers.

Beneath the 2-m crustal layer, the q_t resistances for all 6 soundings correspond well with the general breaks in σ_p' profiles noted earlier at 2, 7, 14, and 18 m depths. From depths of 2 to 18 m, the q_t resistances generally increase with depth and found to lie in the range: 200 kPa < q_t < 850 kPa. Similarly, the penetration porewater pressures increase with depth beneath the crust with the observed ranges: 150 < u_2 < 650 kPa. The u_2 readings all show the interface at 14 m, yet the other strata breaks in the clay layers are not so evident in all soundings.

While the q_t and u_2 readings are reasonably matched for these 6 CPTu soundings at Gloucester, the total sleeve resistance (f_{st}) profiles certainly show some noted scatter and variability. Notably, the f_{st} resistances are at the very low end of strain gauge resolution for CPT sleeve readings for commercial penetrometers, with values generally less than 5 kPa.

A type 1 piezocone sounding was performed by the authors using a mid-face filter element, as shown in Figure 11. Surprisingly, the u_1 and q_t profiles are nearly identical in the depth range from 4 to 12 m, suggesting that the measured cone tip resistances here are essentially all developed by induced porewater pressures. The drop-offs in the u_1 readings observed at 6.7, 9.5, 12.5, 15.5, and 18.5 m are likely due to operational procedure, specifically caused by stress relaxation as rods are added and re-gripping with the hydraulic clamps are necessary for each successive push.

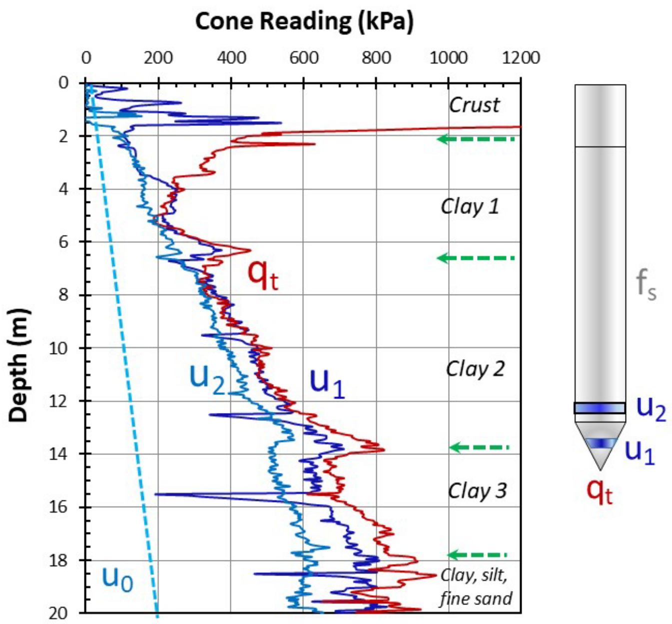

Figure 11. Results of a type 1 piezocone sounding at the NRC-1 site.

3.3. Dissipations testing

Porewater pressure dissipation tests were performed at the site using a type 2 piezocone [13]. The CPTu sounding for dissipation testing is shown in Figure 12 with decays recorded at 1-m intervals to a final depth of 23.65 m. The soil behavioral type (SBTn) based on normalized cone readings (Q and F) and CPT material index (I_c) is also shown in the right hand side. This generally indicates clay (zone 3) from 2 to 3 m and sensitive clay (zone 1) from 3 to 18 m, underlain by clays to sensitive clays.

The 23 individual records of porewater pressure decay with time are presented in Figure 13. Two types of dissipation responses are commonly found: (a) *monotonic*, where the measured u_2 is maximum during penetration and upon stopping, u_2 values decrease with time, and eventually reaches the equilibrium value (u_0) if sufficient time is allocated; (b) *dilatory*, whereby u_2 readings increase after stopping and reach a peak value some time afterwards, and subsequently decrease with time towards u_0. At Gloucester, the majority of dissipations yielded monotonic type responses, excepting the shallower tests (z < 7 m) that indicate a slight dilatory type behavior.

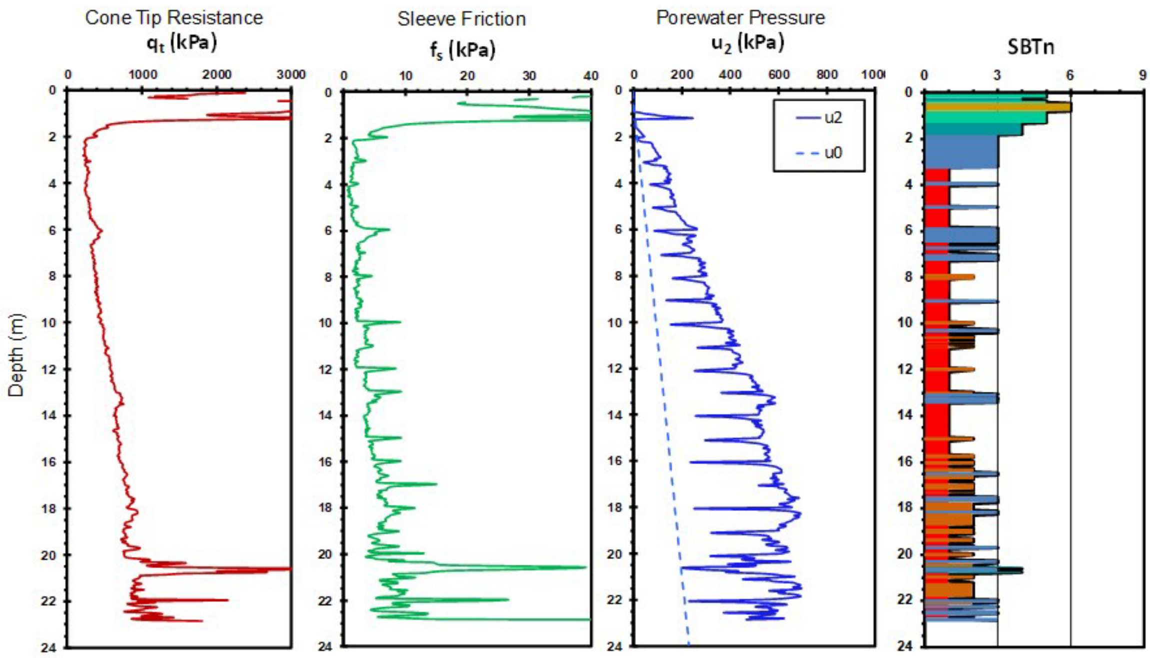

Figure 12. Piezocone sounding with dissipations taken at one-m intervals. Data taken from [13].

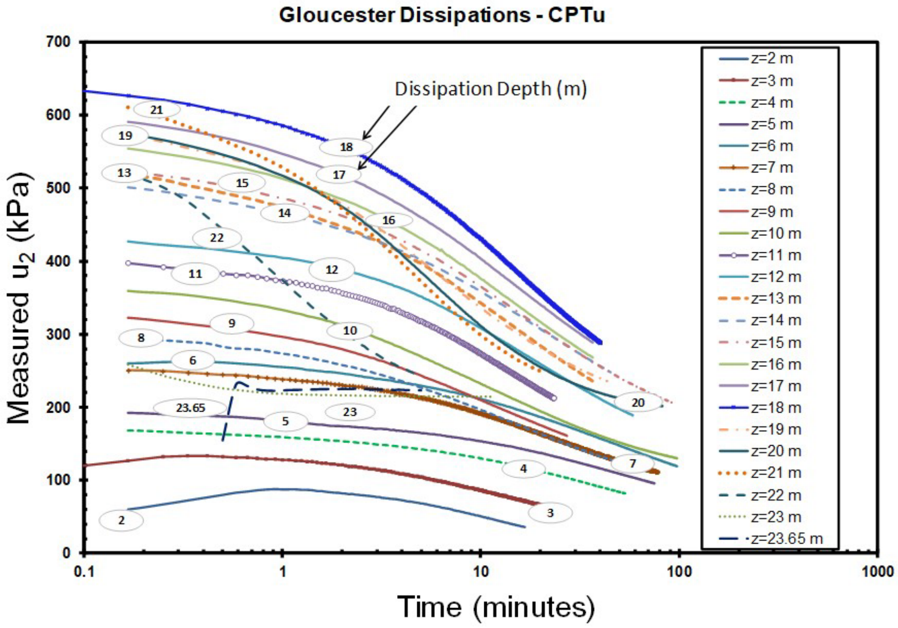

Figure 13. Recordings from porewater pressure dissipation tests at 23 depths in soft sensitive clays at Gloucester.

For each dissipation test, normalized excess porewater pressures can be defined as the value of Δu_2 at any time after stopping the cone advancement, divided by the initial value of Δu_{2i} during CPTu penetration, or $\Delta u/\Delta u_i$. Based on the normalized dissipation records, the results have been collectively grouped into three strata, as shown in Figure 14: (a) upper clay (clay layer 1) with slight dilatory response (z < 7 m), (b) intermediate clay layers (clay layers 2 and 3) with monotonic behavior (8–18 m), and (c) lower stratum (mixed clay-silt-sand) with monotonic response (z > 18 m), however a faster decay rate is observed here than for the intermediate layer.

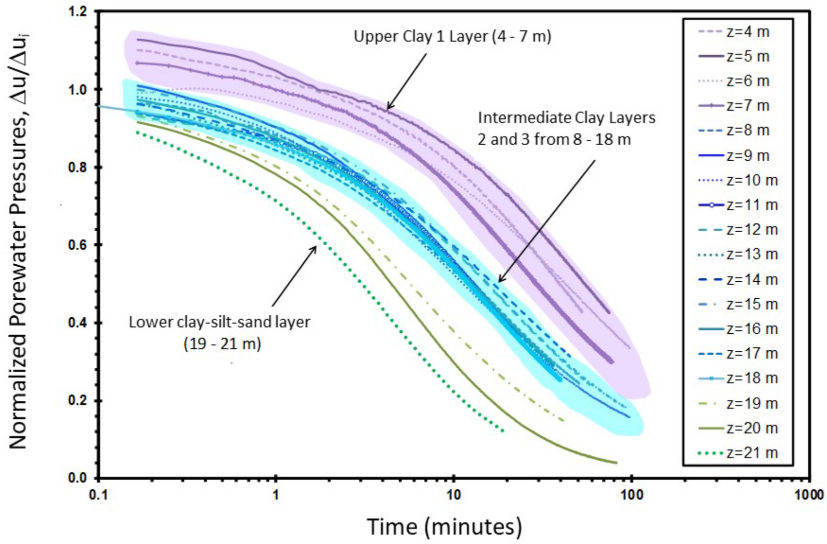

Figure 14. Normalized excess porewater pressure curves in soft clays at Gloucester NRC-1.

3.4. Shear wave velocity

In the past decade, several research groups have performed shear wave measurements at the site, predominantly utilizing a downhole test (DHT) mode via seismic piezocone tests [6,8,9]. These DHT results were conducted in accordance with ASTM D 7400 procedures and shown in Figure 15. Generally, the shear wave velocity (V_s) was obtained by interpretation of either first arrival or first crossover from paired left- and right-wavelets from strikes made using a horizontal beam situated at the ground surface to generate shear waves. The shear wave mode is a vertically-propagating horizontally-polarized wave, designated V_{sVH}.

The overall trend for the V_s profile at the Gloucester site is a slight decrease in the crustal region, until reaching a minimum value at shallow depths of about 4 m, where V_s values are as low as 60 m/s, followed by shear wave velocities steadily increasing with depth, eventually achieving V_s = 230 m/s at a depth of 22 m. The lowest V_s at the 4-m depth corresponds to a clustered set of high natural water contents, as shown by Figure 2.

A special set of continuous-interval shear wave velocities (CiV_s) were recorded at the site during seismic piezocone testing [14]. This was accomplished using an innovative autoseis hammer source that can generate an impact strike about once a second. The derived V_s profile from the CiV_s system is in good agreement with the conventional DHT methods, albeit slightly higher in the depth range of 6 to 8 m.

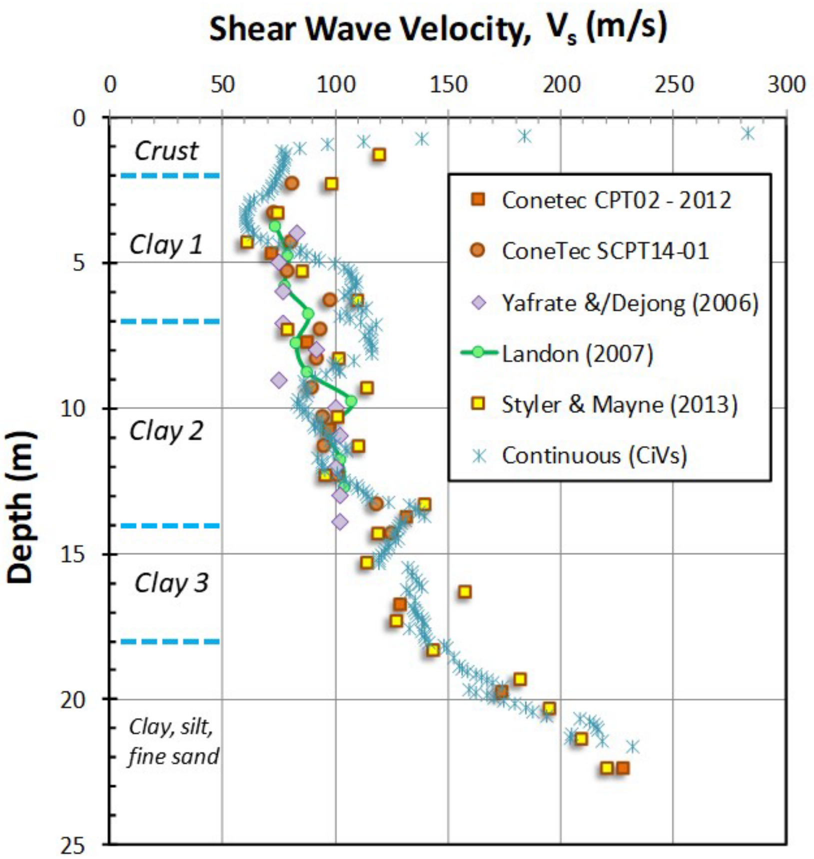

Figure 15. Compilation of shear wave velocities from downhole tests at Gloucester.

3.5. Additional field tests

A variety of other in-situ tests have been completed at the site, including: flat plate dilatometer tests (DMT) [15–17], Pencel and Menard type pressuremeter tests (PMT) [15], full-flow penetrometer testing using both the T-bar [6] and ball [12,18,19], borehole shear tests [15], and variable rate twitch testing [16]. Series of self-boring pressuremeter tests (SBPMT) have also been conducted at the NRC-1 site as well [15,20]. Results from a DMT sounding made at the site are shown in Figure 16 with the pressure readings (p_0 and p_1) and three DMT indices: material index (I_D), dilatometer modulus (E_D), and horizontal stress index (K_D).

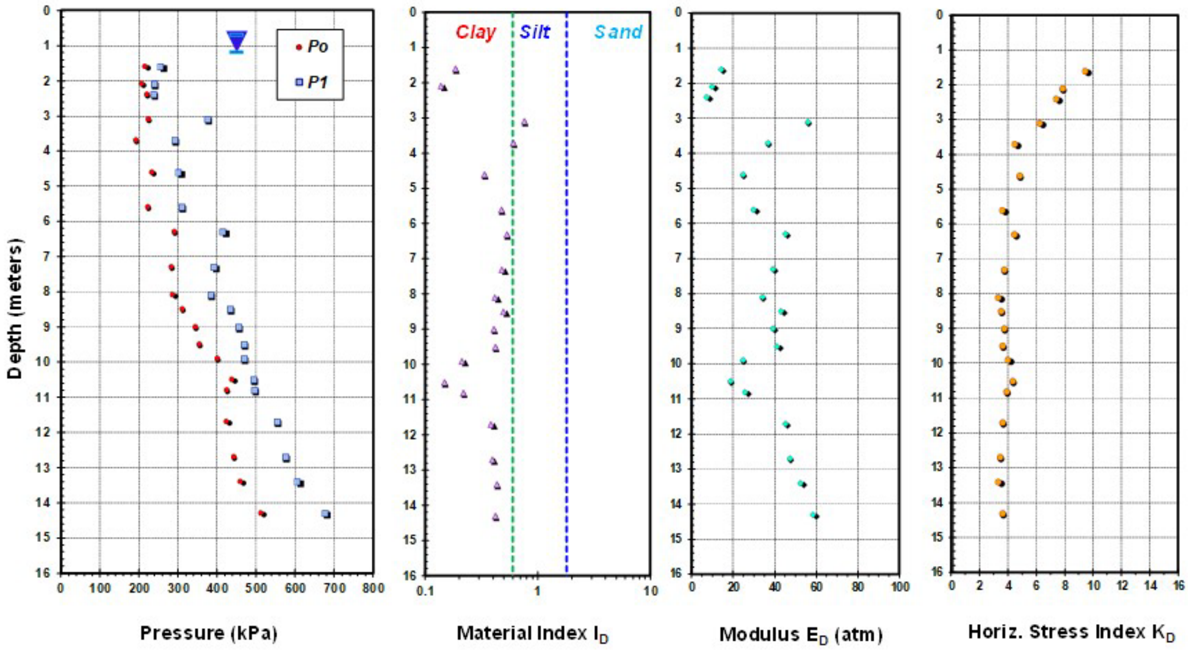

Figure 16. Results of DMT sounding at Gloucester (data courtesy: Alan Lutenegger).

4. GeoParameter interpretations

The evaluation of a selected number of soil engineering parameters for the Gloucester site will be made, including: (a) unit weight, (b) shear strength, (c) stress history, and (d) flow parameters (c_v and k).

4.1. Unit weight

Soil unit weights (γ_t) were measured from undisturbed samples taken at the site using piston tube sampling procedures [4]. An existing relationship for estimating γ_t from CPTu in soft clays relies on a parameter (m_q) that is defined as rate of change in cone resistance with depth [21]:

$$m_q = \Delta q_{t/}\Delta z \tag{1}$$

and defined through the origin. For Gloucester, a value of m_q = 42.8 kN/m^3 was determined, as shown by Figure 17. The relationship for unit weight is given by:

$$\gamma_t = \gamma_w + 0.125 \cdot m_q \quad (2)$$

where γ_w = 9.81 kN/m^3 = unit weight of water. An improved estimation is given in terms of cone tip resistance in kPa and slope parameter m_q in kN/m^3:

$$\gamma_t = 0.636(q_t)^{0.072} \cdot (10 + 0.125 \cdot m_q) \quad (3)$$

Both estimations are shown in general agreement with the measured unit weights that have an overall mean value γ_t = 15.8 kN/m^3 at the site.

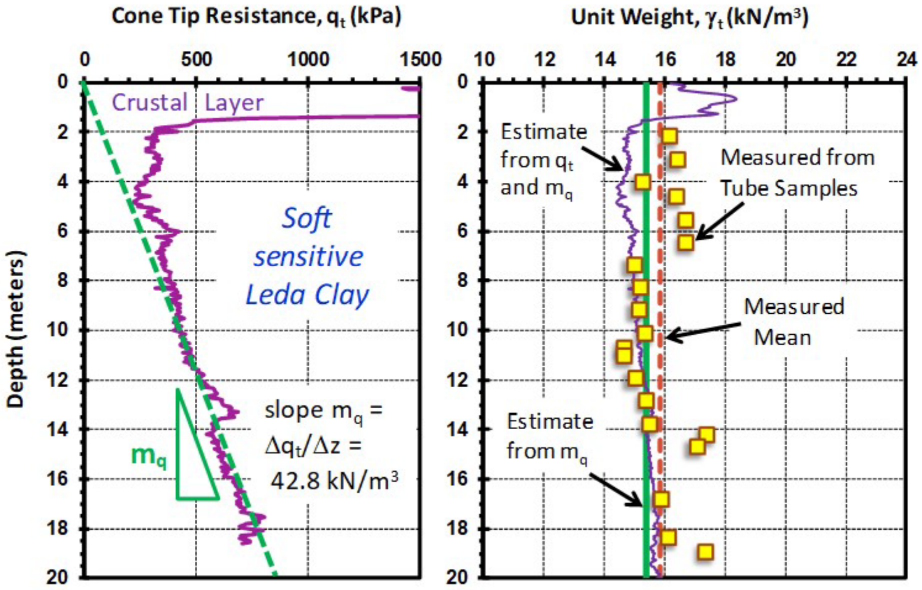

Figure 17. Evaluation of soil unit weight from CPTu at Gloucester NRC-1.

4.2. Undrained shear strength

For geotechnics involving soft clays, an important parameter to determine is the undrained shear strength (s_u). While seemingly straightforward, there are in fact many modes of shear, including triaxial compression, simple shear, and triaxial extension, as well as modes associated with plane strain and true triaxial conditions. For discussion herein, a triaxial compression mode is adopted, often denoted as s_{uc}.

With piezocone data, the undrained shear strength (s_{uc}) can be evaluated on the basis of net cone resistance ($q_{net} = q_t - \sigma_{vo}$), excess porewater pressure ($\Delta u = u_2 - u_0$), and effective cone resistance ($q_E = q_t - u_2$), from the following expressions:

$$\sigma_p' = q_{net}/N_{kt} \quad (4)$$

$$\sigma_p' = \Delta u/N_{\Delta u} \quad (5)$$

$$\sigma_p' = q_E/N_{ke} \quad (6)$$

Based on a recent calibration of CAUC-CPTU data from 62 clay sites, a review by Agaiby [23] in the calibration of these bearing factors determined that they tracked reasonably well with the porewater pressure parameter, B_q:

$$N_{kt} = 10.5 - 4.6 \cdot \ln(B_q + 0.1) \quad (7)$$

$$N_{\Delta u} = 7.9 + 6.5 \cdot \ln(B_q + 0.3) \quad (8)$$

$$N_{ke} = 4.5 - 10.6 \cdot \ln(B_q + 0.2) \quad (9)$$

For Gloucester, values of B_q are in the range of 1.2 to 0.80 in the soft clay layers. The specific values of B_q at each depth can be used and these determined mean values (and corresponding standard deviations) of the bearing factors in the depth ranges of 2.5 to 18.5 m: $N_{kt} = 10.14 \pm 0.50$, $N_{\Delta u} = 9.51 \pm 0.59$, and $N_{ke} = 2.73 \pm 1.06$. Using these values gave rather comparable profiles of s_{uc} when compared with the triaxial tests reported at the site [4,8], as well as the VST data discussed previously, as indicated by Figure 18.

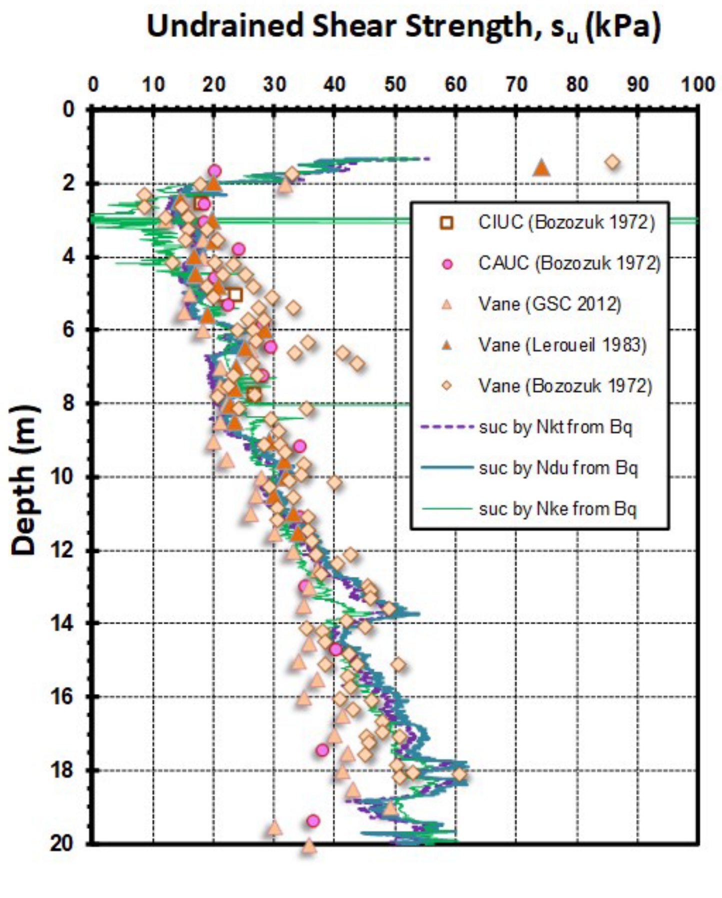

Figure 18. Interpretation of undrained shear strength profile using CPTu data at Gloucester.

4.3. Yield stress ratio

The profile of YSR = σ_p'/σ_{vo}' from two series of consolidation tests performed on undisturbed samples from the NRC-1 site is presented in Figure 19. Also shown are the estimated YSRs using the continuous shear wave velocity (CiVs) measurements (m/s) and an empirical relationship for yield stress (kPa) given by [24]:

$$\sigma_p' = 0.106 \cdot V_s^{1.47} \tag{10}$$

In addition, a well known relationship for evaluating σ_p' in clays from CPTu is superimposed where the results from CT-CPT-01 are used in the expression [25]:

$$\sigma_p' = q_{net}/3 \tag{11}$$

Overall, the profile of YSR decreases from about 2.5 at 2 m depth to around 1.3 at 18 m.

An improved interpretation of YSR from CPTu is offered by a hybrid analytical solution based on spherical cavity expansion - critical state soil mechanics (SCE-CSSM) that utilizes both ϕ' at q_{max} and ϕ' at M.O., corresponding to the destructured clay and intact clay, respectively [11,23].

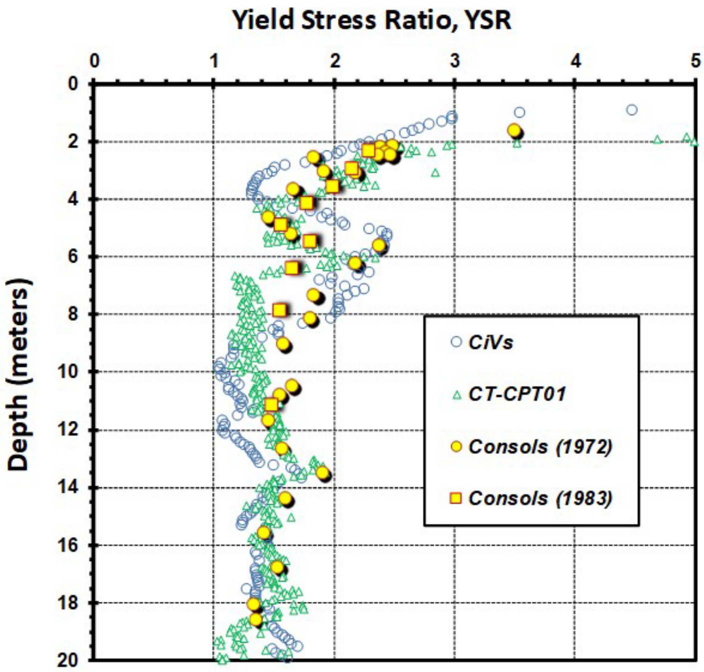

Figure 19. Profile of yield stress ratio (YSR = σ_p'/σ_{vo}') from SCPTu at Gloucester site.

4.4. Coefficient of consolidation

Results of the piezo-dissipation tests have been assessed to determine the profile of coefficient of consolidation (c_{vh}) at the site. Herein, a SCE-CSSM solution has been utilized [26]. For the intermediate layer clay at NRC-1, the excess porewater pressures (Δu) have been normalized to their initial value measured during penetration, as shown in Figure 20. For dissipation interpretations, a representative degree of consolidation of 50% is usually adopted, with the results from 8 to 18 m depths indicating a measured characteristic t_{50} between 10 and 20 minutes, and a mean value given by t_{50} = 13 minutes.

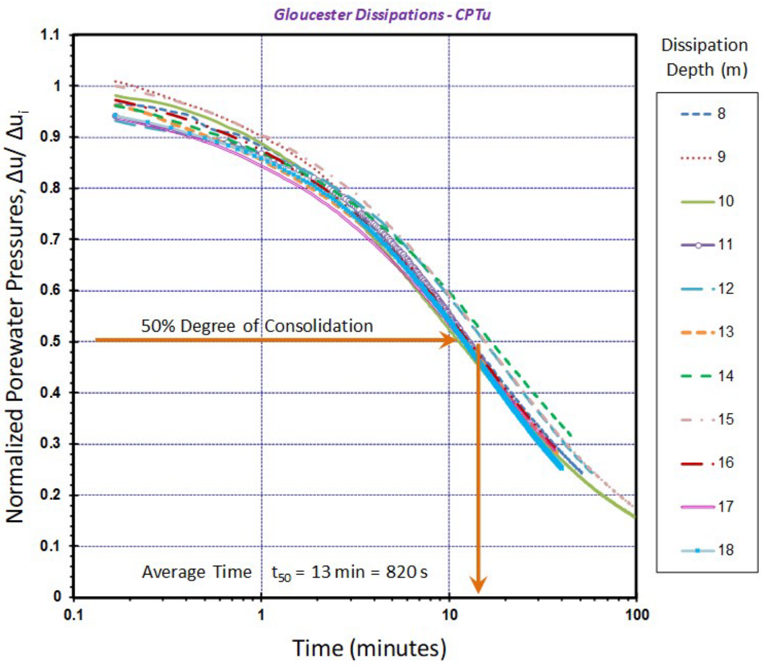

Figure 20. Dissipation curves in middle clay layer at Gloucester for determining t_{50} values.

For monotonic dissipations, the SCE-CSSM analytical solution gives [26]:

$$c_{vh} = \frac{T_{50}' \cdot (a_c)^2 \cdot (I_R)^{0.75}}{t_{50}} \tag{12}$$

where the time factor at 50% completion is T_{50}' = 0.030, a_c = radius of probe, and $I_R = (G/s_u)$ = rigidity index, and G = shear modulus [11]. The operational rigidity index is obtained from:

$$I_R = \exp\left[\frac{1.5 + 2.925 M_{c1} \cdot a_q}{M_{c2} - M_{c1} \cdot a_q}\right] \tag{13}$$

where $M_c = 6 \sin\phi'/(3 - \sin\phi')$, the value M_{c1} corresponds to ϕ_1' at q_{max} and M_{c2} to ϕ_2' at M.O., and the parameter a_q is the slope of $\Delta u_\sigma = u_2 - \sigma_{vo}$ versus $q_{net} = q_t - \sigma_{vo}$. For Gloucester, a value of a_q = 0.693 is determined from the piezocone data, as shown in Figure 21. Using ϕ_1' = 28° (M_{c1} = 1.11) and ϕ_2' = 39° (M_{c2} = 1.59) together with a_q gives an operational value of I_R = 95.

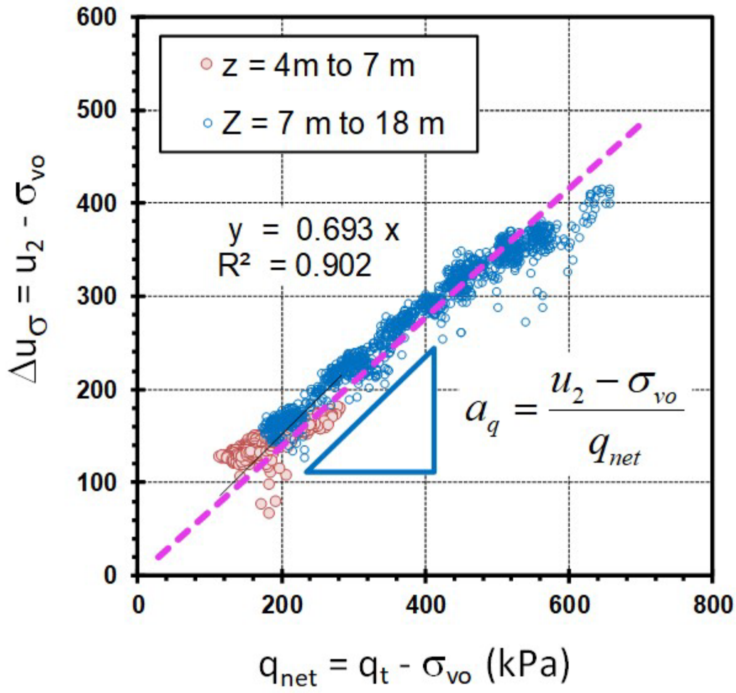

Figure 21. Plot of Δu_σ vs. q_{net} to obtain slope parameter (a_q) for rigidity index evaluation.

The derived profile of c_{vh} from this CPTu dissipation interpretation is presented in Figure 22. The values are seen to be in general agreement with independent assessments of c_{vh} at the site from laboratory permeameter tests [4,28] and field piezometer data [4,11].

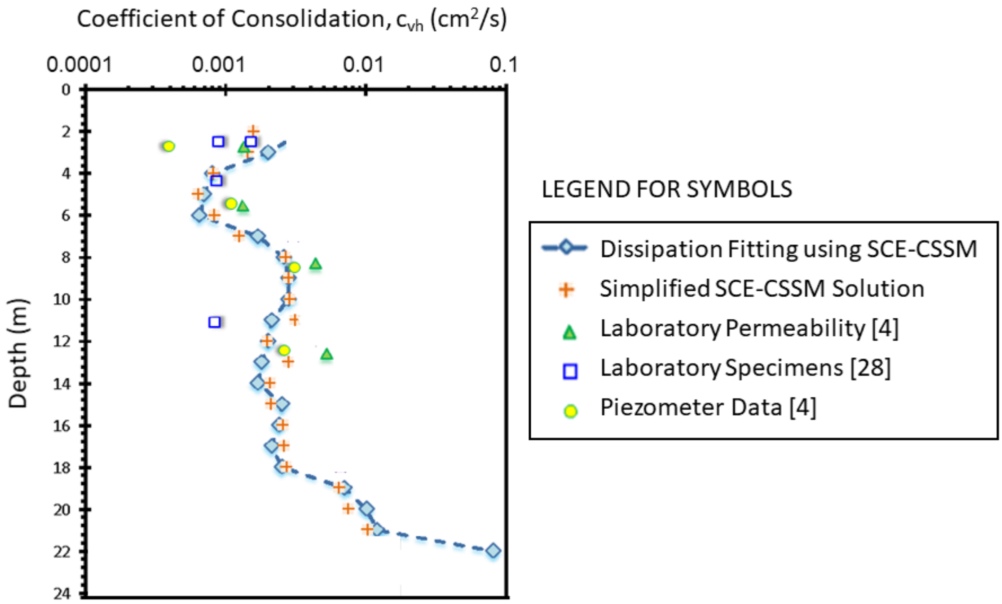

Figure 22. Interpreted profile of coefficient of consolidation with depth at Gloucester [11].

4.5. Hydraulic conductivity

The hydraulic conductivity (k), or coefficient of permeability, has also been measured at the Gloucester NRC-1 site. Figure 23 shows a compilation of k values obtained from independent tests at the site, including: laboratory permeameter tests on vertical [29] and horizontal specimens [4], field piezometer tests [4], and CPTu dissipation tests [11]. A simple empirical approach developed for monotonic dissipations is given approximately by [32]:

$$k\ (\mathrm{cm/s}) \approx (251 \cdot t_{50})^{-1.25} \tag{14}$$

where t_{50} is input in seconds. For NRC-1, the t_{50} values reported in [13] have been applied and provide a reasonable evaluation of hydraulic conductivity, as evident from Figure 23.

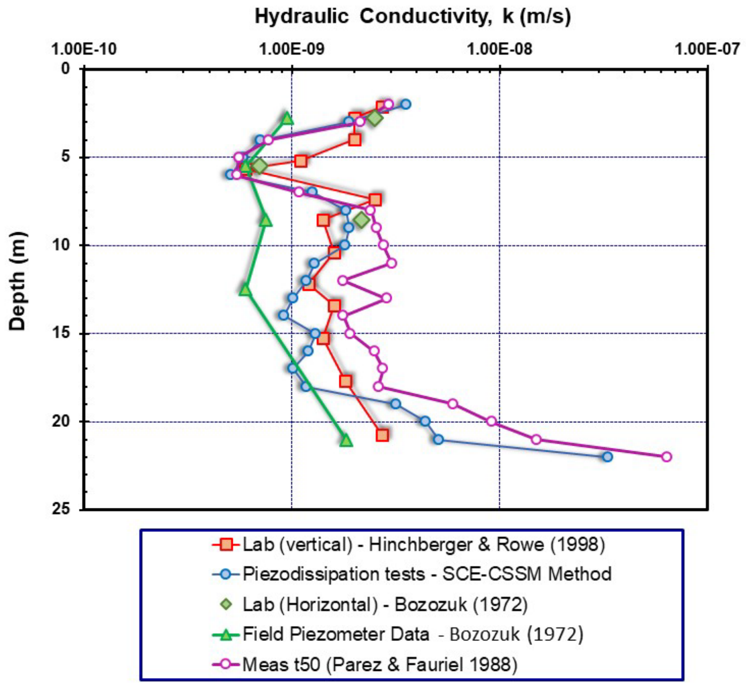

Figure 23. Profile of hydraulic conductivity at the Gloucester NRC-1 test site.

5. Full scale load testing

Over the past 6 decades, the Gloucester testing grounds have been used to provide information concerning the performance of large structures on soft sensitive clays of the Champlain Sea. Following WWII, problems with footing settlements on these structured clays were monitored and studied [3]. The initial and comprehensive geotechnical site characterization by Bozozuk [4] involved researching the performance of a full-scale instrumented and monitored test embankment that was constructed at the site [5,27–29].

Pile load testing programs have been implemented at the site, including full-scale compression and tension load testing on drilled shaft foundations [3]. A more recent series of compression and tension load tests on driven steel pipe piles, both open-ended and closed-ended have also been performed [31].

6. Conclusions

The NRC-1 geotechnical test site at Gloucester, Ontario has served an important geological site for laboratory, in-situ, geophysical, and full-scale experiments of the infamously sensitive Leda clays in eastern North America. In this paper are presented some selected examples from laboratory consolidation, triaxial, and index tests and in-situ programs that included VST, CPTu, DMT, and V_s measurements. Almost seven decades of documented studies and sophisticated research, a full understanding of soil behavior and performance of Champlain Sea clays is evolving and becoming ever more evident. The continuation of geotechnical research at this and other established testing sites will be of great benefit to soil researchers, geological and geotechnical engineers and local communities.

Conflict of interest

The authors declare no conflict of interest in this paper.

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