Geotechnical Characterization of a Glacio-Lacustrine Clay Deposit of the Abitibi Region – Case Study

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ABSTRACT
This paper presents the summary of laboratory tests results obtained during a geotechnical characterization program conducted at the site of a gold mine located in the Abitibi region (name and exact location to remain undisclosed), where a part of the Tailings Storage Facility (TSF) is built over a glacio-lacustrine clay deposit. The laboratory testing program was oriented to comprehend the stress-strain behaviour of the clay and to determine if the clay is subjected to strain weakening for different undrained shear modes and overconsolidation ratios (OCR). The laboratory testing program included Consolidated Isotropically and Anisotropically Undrained Shear tests, triaxial compression (CIUc and CAUc) and triaxial extension (CAUe), Direct Simple Shear tests (DSS), Cyclic Direct Simple Shear (CDSS) and Incremental Loading (IL) one-dimensional compression tests. Due to the importance of the sample's quality to conduct this series of tests, special care was taken to collect 8-inches high quality clay samples using the Laval sampler in addition to regular (2.875-inches) thin wall (Shelby) tubes, in an area where the clay has not been loaded by the TSF and is still in an overconsolidated state.

RÉSUMÉ
Cet article présente un sommaire des résultats obtenus lors d'une caractérisation géotechnique réalisée sur une argile varvée d'origine glacio-lacustre présente par endroits sous le parc à résidus miniers d'une mine d'or, localisée en Abitibi. Le nom et la localisation exacte doivent demeurer confidentiels. Le programme de laboratoire a été orienté de façon à étudier le comportement contrainte-déformation de cette argile et plus particulièrement à évaluer sa susceptibilité à l'anti-écrouissage selon différents modes de cisaillement non drainés et différents rapports de surconsolidation. Les essais en laboratoire comprennent des essais triaxiaux consolidés isotropiquement et anisotropiquement et cisaillés en condition non drainée en compression (CIUc et CAUc) et en extension (CAUe), des essais de cisaillement simple monotones (DSS) et cycliques (CDSS) et des essais de consolidation par chargements incrémentaux (IL). En raison de l'importance de la qualité des échantillons, l'échantillonneur Laval (8 pouces) a été utilisé en plus des tubes Shelby standards (2,875 pouces), et ce, majoritairement dans un secteur où le dépôt glacio-lacustre est demeuré surconsolidé.

1 INTRODUCTION

The mine is a gold mine located in the Abitibi green belt geological region in the Province of Quebec. Tailings are discharged in a tailings storage facility (TSF) that was partially built on a varved glacio-lacustrine deposit formed at the time of the former Barlow-Ojibway Lake. This type of geological deposit offers the advantage of being nearly impervious to contaminant migration but presents other challenges.

Some of those include its susceptibility to generate shear-induced pore pressure when subjected to a deviator stress (undrained shear behaviour) and possibility for strain-weakening, i.e. a decrease in shear strength as the clay undergoes shear strain. The latter is not systematically observed in such material, and this is the main topic of this paper.

2 GEOLOGICAL SETTING

The Mine is located within the footprint of the former glacial lake Barlow-Ojibway (Figure 1), where glacio-lacustrine varved clays are known to be existing between elevations 320 m and 274 m (MERN, 1973). At the studied site, the varved clays are present on a thickness of about 12 m, between elevations 320 and 308 m. Figure 2 shows a photo of the varves encountered at the site. The finer portion identified by the darker lenses formed in Winter (clay) are nearly 1 cm thick while the coarser lenses formed in Summer (silt) are less than 0.5 cm thick. There are a few publications worth reading for those interested in the topic including Leroueil et al. (1985), Quigley (1980) and Kenney and Chan (1973).

3 DESCRIPTION OF THE GEOTECHNICAL CHARACTERIZATION

Throughout the years, a series of geotechnical investigations including borehole sampling, Nilcon Vane testing, installation of monitoring instruments and Seismic Cone Penetration Tests with porewater pressure measurements (SCPTu), were conducted at the site. Several clay samples were collected at the site using 2.875-inches thin wall tubes. Historically, most of the clay samples were collected under the TSF where the preconsolidation pressure was exceeded or nearly exceeded due to the weight of the tailings. This led to sample qualities that did not allow confirming with confidence the stress-strain behaviour of the clay.

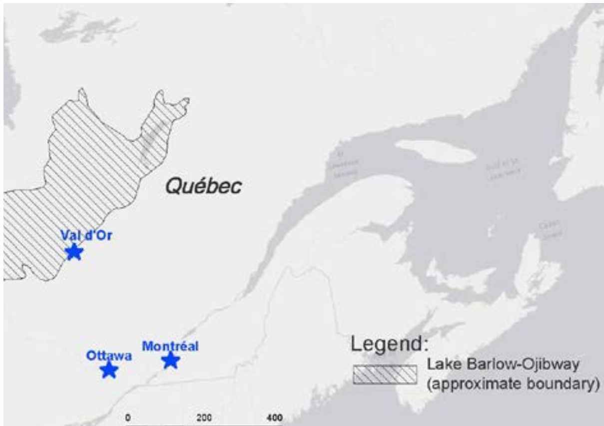


Figure 1. Approximate boundary of Lake Barlow-Ojibway

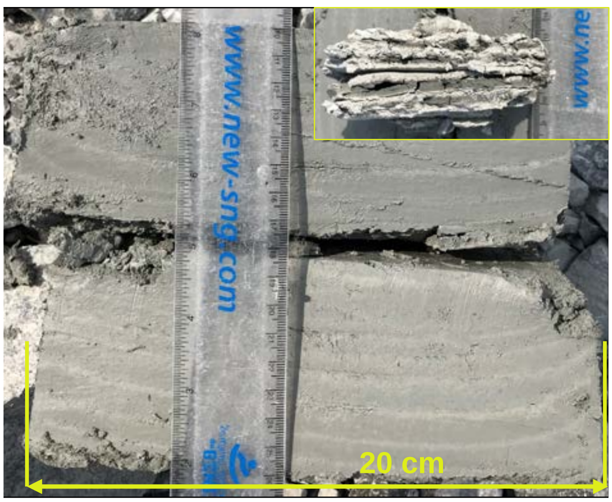


Figure 2. View of varves from wet and dried samples collected at the toe of the dike with the Laval sampler

A geotechnical investigation was undertaken in May 2018 to retrieve high quality samples with the Laval sampler (La Rochelle et al. 1981) in an area where the clay was expected to be overconsolidated (area where no previous loading was reported) and to submit those to specialized laboratory tests that would allow understanding its stress-strain behaviour with more confidence and determining its susceptibility to strain-weakening. To maximize the chances of collecting high quality samples, the boreholes were drilled at a location where marginal loading had occurred, downstream of an existing dike (see Figure 3). Two different samplers were used namely, a Laval sampler, which is 8 inches in diameter, and 2.875-inches thin wall Shelby tubes. The Laval samples were extracted and prepared at the site. All the samples were transported by car to three different laboratories (Université de Sherbrooke and 2 different Golder Associates Ltd laboratories in Ontario).

The geotechnical laboratory testing program included index testing; Specific Gravity (Gs), Particle Size Distributions (PSD), Water Content (Wn), Plastic Limits (PL) and Liquid Limits (LL); Swedish Cone (SC); Consolidation Tests by incremental loading (IL); monotonic Direct Simple Shear Tests (DSS); Cyclic Simple Shear Tests (CSS); Triaxial Tests (CIUc, CAUc and CAUe). The results of this investigation are presented herein. Additional information that has been gathered over the last years at the site are presented for comparison and discussion purposes.

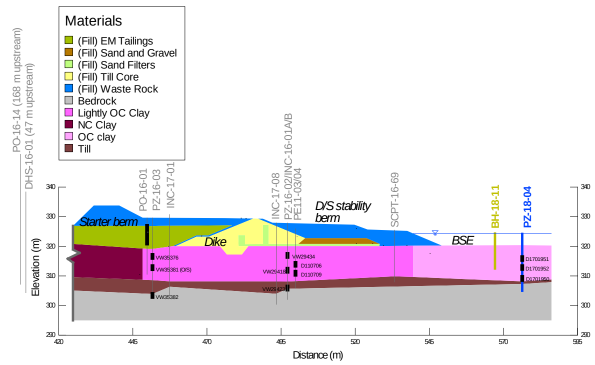


Figure 3. Cross-section of the dike and locations of soundings

4 CONSISTENCY LIMITS AND INDEX TESTS

4.1 Natural water content and consistency limits

The liquid and plastic limits conducted on the varved samples showed different clay types such as high, intermediate and low plasticity clays (CH, CI and CL) and low plasticity clayey silt to silt (CL-ML and ML). Figure 4 shows the profiles of Wn, LL and LP with elevation. The liquid limit (LL) was defined mainly using the Casagrande cup, but two values were defined using the Swedish cone. The values from the Swedish cone remain consistent with those defined using the Casagrande cup. The LL varies from 21% to 64% while the PL varies from 17 to 24%. The plasticity index (I_P) varies from 4 to 41%. The natural water content (Wn) varies from 28% to 68%, with liquidity index (I_L) above 1.0 for 16 out of 18 values. The laboratory test results obtained during the 2018 campaign remain consistent to those obtained in the past investigations. However, values obtained from the Laval samples are generally in the upper bound as shown in Figure 4. The water content, the plastic limit and the liquid limit are found to increase as elevation decreases until it reaches maximum values between 313 and 316 m approximately. The consistency limits then decrease with elevation.

4.2 Specific gravity, particle size distribution and mineralogy

On the eleven samples tested, the specific gravity of the clayey unit varies from 2.73 to 2.77 while the unit weight varies from 15.7 to 16.6 kN/m^3. Four PSD were performed on the clay samples collected during this campaign. The percentage of fines varies from 95% to 100% while the percentage of particles smaller than 0.02 µm is varying from 19% to 22% of the fine fraction.

Semi-Quantitative X-ray diffraction analyses were conducted on two clay samples retrieved approximately one kilometer North of the studied area. It was found that plagioclase, quartz, microcline and amphibole are the main mineral assemblages identified from this clay, which is consistent with results for Eastern Canada clay from Locat et al. (1984).

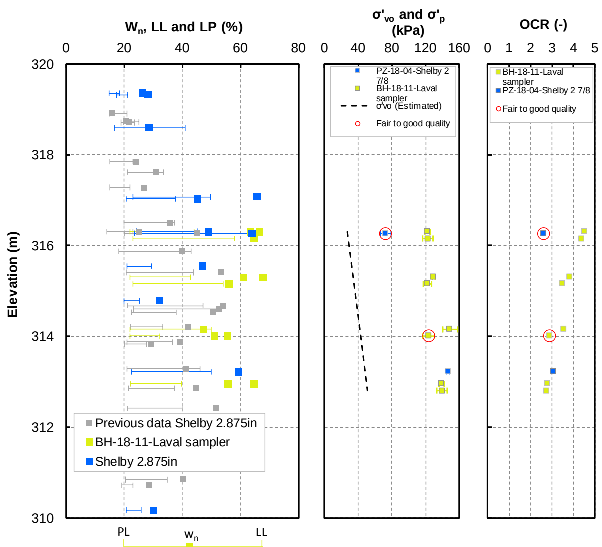


Figure 4. Variation of Wn, LL and LP, preconsolidation pressures and OCR with elevation

5 SOIL STATE, DISTURBANCE AND IL TESTS RESULTS

Eleven samples were submitted to IL tests. The sample quality was determined using the Sample Quality Category (SQC) from Lunne et al. (2006) and were found to be Excellent (SQC:1) for 9 specimens including 1 2.875-inches Shelby tubes and Fair to good quality (SQC:2) for 2 specimens including 1 2.875-inches Shelby tube. The initial void ratios are varying from 1.46 to 1.86. The OCR are varying from 2.38 to 4.89. Figure 4 shows the overconsolidation ratios (OCR) determined by comparing the preconsolidation pressures, obtained from the Casagrande method, to the in situ effective stress estimated assuming hydrostatic conditions. The preconsolidation pressure slightly increases with depth whereas the OCR decreases with depth. The recompression index (Cr) varies from 0.026 to 0.08 while the compression index (Cc) varies from 0.83 to 1.77. Two of the lowest Cc were obtained from samples that showed a SQC of 2. The results show that Cc can vary significantly within the same sample due to the varves. For instance, three IL tests were conducted on TM-3 (a, b-middle of sample and b-end of sample) and led to Cc of 1.75, 1.12 and 1.09 respectively.

The consolidation curves are presented in Figure 5. A comparison was made between samples collected using 2.875-inches tubes in borehole PZ-18-04 and the Laval samples from adjacent borehole BH-18-11. At 8.2 m approximately (Figure 5b), the compression curve from the 2.875-inches tubes samples is typical of slightly disturbed specimens as shown by its SQC of 2. A lower preconsolidation pressure and a lower Cc were obtained as compared to the Laval samples retrieved at a similar depth. In the other case (Figure 5c), the compression curves from the 2.875-inches tubes samples and the Laval samples showed a very similar behaviour. These two samples showed a SQC of 1. The IL test results showed that one of the two 2.875-inches tubes samples submitted to the testing program revealed a SQC of 1 while seven over the eight samples retrieved with the Laval sampler showed a SQC of 1. This reinforces the use of a larger diameter sampler to maximize the chances to collect high-quality samples.

The coefficient of consolidation above the preconsolidation pressure c_v obtained from the consolidation curves varies from 2.0x10^{-9} to 2.1x10^{-8} m^2/s while the compressibility m_v varies from 2.5x10^{-5} to 1.6x10^{-3} m^2/kN. Based on the consolidation curves obtained in laboratory, the vertical hydraulic conductivity k_v varies from 6.7x10^{-12} to 2.7x10^{-9} m/s past the preconsolidation pressure (normally consolidated part) while it varies between 7.9x10^{-12} and 1.5x10^{-10} m/s at σ'_{v0} estimated based on hydrostatic conditions.

Table 1. Results from IL tests

ID[1]	e_0 (-)	σ'_v (kPa)	σ'_p (kPa)[2]	OCR (-)	Cc (-)	Cr (-)	SQC (-)[3]
TS-6	1.78	28	67	2.38	0.94	0.039	2
TS-10	1.67	48	145	3.03	1.35	0.035	1
TM-3a	1.82	27	125	4.63	1.75	0.039	1
TM-3b (MOS)[4]	1.72	28	129	4.61	1.12	0.08	1
TM-3b (EOS)[5]	1.77	28	137	4.89	1.09	0.047	1
TM-4a	1.86	34	131	3.87	1.75	0.037	1
TM-4b	1.61	35	127	3.63	1.09	0.041	
TM-5b	1.46	42	158	3.76	0.84	0.031	1
TM-5c	1.53	43	131	3.04	0.83	0.026	2
TM-6a	1.78	50	142	2.85	1.77	0.039	1
TM-6b	1.46	51	146	2.86	1.02	0.059	1

[1] TM: Laval sampler / TS: 2.875 inches Shelby tube
[2] From Casagrande method
[3] Sample Quality Category from Lunne et al. (2006)
[4] MOS: Middle of sample
[5] EOS: End of sample

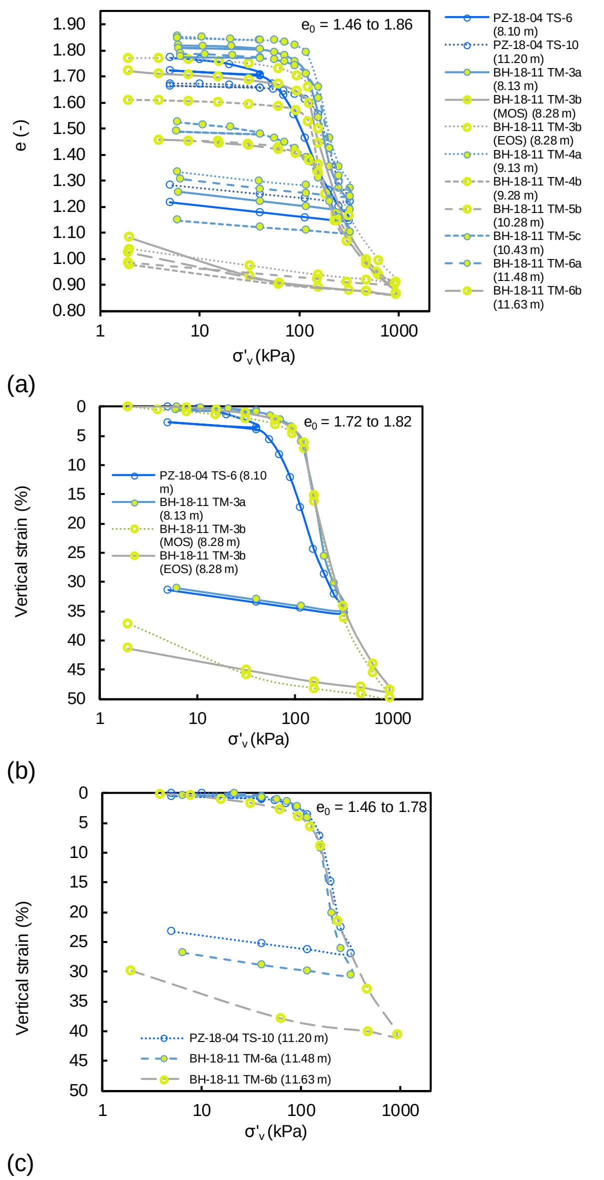


Figure 5. IL One-dimensional compression curves: (a) from different depths, (b) and (c) 2.875-inches tube-TS vs Laval samples-TM from two different depths

6 SMALL-STRAIN PROPERTIES, DEGRADATION AND DAMPING CURVES

One seismic cone penetration test including shear wave velocity measurements (V_s) was carried out in an area where the clay is marginally overconsolidated (at SCPT-16-69). At this location, a peak undrained strength between 32 kPa and 62 kPa was measured with the Nilcon. Maximum shear moduli (G_{max}) between 21 MPa and 45 MPa were estimated from in situ V_s measurements between 113 and 171 m/s (V_{s1} between 108 m/s and 158 m/s). It was found that the V_s values can be well estimated by the equation proposed by Hegazy and Mayne (2015).

The shear stiffness degradation (G/Gmax) curve and the damping curve were also defined from clay samples collected with 2.875-inches tubes from boreholes DHS-16-01 and PO-16-14 and then tested with the TxSS apparatus (Karray et al. 2016). Figure 6 shows the results obtained from the analysis of the TxSS results. The degradation and the damping curves remain consistent with the relations proposed by Vucetic and Dobry (1991) and largely used in the practice up to a strain level of 0.3%. Beyond this level, the results and more specifically the damping curve tend to be more consistent with more recent results proposed by Okur and Ansal (2007). It is important to note that the results proposed by Vucetic and Dobry (1991) and by Okur and Ansal (2007) are available for clay with low sensitivity. The sensitivity seems to play an important role on the damping curves at intermediate to large strains where the generation of pore water pressure could be related to the sensitivity of the clay. This means that adopting the existing damping curves would result in an amplification of the ground response at intermediate to large strain level compared with the proposed relationship of Vucetic and Dobry (1991).

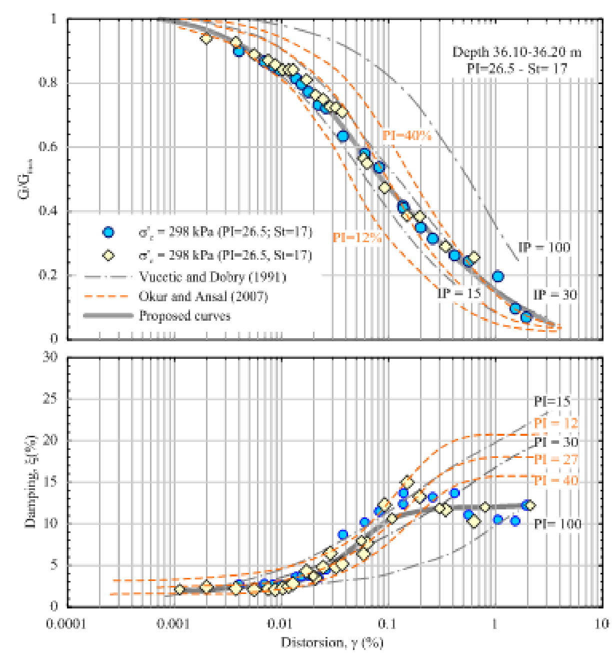


Figure 6. Shear stiffness and damping vs shear strain from two specimens trimmed from a 2.875-inches tube retrieved at PO-16-14 under the TSF

7 UNDRAINED SHEAR STRENGTH, SENSITIVITY AND BRITTLENESS INDEX

7.1 Field Vanes and Swedish Cone Test Results

One field vane profile (FVT) was carried out a few meters apart from BH-18-11 using a Nilcon device. The peak, post-peak and remoulded undrained shear strengths

(Su) were obtained for each depth. Peak and post-peak values were defined for a vane rotation of 6 °/min and remoulded values were measured after 20 turns of the vanes. Figure 7 shows the three profiles with elevation. The depths at which the 2.875-inches Shelby tubes TS-6 and TS-10, collected from PZ-18-04, and the four Laval samples collected from BH-18-11 (TM-3 to TM-6) were retrieved, are also shown in Figure 7. These samples were used for most of the laboratory testing program and the subsequent analyses.

The consistency of the clay is firm based on peak Su generally between 25 kPa and 40 kPa. The post-peak Su is generally half the peak Su with sensitivity (S_t) varying between 1.5 and 2.4. S_t estimated when comparing the peak to the remoulded values obtained from the Nilcon, are between 4.9 and 23, while it varies between 15.0 and 21.8 based on the peak and remoulded Su obtained from the Swedish cone test results. These S_t values are typically associated to extra-sensitive to quick clays (CGS, 2012). Brittleness index using peak and remoulded Su from the Nilcon and the Swedish cone, [I_B=(Su_{peak}-$Su_{remoulded}$)/Su_{peak}] are between 0.80 and 0.96 while I_B using the post-peak values from the Nilcon are between 0.32 and 0.59.

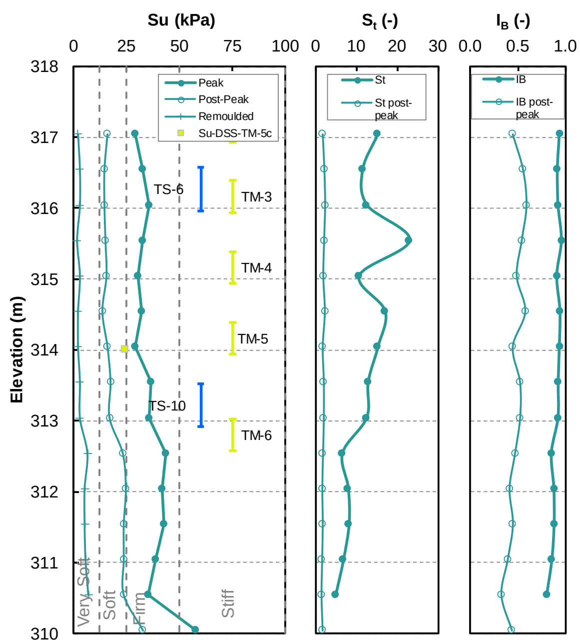


Figure 7. Undrained shear strength profiles from Nilcon at PZ-18-04, Sensitivity (S_t) and Brittleness Index (I_B)

7.2 Triaxial Test Results

Table 2 presents a summary of the four triaxial tests conducted on the Laval samples. Each confining stress was selected to reproduce different consolidation or shear stress paths. Specimens TM-3a and TM-4a underwent an anisotropic consolidation stage using a K (ratio of horizontal effective stress,σ'_h to vertical effective stress, σ'_v) of 1.5 to represent stress-induced anisotropy at the toe of the dike. They were then sheared in extension and compression, respectively. This K has been defined based on 2D finite difference stress-strain analyses of the dike. The confining stresses were chosen to remain within the overconsolidated range. TM-5c was consolidated isotropically and then shear in compression to examine any differences between consolidation modes (CIUc vs CAUc). TM-6a was consolidated to a normally consolidated state using a K of 0.6 and then sheared in compression. This K was estimated from Kulhawy and Mayne (1990) to represent the normally consolidated state. The consolidation phase was performed in three stages for TM-3a, TM-4a and TM-5c while TM-6a was consolidated in 5 stages in order not to overstress the specimen. A rate of shearing of 0.5%/h was used in all triaxial tests.

Table 2. Results from triaxial tests

ID	σ'_{vc} (kPa)	K (-)	OCR (-)	S_u=q_{peak}/2 (kPa)	S_u/σ'_{vc} (-)	γ_{peak} (%)
TM-3a *CAUe*	40	1.5	3.13	23.4	0.59	0.5
TM-4a *CAUc*	40	1.5	3.29	52.9	1.32	2.4
TM-5c *CIUc*	40	1.0	3.27	27.6	0.70	5.9
TM-6a *CAUc*	250	0.6	1.00	99.0	0.40	0.9

A peak undrained shear strength ratio (S_u/σ'_{vc}) of 0.59 was obtained in extension. Specimen TM-4a that was consolidated in the same conditions, but then sheared in compression showed a peak undrained shear ratio S_u/σ'_{vc} of 1.32. Those two ratios are typically high for such a material. However, the ratio $(S_u/\sigma'_{vc})_{ext.}/(S_u/\sigma'_{vc})_{comp.}$ remains consistent with the literature (Kelly et al. 2016) with a value of 0.44. TM-5c and TM-6a resulted in (S_u/σ'_{vc}) of 0.7 and 0.4, respectively, which remains consistent with the literature for such OCRs (3.27 and 1.00 respectively).

Figures 8 and 9 show the deviator stress versus the vertical strain and the stress paths for each triaxial tests, respectively. Figure 8 shows that the peak deviator stress (q_{peak}) was reached at a vertical strain between 0.5% in CAUe test and at 5.9% in CIUc test. Specimens that underwent anisotropic consolidation stages (CAUe and CAUc) underwent more strain weakening ([I_B=(Su_{peak}-$Su_{post\text{-}peak}$)/Su_{peak}] between 0.26 and 0.32) than TM-5c (I_B=0.18) that was isotropically consolidated. These specimens also reached the peak deviator stress at a lower strain level than the isotropically consolidated specimen (TM-5c).

Figure 9 shows that two failure lines could be defined from the compression tests (31° and 43°), and this suggests that this clay deposit presents a spatial variability that might be due to its varved nature. The evolution of the principal stress ratios (q/p') versus vertical strain (not shown herein) yielded to the same observation.

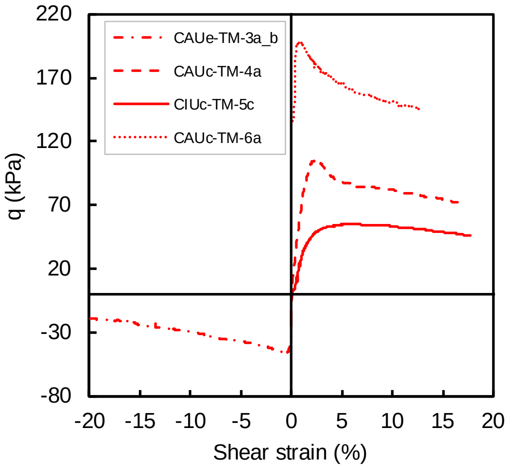


Figure 8. Deviator stress versus shear strain in undrained shear in triaxial conditions

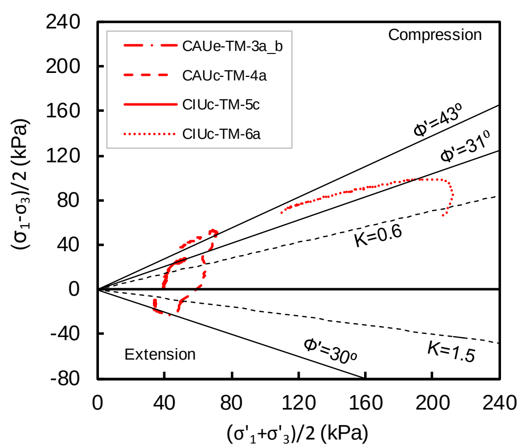


Figure 9. Effective stress path in undrained shear in triaxial conditions

7.3 Undrained Monotonic Direct Simple Shear (DSS) Test Results

Table 3 shows the results of the undrained DSS tests carried out on Laval samples and 2.875-inches tubes samples. 2.875-inches tubes samples submitted to DSS tests were collected underneath the TSF where the clay is now normally consolidated. Undrained DSS were conducted in the TxSS apparatus (Karray et al. 2016) in DSS mode, thus in truly undrained simple shear condition rather than constant-volume. The evolution of undrained shear strength ratios versus the shear strain for each DSS is presented in Figure 10. The vertical confining stresses (associated with values of S_u/σ'_{vc}) applied to clay samples during the consolidation phase were defined to reproduce both normally and overconsolidated states. A rate of shearing between 1%/h and 9.7%/h was used in the DSS tests.

Most of the samples underwent only slight strain weakening in simple shear as shown by brittleness indexes ($[I_B=(Su_{peak}-Su_{post\text{-}peak})/Su_{peak}]$) generally below 10%. The undrained stress ratios measured at different OCRs is consistent with previous results published on varved clays from the same geological deposit (Carlsen 1972). In return, at low OCR, the undrained stress ratios seem higher than those published by Lutenneger and DeGroot (2002) and Ladd and DeGroot (2004) for Connecticut Valley Varved Clays.

Table 3. Results from DSS

ID	σ'_{vc} (kPa)	OCR (-)	$S_{u,peak}$ (kPa)	$S_{u,peak}/\sigma'_{vc}$ (-)	γ_{peak} (%)	I_B (-)[1]
TM-3b	102	1.27	32	0.31	25.0	0.0
TM-4b_a	71	1.79	33	0.46	3.2	0.13
TM-4b_b	89	1.43	32	0.36	2.4	0.25
TM-4b_c	68	1.87	27	0.40	3.5	0.09
TM-5b_b	55	2.87	23	0.42	8.0	0.0
TM-6b	255	1.00	59	0.23	7.4	0.07
17-01 (TS-12)	52	3.75	26	0.51	20	0.03
	146	1.34	42	0.29	8.0	0.09
	358	1.00	78	0.22	10.7	0.08
16-01 (TS-11)	300	1.00	68	0.23	10.9	0.06
16-14 (TS-13)	420	1.00	86	0.20	9.2	0.11

[1]$I_B=(S_{u,peak}-S_{u,post\text{-}peak})/S_{u,peak}$

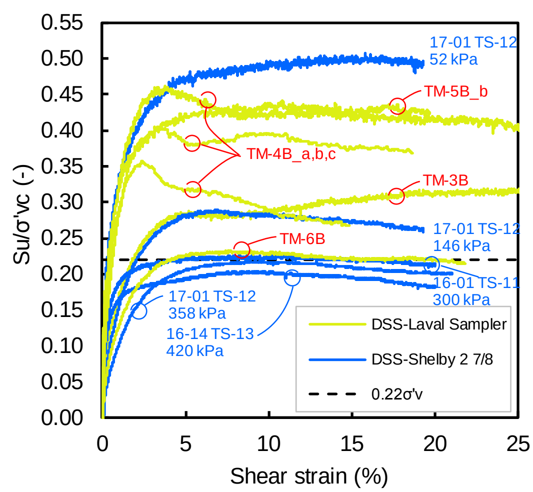
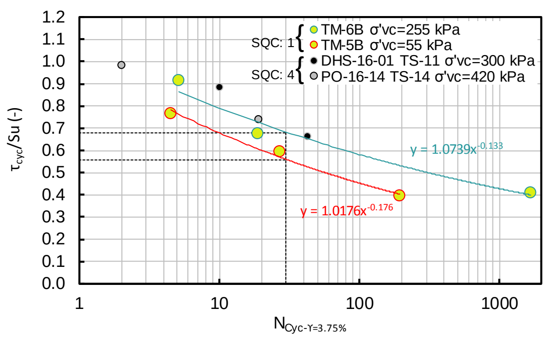


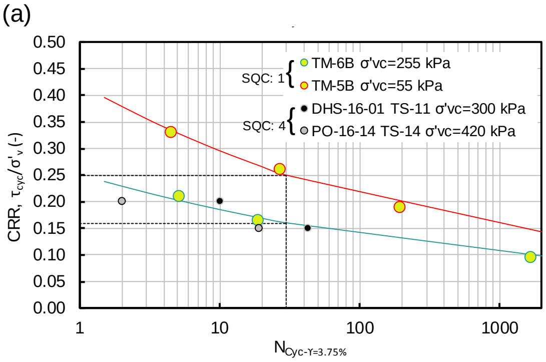
Figure 10. Undrained shear strength ratio versus shear strain in Direct Simple Shear (DSS)

7.4 Constant-volume Cyclic Direct Simple Shear (CDSS) Test Results

Table 4 shows the results of constant-volume CDSS carried out on Laval samples collected at the toe of the dike and 2.875-inches tubes samples collected underneath the TSF. The samples were submitted to a sinusoidal displacement function with a frequency of 0.1 Hz using a GDS cyclic simple shear apparatus. Figure 11a shows the ratio (τ_{cyc}/S_u) versus the number of cycles to reach 3.75% of cyclic shear strain. At a number of 30 cycles, τ_{cyc}/S_u is between 0.66 and 0.75 for the normally consolidated specimens and 0.56 for the overconsolidated specimens. According to Idriss and Boulanger (2008), 30 uniform loading cycles represents a magnitude of 7.5. The cyclic resistance ratio (CRR) is shown in Figure 11b. For the same number of cycles, a CRR of 0.16 and 0.25 was obtained for the normally consolidated specimens and the overconsolidated specimens, respectively.



(a)



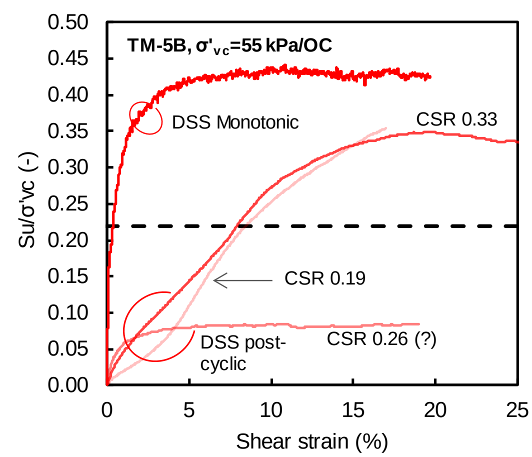
(b)

Figure 11. Results from CSS: (a) τ_{cyc}/S_u, (b) CRR

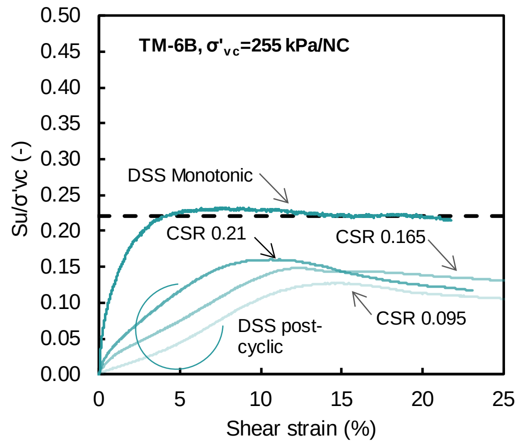
Specimens were submitted to a post-cyclic monotonic simple shear test following the cyclic test. Figure 12a shows Su/σ'vc versus shear strain for the post-cyclic tests results for the overconsolidated specimens TM-5B and Figure 12b shows the results for the normally consolidated specimens TM-6B. The monotonic DSS tests results are shown for comparisons purposes. In both cases, the specimens tested in post-cyclic show a smaller rigidity and a reduction of the undrained stress ratio (Su/σ'vc) with strain. The post-cyclic simple shear test identified by a CSR of 0.26 resulted in a low ratio Su/σ'vc and it is believed that something went wrong during the test. The overconsolidated specimens yielded in ratios of the post-cyclic undrained shear strength to the monotonic DSS (Su,p-c/Su,DSS) between 83% and 85%. Normally consolidated specimens resulted in ratios Su,p-c/Su,DSS between 55% and 69% for the Laval samples and between 68% and 88% for the 2.875-inches tubes.

Table 4. Results from CDSS

ID/ CSR	σ'vc (kPa)	OCR (-)	$N_{cyc,\ \gamma=3.75\%}$ (-)	τ_{cyc}/S_u (-)	τ_{cyc}/S_u $N_{cyc=30}$ (-)	$S_{u,p\text{-}c}/S_{u,DSS}$ (%)
TM-5b						
0.19	45.6	3.46	194	0.40		84.6
0.26	56.6	2.95	27	0.59	0.56	20.1 (?)
0.33	53.0	2.98	4.5	0.77		83.1
TM-6b						
0.095	258	1.00	1670	0.41		54.8
0.165	249	1.00	18.8	0.67	0.68	63.7
0.21	253	1.00	5.2	0.92		68.7
16-01 (TS-11)						
0.15	300	1.00	43	0.66	0.75	72
0.20	300	1.00	10	0.88		80
16-14 (TS-13)						
0.15	420	1.00	19	0.74	0.66	88
0.20	420	1.00	2	0.98		68



(a)



(b)

Figure 12. Post-cyclic undrained direct simple shear: (a) OC specimens, (b) NC specimens

8 CONCLUSION

A geotechnical investigation was conducted and samples were collected using the Laval sampler and conventional 2.875-inches Shelby tubes, at the toe of a Tailings Storage Facility dike located in Abitibi. The objective was to collect high quality varved clay samples to conduct a series of geotechnical laboratory tests. Nine out of eleven samples collected with these samplers showed an excellent quality (SQC: 1) according to Lunne et al. (2006). The sole Laval sample that showed a fair to good quality was collected in a zone of lower plasticity varved soil. One of the two Shelby tubes collected showed an excellent quality (SQC:1) while the second one revealed a fair to good quality (SQC:2). The laboratory testing program involves index characterization tests, one-dimensional compression tests, CAUc, CAUe and CIUc triaxial tests, monotonic and cyclic direct simple shear tests.

The samples collected showed that the varves are made from a low plasticity silt (ML) to a high plasticity clay (CH) of firm consistency based on the in situ peak undrained shear strength measured with the Nilcon vane. One-dimensional compression tests showed overconsolildation ratios between 2.6 and 4.9 decreasing with depth.

The Nilcon vane undrained shear strength yielded to sensitivity between 4.9 and 23 when calculated from the peak and the remoulded values at same depth, while peak and post-peak values resulted in sensitivity between 1.5 and 2.4. These undrained shear strengths yielded I_B values between 0.80 and 0.96 (peak and remoulded) while I_B using the post-peak values are between 0.32 and 0.59.

Numerous clay specimens were tested in triaxial and in simple shear conditions at different confining stresses defined to reproduced both the normally and overconsolidated states. The clay specimens tested in triaxial conditions underwent strain weakening with I_B (peak and post-peak) values varying between 0.18 and 0.32. Most of the samples tested in monotonic simple shear underwent only slight strain weakening as shown by I_B (peak and post-peak) generally below 0.10. These results suggest that this clay would typically undergo a ductile stress-strain behaviour under simple shear up to a shear strain level of 20% but could undergo significant strain weakening under compression and extension shear modes. The post-peak Nilcon test results showed that this clay could undergo a slight strain weakening beyond 20% shear strain and reach half of the peak Su. The undrained strength ratios measured at different OCR and different shear modes are consistent with previous results published on varved clays from the same geological deposit.

Cyclic simple shear tests resulted in a CRR of 0.16 for the normally consolidated specimens and 0.25 for the overconsolidated specimens when loaded under 30 uniform cycles at a loading frequency of 0.1 Hz. Post-cyclic simple shear tests resulted in a decrease varying between 15% and 17% of the peak monotonic undrained strength for the overconsolidated specimens, and between 12% and 45% for the normally consolidated specimens.

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