Ground characteristics for the design of the Edmonton North LRT Twin Tunnels

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ABSTRACT
A comprehensive geotechnical investigation was undertaken as part of the detailed design of the City of Edmonton's North LRT expansion. The investigation included a series of in-situ testholes including a number of seismic CPTu holes and a pre-bored pressuremeter hole. The strength and deformation parameters obtained from the current investigation were then compared to the findings of a number of historical investigations associated with previous LRT extensions as well as several heavy civil projects located throughout the City of Edmonton.

RÉSUMÉ
Une étude géotechnique complète a été entreprise dans le cadre de la conception détaillée de l'expansion nord du réseau de train léger sur rail (LRT) de la ville d'Edmonton. L'étude comprend une série de forages d'essai in-situ dont quelques essais de pénétration au cône sismique CPTu et un essai au pressiomètre préforé. Les paramètres de résistance et de déformation obtenus dans cette étude sont comparés aux résultats de plusieurs études antérieures concernant des expansions passées du LRT ainsi que plusieurs grands projets de génie civil réalisés à travers la ville d'Edmonton.

1 INTRODUCTION

Over the past 40 years, the City of Edmonton has developed a Light Rail Transit (LRT) system consisting of both surface and subsurface rails. The LRT system originated in the city centre and was constructed primarily underground in the 70's and 80's. The alignment to the southern part of the city, consisting of above and below ground track, was constructed in the late 90s and early 2000 and is generally referred to as the SLRT extension. The City of Edmonton is currently preparing for the construction of the North LRT expansion to connect the Northern Alberta Institute of Technology (NAIT) with the existing infrastructure. This expansion will include construction of twin, Sequentially Excavated tunnels starting at the existing Churchill Station in the south and extending north to the Station Lands Station, where the alignment turns west and surfaces at MacEwan College Station. The remainder of the alignment will continue north on surface tracks to the NAIT campus.

Each LRT expansion has undergone a comprehensive geotechnical investigation as part of the alignment detailed design. The early investigations relied on more traditional borehole investigation techniques to establish the ground characteristics. Considerable effort in the early phase of the LRT development was carried out to establish the laboratory properties of Edmonton soils. Site investigations consisted of conventional auger drilling with disturbed samples recovered using the Standard Penetration Test (SPT) at regular intervals. In most cases, relatively undisturbed Shelby tube samples have also been recovered and subjected to rigorous laboratory testing. In past investigations, laboratory testing has consisted of Unconfined Compression (UC) tests, Unconsolidated Undrained (UU), Consolidated Undrained (CU) and Consolidated Drained (CD) triaxial tests. Research programs carried out at the University of Alberta have also historically been undertaken to provide additional characterization for various aspects of the City's LRT expansion as well as for the construction of several down-town high rise structures.

With the improvement to in-situ testing technology and advanced data logger systems, the site investigation techniques for the various LRT extensions have evolved to incorporate more reliance on in-situ testing and less laboratory testing. One aspect of the geotechnical design that has not changed significantly over the years is the difficulty associated with determining an appropriate modulus of deformation to predict tunnel induced displacements within heavily overconsolidated materials common to most of Edmonton and the surrounding areas. Since the early 1970's in-situ testing has been used in the City of Edmonton to help estimate stiffness parameters and the variation to the state of stress within the various formations. These early in-situ studies were undertaken not only as part of the LRT rail expansion and the associated network of subsurface stations, but also for other heavy civil projects throughout the city. In-situ testing has typically consisted of pressuremeter (PM) testing and in more recent years, the use of Seismic Cone Penetration Tests (SCPTu). This paper discusses the results of the previous testing for the various LRT extensions and structures as well as presents updated data from the most recent series of investigations carried out as part of the NLRT expansion. The laboratory geotechnical properties are compared to those obtained from the in-situ testing.

2 GEOLOGICAL SETTING

The soil stratigraphy throughout the City of Edmonton is generally very consistent due to the depositional history associated with the advancement and retreat of the Cordilleran and Laurentide ice sheets during the last ice age (May and Thomson, 1978). Slight variations are known to occur throughout the city, particularly near the North Saskatchewan River valley, but in general, the subsurface conditions summarized below and illustrated in Figure 1 have been encountered along the LRT alignment.

Lacustrine Clay: The majority of the surficial materials throughout the city consist of a glacio-lacustrine silt and clay known as the Lake Edmonton Clay (LEC). The silt and clay is generally firm to stiff and is typically about 4 to 6 metres thick. The LEC is rhythmically bedded though the spacing of the varves are generally quite wide and are typically very difficult to observe visually in most site investigations.

Outwash Sands: In some parts of the city, a layer of fine to medium grained (5% to 15% silt), uniformly graded “hour-glass” sand is encountered immediately below the Lake Edmonton Clays. Outwash sands are in-filled channels of pre-Laurentide fine sands. Typically the sand is compact to very dense and was encountered throughout the alignment of the University to Jubilee SLRT stations. The thickness of the outwash sands along the SLRT alignment was reported to vary from between 8 to 11 m (Boone et al. 2002), though thicker deposits exist in several areas further south of Whitemud Dr.

Glacial Till: The Glacial Till (Till) deposit is approximately 6 to 16 metres thick and generally very stiff to hard. According to Thomson et al (1981) local ice advances and retreats resulted in two till sheets referred to as the upper and lower tills. The lower till has a thickness of approximately 6 m and is greyish in colour with prominent rectangular jointing. The upper till is brownish and columnar-jointed. Like most tills, the glacial till is highly fissured with frequent oxidation stains located in the fissures. The intact strength of the glacial till is considerably stronger than the soil mass as a whole due to its blocky nature. The till was laid down on the Saskatchewan Sands and Gravels in the preglacial channels or in contact with the bedrock in the upland areas. Pockets of Intra-Till Sand with a varying percentage of fines are randomly encountered within the glacial till. The size and extent of the sand pockets can vary significantly from tens of centimetres to several metres in length. Cobbles and boulders are also present within the soil matrix, though are not frequently encountered. The interface between the glacial till and the LEC is gradational (where the outwash sands are not present) whereas the contact with the underlying sand and gravel formation is generally sharp and planar as observed in exposures throughout the city.

Empress Formation (Saskatchewan Sand and Gravel): Below the glacial till the pre-Laurentide sand and gravel deposits may be encountered in preglacial bedrock channels. These deposits have only been observed along the NLRT alignment and here the Empress Formation is approximately 13 metre thick. These sediments are generally compact to very dense and tend to fine upwards towards the contact with the overlying glacial till. The deposits are frequently cross-bedded with coarser deposits of sand and gravel and also with silts and clays.

It is not uncommon to encounter laminations of high plastic clay within the sands varying between 10 and 100 mm thick. Slickensided clay laminations were observed at Elevation 643 m along the NLRT alignment. These discontinuous bedded clay laminations observed at Station Lands showed 30 percent silt sizes and 70 percent clay sizes, with Liquid Limits ranging between 72 and 82 percent and Plastic Limits between 24 and 27 percent.

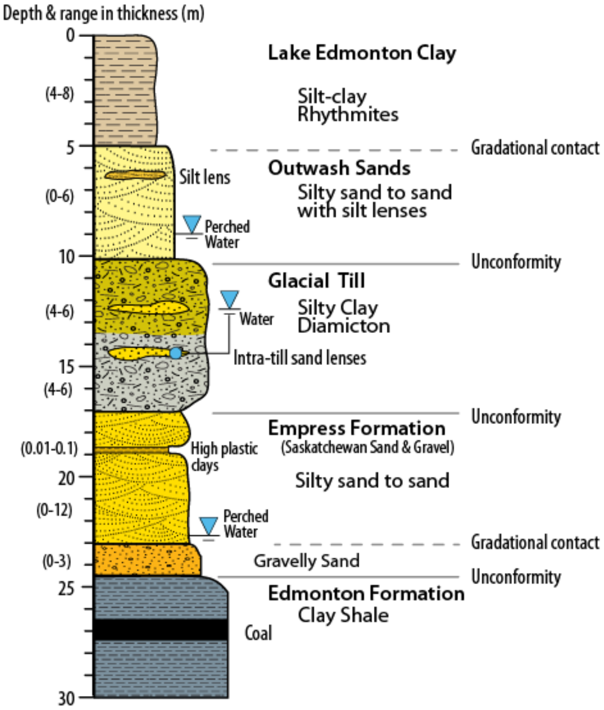

Figure 1: Generalized stratigraphic profile encountered along City of Edmonton’s LRT alignment

Bedrock: Edmonton Formation Bedrock consisting of soft sedimentary deposits of claystone, sandstone and siltstone with varying thickness is encountered below the Empress Formation. This sequence is commonly referred to as Clay shale. The contact with the SSG formation above is sharp and for the most part planar with a slight dip to the northeast within the city. Intermittent coal beds of variable thickness and extent are known to be present within the Edmonton Formation and historical mines are located throughout the North Saskatchewan River valley. The Edmonton Formation also contains up to centimetre thick bentonite layers in the upper portions of the bedrock. These layers are exposed in the river valley and often form part of the rupture surface for river valley landslides.

Groundwater: Perched groundwater is occasionally encountered on the surface of the glacial till but is most commonly encountered within the Intra-till sand pockets. These saturated sand pockets are the major source of seepage and soil instabilities during tunnel construction. Groundwater seepage rates of 4 to 5 L/min can be expected when these sand pockets are encountered.

Where the Empress Formation is encountered it often acts as an under-drain for the overlying soils with the groundwater occurring as a perched water table metres above the surface of the Clay shale bedrock.

3 GEOTECHNICAL CHARACTERISTICS

3.1 Laboratory Strength Testing

A series of laboratory studies have examined the Till in an attempt to quantify the short-term and long-term strength parameters. The majority of the samples were recovered using conventional Shelby tubes, though several investigations took advantage of deep excavations in the downtown area of the city to collect block samples for testing. Several studies also used specially designed sampling tools that were used exclusively for their investigations. A brief description of the various studies collected to date is provided below.

DeJong (1971) tested samples trimmed from block samples recovered from the foundation excavations of the CN Tower with marginal success. Three samples were prepared for UU tests and only 2 samples were of sufficient quality for CU tests. DeJong reported a high degree of scatter associated with the testing, which he attributed to the unloading stresses and disturbance that occurred during sampling and test preparation.

Morgenstern and Thomson (1971) attempted to demonstrate the degree of disturbance typical to samples recovered from the Till by testing a total of 37 samples prepared from block samples, Shelby tubes and a pitcher sampler specially designed at the University of Alberta for sampling of the Till. The UU tests indicated a high degree of disturbance in the specimens prepared from the block samples which resulted in the lowest strength. These tests suggested that samples recovered from Shelby tubes resulted in data that, though, slightly lower than those recovered using the pitcher sampler, were still consistent and likely representative of the intact strength of the Till.

A series of UC and UU tests carried out as part of the initial North East LRT extension (Doohan et al., 1975) between Central and Churchill Stations indicated a wide range of compressive strength of the Till. The UU test results indicated compressive strengths close to that of rock while the unconfined compression tests tended to fall within a range that are typically reported for the Till within the city

Medeiros (1979) carried out a series drained triaxial and plane strain tests on block samples as part of the excavation design associated with the Central LRT Station. In an attempt to not alter the natural moisture content, Medeiros did not saturate any of the samples prior to commencing the experiment. In addition, because the coefficient of consolidation was approximately 0.02 cm^2/s, complete consolidation of the samples was not carried out prior to testing, nor was it believed that full consolidation could be achieved using the equipment available at the time as the pre-consolidation pressure was estimated to be in the 1 to 1.5 MPa range. Medeiros reported that the materials, though heavily overconsolidated exhibited strain hardening profiles while the pore pressure measurements corresponded with dilation as expected.

Tweedie and Smith (1980) used UU tests on Shelby tube samples as part of the geotechnical characterization for the LRT extension from Central Station to the Government Centre Station. El-Nahhas and Harris (1981) also carried out a series of UU tests on trimmed samples from intact block samples as part of the detailed design of the Government Centre Station on the same project. Both of their findings suggested that reasonably consistent results could be obtained from both Shelby tube and block samples.

Whittebolle (1983) carried out a series of UU and CD triaxial tests on block samples to help better understand the fabric of the Till. Like Medeiros, Whittebolle did not fully saturate or consolidate the samples prior to testing in order to maintain the natural moisture content. Whittebolle did not report the moisture contents of the samples that he tested and therefore his strength values were not comparable to the other tests where the moisture content was reported. Review of the stress-strain profiles, like Medeiros, indicated that the Till was strain hardening and dilation in the pore pressure measurements, despite the known overconsolidation.

Soliman and McRoberts (2009) conducted three UC tests on samples recovered from the Till as part of the investigation associated with the detailed design of the current NLRT extension. As part of the NLRT detailed design the authors conducted a total of nine UU tests on block samples recovered from the Station Lands excavation. The results of these data correspond well with the existing data.

The results of the various investigations indicate a relatively wide range of scatter suggesting the high degree of disturbance and/or possibly state dependence of the material. The results of the historical laboratory strength testing plotted against the natural moisture content are shown below in Figure 2.

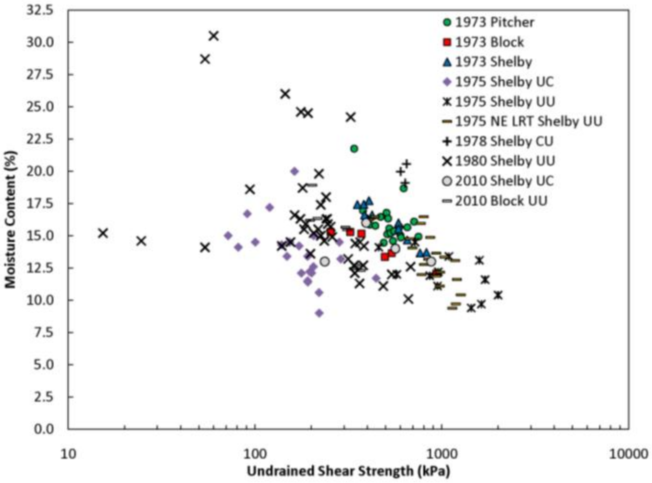

Figure 2: Summary of glacial till undrained shear strength.

3.2 Early Experience with In-Situ Testing

A number of in-situ investigations have been carried out in the past throughout the city as part of several projects ranging from foundation construction to deep excavations associated with several LRT stations. Morrison (1972) was the first to publish results of Menard Pre-Bore Pressuremeter (PBPM) testing carried out near to the CN and the Alberta Government Telephone (AGT) towers. His tests focused mainly on the Till, SSG and the underlying clay shale bedrock. Due to technical limitations at the time of the tests, very little was known in terms of data recovery for further interpretation. As a result, none of the tests were carried out beyond the initial yielding of the soil, nor were there any unload-reload curves recorded to better observe the shear modulus. Consequently, there was no measurement of the unloading curve shown by Houslby et al. (1986) to be necessary for assessing the undrained shear strength of the soil. The results however indicated a consistent deformation modulus with depth within the Till at approximately 100 to 175 MPa. The deformation modulus within the SSG was reported to range from 150 to 200 MPa.

DeJong (1971) monitored the settlement associated with the CN Tower, Avord Arms and the AGT Tower. By measuring the settlement of the three structures over time, and possessing a detailed knowledge of the foundation system, DeJong was able to back calculate the deformation modulus for two of the three sites. Because the CN and AGT Towers were founded on spread footings, DeJong was able to accurately assess the applied loads with time. The Avord Arms however presented significant difficulties related to estimating the applied loads due to the unknown dimensions of the Franki piles installed as the foundation. Back calculations from measured displacements of structures can be further complicated due to subsurface anomalies below the foundation. In the City of Edmonton, because the stratification is mostly level it provides a good basis for eliminating the error associated with variable site conditions and as a result, the findings of DeJong are likely representative of the site conditions for the two towers founded on spread footings. Based on the CN Tower monitoring program, DeJong found that there was an influence to the deformation modulus related to the general position of the footings. He observed that the footings constructed around the perimeter of the tower experienced lower loads than the mat placed in the central core of the structure. As a result, the deformations and consequently, the deformation modulus varied from the exterior to the interior with the core footings exhibiting higher moduli. These differences decreased until convergence near completion of the tower and the design loads for each footing were realized where an overall deformation modulus of approximately 450 MPa was calculated.

Smith and Harris (1980) conducted a series of Self Bore Pressuremeter (SBPM) and PBPM tests as part of the Bay and Corona LRT Stations located on Jasper Avenue at 104th and 107th Streets, respectively. The focus of the investigations was to obtain the deformation moduli of the Till and SSG as part of the station designs. The SBPM and PBPM probes were early versions of the Camkometer, however modern datalogging equipment was not available at that time and much like the Menard tests carried out by Morrison, the data was recorded manually. Also, the test methodology had not developed to include multiple unload-reload curves so the deformation modulus was assessed using only the initial loading curve. The deformation modulus was found to vary from 22 to 170 MPa in the Till, though several tests were considered by the operator to be disturbed. Within the SSG, the deformation modulus was measured to range from 44 to 93 MPa. There was no indication of disturbance with respect to these tests.

A series of SBPM tests were carried out by Tweedie et al. (1986) as part of the detailed design of the University Station construction. The main focus of the investigation was to ascertain the deformation modulus within the clay shale for the construction of a secant pile wall, though several tests were carried out in the Till. For this investigation, a Camkometer was also used and testing methods had included at least one unload-reload curve as well as measurement of the final unload curve. The results of the University Station testing within the Till indicated a deformation modulus of 310 MPa.

As part of the twin tunnel design for the SLRT extension from University Station to Jubilee Station, a single SCPTu test was carried out near to the north side of the Education Car Park. The SCPTu was advanced to a depth of 16.75 m depth and the shear waves were initiated using a Buffalo Gun at the ground surface. The estimated range of deformation modulus for the Till based on a Poisson's ratio of 0.4 was 66 to 320 MPa. The deformation modulus reported here is corrected to engineering strain assuming that a reduction of 30% is required to convert from a very small dynamic strain to a static engineering strain (Mayne, 2001).

The results of the current and previous measurements of the deformation modulus as reported since the early 1970's are shown below in Figure 3.

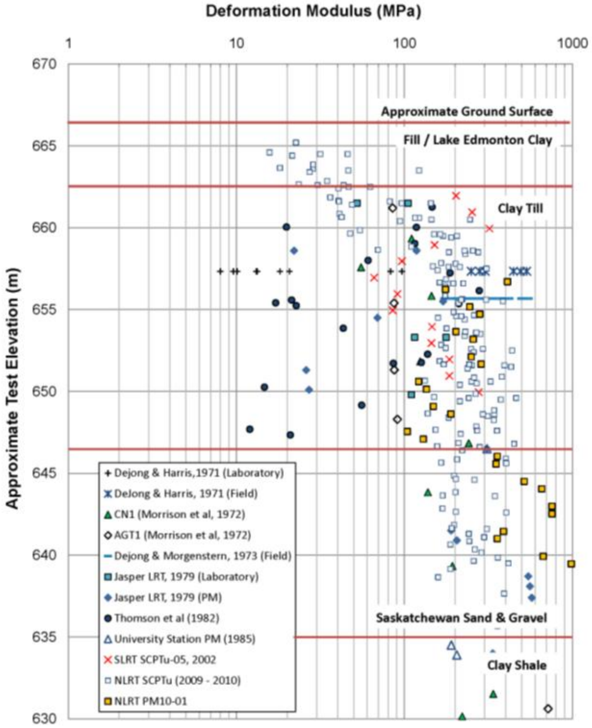

Figure 3 – Deformation modulus for the City of Edmonton

3.3 Current In-Situ Investigations

As part of the NLRT extension project, a series of 8 SCPTu tests and one PBPM borehole were advanced throughout the proposed alignment. The focus of the investigation was to characterize the strength and deformation modulus of the soils throughout the twin tunnel alignment as they relate to SEM tunnelling though the Till, SSG and mixed face conditions combining the Till and SSG.

The SCPTu tests were positioned throughout the twin tunnel alignment and were advanced to refusal. In some cases, upon meeting refusal, the cone was removed from the testhole and a portion of the very dense/hard material was augered out. Following excavation of the borehole material, the SCPTu was continued at a depth 1 to 2 m below the previous refusal depth. Seismic data was recorded at regular intervals throughout each test using a beam and hammer system at the ground surface. Based on the seismic tests, the deformation modulus within the Till ranged from 94 to 462 MPa, though was typically around 225 MPa. Within the SSG, the deformation modulus was found to range from 175 to 525 MPa, though was typically around 330 MPa.

For the PBPM tests, the borehole was advanced to a depth of 6.4 m below the ground surface prior to testing in order to ensure that the test pockets were within the Till. In each case, the borehole was advanced using a 70 mm diameter tri-cone bit with mud rotary drilling methods. The hole was advanced by 1.5 m increments providing space for two test pockets per advancement. Following advancement of the hole, the PBPM was inserted to the base of the test pocket and the test commenced. Upon completion of the test, the probe was lifted 0.75 m and each step was repeated. In each case, the borehole was loaded past the yield point to a maximum radial strain of around 10%. The stress-strain curve was monitored in real time at the ground surface throughout the test. Strains of the individual feeler arm were also displayed and in the event that any individual arm indicated excessive strain relative to the others, the test was terminated in order to prevent damage to the membrane. Each test was subjected to at least 3 to 4 unload-reload curves in order to assess the elastic deformation modulus. In most cases, the slopes of unload-reload curves were found to be parallel and the relative hysteresis between the unload-reload curves was also considered small suggesting accurate results.

The nature of the material is also clearly indicated in the real-time output of the stress-strain curves. Cohesive materials typically take the shape of an elastic-perfectly plastic stress strain curve, similar to the analytical method described by Gibson and Anderson (1961). Cohesionless materials tend to exhibit an elasto-plastic shape characteristic of the frictional nature of the soil. Hughes et al. (1977) provide an analytical method for determining the friction angle and dilation angle for very dense purely frictional materials. Within the Till, the stress-strain curves resembled either purely cohesive materials or a cohesive-frictional material, while the SSG was generally a typical very dense frictional curve. Typical stress-stress strain curves from the Till and SSG are shown together in Figure 4 to illustrate the contrast of the stress-strain profiles. Each plot is shown with its respective analytical result to show the general accuracy of the model used.

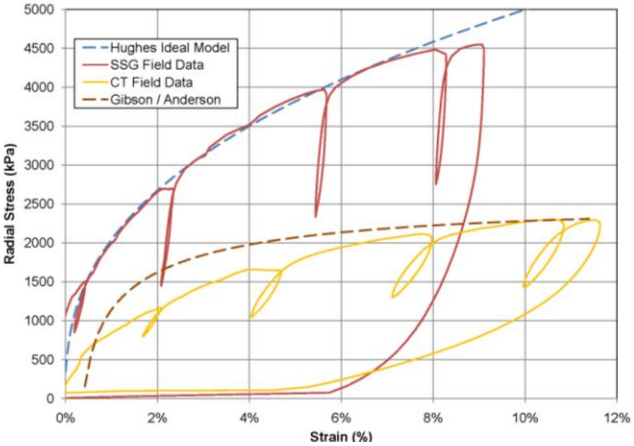

Figure 4: Typical stress-strain curves for glacial till & SSG

The undrained shear strength of the Till was assessed using the Gibson Clay Model (Gibson and Anderson, 1961) fitted to the loading stress-strain curve. Additional undrained shear strength values were assessed by inverting the stress strain curve and applying the Gibson Clay Model to the unload curve and dividing the resultant undrained shear strength by two. The final method of assessing the undrained shear strength was provided by the log method. By plotting the loading curve (less the unload-reload curves) versus the log of the radial strain, the slope of the curve represents the undrained shear strength. Of the three methods, the inverted Gibson method generates the most conservative undrained shear strength while the log method provides in the least conservative results.

The frictional (drained) parameters were assessed using the Hughes Sand Model (Hughes et al., 1977) and fitting the idealized Hughes model to the measured stress-strain curves. In each case, the initial portion of the measured stress-strain curve had to be shifted to adequately model the soil response and to remove any borehole disturbance.

The degree of disturbance was indicated in the real-time display based on the amount of radial strain required prior to lift-off occurring. This was clear on several samples including the final sample where the test pocket was advanced 3.0 m within the SSG instead of the typical 1.5 m. By advancing the borehole ahead, approximately 1 hour elapsed prior to the testing of the last section. This allowed sufficient time for relaxation and borehole convergence to occur and subsequently provided a clear indication of the change in modulus within the sand with time.

Based on the findings of the PBPM borehole drilled at the Station Lands, the range of soil parameters shown in Table 1 have been estimated for the Till and SSG. The minimum and maximum values for each deposit are indicated in the upper and lower cells respectively. Estimates of the elastic modulus were calculated assuming a Poisson's ratio of 0.4 in accordance with most other historical studies in the area. The use of υ=0.4 is based on the assumption that the Till is responding as an undrained material, but due to the degree of saturation, some volume change is permitted. All frictional parameter evaluations were made by using a critical friction angle of 32°. Hughes et al. (1977) indicated that of all of the parameters used in the evaluation of the frictional characteristics, the critical friction angle should be considered to be the most stable for a given material. This holds true especially for a relatively uniform graded, very dense sand similar to the SSG deposit.

Table 1: NLRT PBPM Tests Results

Material Type	E (MPa)	S_u (kPa)	Limit Pressure (kPa)	ϕ' (°)
Till	104 (min)	95	1,500	30
	410 (max)	675	4,500	36
SSG	351 (min)	475	4,700	32
	986 (max)	1100	8,100	37

4 COMPARISON OF LABORATORY AND IN-SITU RESULTS

The results of the historical and current laboratory experiments were compared with the findings of the historical and current in-situ investigations in order to observe any correlation between the two methods. When the results are plotted together, only the overall trend of the material becomes clear allowing for an overall average value to be estimated. However when individually analysed statistically, several observations quickly begin to stand out. In most cases, the undrained shear strength of the laboratory tested samples is higher than those measured in the field and when the deformation modulus is considered, the opposite is true.

4.1 Undrained Shear Strength

The undrained shear strength of the Till relative to the natural moisture content for the historical laboratory experiments is shown in Figure 2. Due to the inability to capture the macrostructure of the Till in a laboratory sample, the effect of fissures and discontinuities such as sand pockets are not included in these data. Because the soil is heavily jointed, strength tests must be able to account for slippage and failure along these preferential planes. As a result, the laboratory testing solely evaluates the microstructure (fabric) of the soil. Based on the stress history of the Till, it is expected that intact samples are extremely stiff and will exhibit high undrained shear strengths. It is only when the overall structure of the soil is considered, can the macro-scale strength of the material be determined. In-situ testing tends to provide slightly lower undrained shear strengths because a fraction of the fissures are included in each test. Fissures are allowed to close and slippage along fractures can occur resulting in lower measured strengths when compared to a fully intact sample.

When the results of the laboratory experiments are statistically compared to those of the current PBPM tests as shown below in Figure 5, the above observations are clearly shown. The average laboratory undrained shear strength was found to range from 15 to 1988 kPa averaging 522 kPa and a standard deviation of 353 kPa. Based on the data shown in Figure 2, the average is likely to be somewhere around 400 kPa but due to the large degree of scatter the arithmetic mean is slightly skewed.

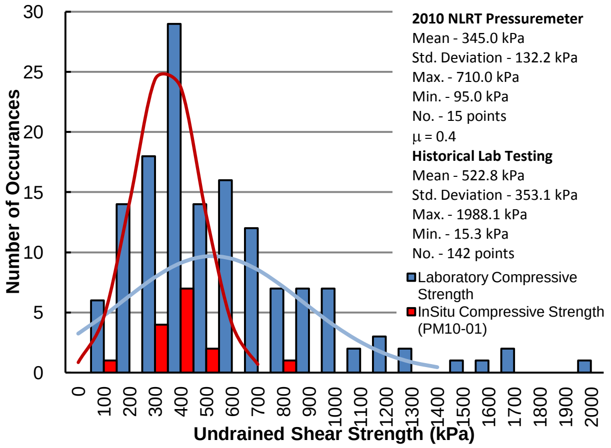

Figure 5: Comparison of glacial till laboratory and in-situ undrained shear strength

4.2 Modulus of Deformation

When the modulus of deformation obtained from the laboratory and in-situ testing is compared, there is a discernable trend for the Till. In the Till, there is a clear trend of values in the range of 200 to 300 MPa with the E value increasing with depth. Interestingly, when the most recent series of PBPM results within the Till are examined, the deformation modulus actually decreases with depth until the contact with the underlying SSG. These findings have not been observed in any of the previous studies, and in fact it has been suggested by Bayrock et al. (1962) that two distinct till sheets exist, an upper brown till and a lower grey till. In most cases, the lower grey till is considered to be stiffer than the overlying formation which is consistent with the findings of the various SCPT tests carried out as part of the current NLRT investigation.

When the results of the various test methods are statistically compared, it is expected due to the influence of stress history and high degree of state dependence of the overconsolidated materials that the deformation modulus obtained in the laboratory will be considerably lower than that of the in-situ methods which is clearly the case. It is also obvious, based on the findings of Morgenstern and Thomson (1971) that the degree of disturbance during sampling plays a significant role in determining an appropriate modulus. In addition, it was anticipated that due to the limitations of the previous in-situ testing done throughout the 70's and 80's that the measured in-situ moduli would be slightly lower than those obtained from the most recent, state of the art test methods. In reality, the previous in-situ tests compared quite well with the current findings indicating that the previously reported values are valid and are likely representative of the conditions at each test location.

A comparison of the deformation modulus measured from the various laboratory and in-situ tests are shown below in Figure 6.

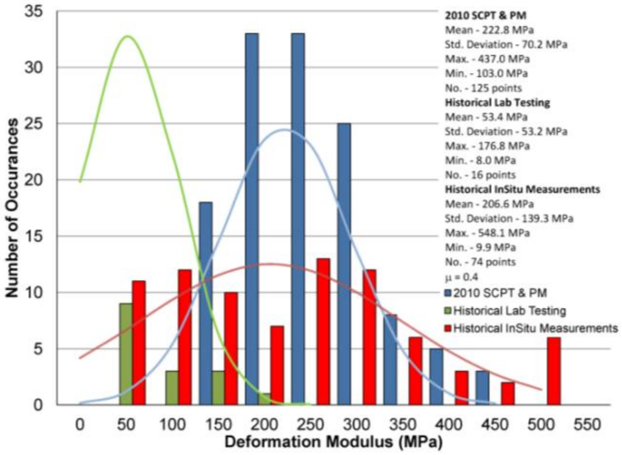

Figure 6: Comparison of glacial till laboratory and in-situ deformation modulus

5 DISCUSSION

Many attempts have been made throughout the last 40 years to provide stress strain profiles for heavily overconsolidated soils similar to those encountered throughout the Edmonton region. The major difficulty with determining the deformation modulus and the shear strength of these materials is the soil's strong state dependence and discontinuous nature. A good deal of reliance is customarily placed on results obtained from triaxial testing on small diameter samples, but more often than not, these samples are not able to capture the overall macrostructure of the formation and instead only provide a glimpse at the microstructure at a specific state of stress. It has been demonstrated that some error associated with sample disturbance and state dependence is removed with the use of consolidated triaxial tests (Ladd, 1964). Bjerrum and Lo (1963) reported that by allowing time for the consolidation process to extend into secondary compression stage for triaxial samples, (approximately one log cycle), that there was an increase in the deformation modulus attributed to re-aging of the material.

When the type of sample recovery method is considered, Lo et al. (1971) report that the deformation moduli measured from triaxial tests carried out on block samples are nearly 4 to 7 times greater than those recorded from tests carried out on samples recovered using Shelby tubes, suggesting a high degree of disturbance from "relatively undisturbed" samples. This finding was not supported by Morgenstern and Thomson (1971) who demonstrated that in samples of heavily overconsolidated soils recovered from around the Edmonton area, Shelby tube samples tend to exhibit higher strength than tests carried out on trimmed block samples. It is thought that the lower shear strengths are associated with a substantial stress release necessary for sampling (excavation to sample location) and the amount of effort required to prepare the samples prior to testing, it is likely that by the time the block sample is mounted and tested, the internal structure is severely damaged and possesses a high number of micro-cracks within the soil matrix.

In order to provide more cost effective test methods and in some cases, compensate for the lack of reliability associated with laboratory testing, in-situ test methods have been developed to better capture the response of soils in a relatively undisturbed state and under stress conditions similar to those expected during construction. Most in-situ test methods however have been developed for soft soil conditions and are not particularly well suited for hard/very dense soils common to most glaciated regions.

Seismic cone penetration tests are effective at providing very small strain, shear moduli, provided that the material can be penetrated by the cone tip and the geophone. Pressuremeters are also very effective for providing shear moduli in terms of engineering strain (Mayne, 2001). In hard/very dense soils, self-bore pressuremeters cannot penetrate the soils and therefore require the use of pre-bored pressuremeters resulting in some stress reduction/relaxation prior to testing.

Initial pressuremeters like the Menard-type probes were developed based on the concept of expanding the borehole by pressurizing the walls with a known volume of fluid. This type of pressuremeter is still widely used today, but is limited by the fact that the data is collected manually and ultimately the number of data points defining the stress-strain curves is limited. Wroth and Hughes (1973) developed a pressuremeter that was considerably more automated and ultimately defines the stress strain curves using several hundred points which are recorded by a data logger and displayed at the ground surface in real time. The Camkometer developed at Cambridge University in the 1970's uses spring loaded strain gauges "feelers" that are in contact with the membrane at three locations positioned at 120° to one another. The strain gauges measure the radial displacement of the pressuremeter probe under a known applied pressure. Expansion of the borehole is provided by means of a regulated nitrogen gas source at the ground surface and the applied pressure is measured using a pressure sensor positioned within the probe.

Clearly determining the deformation modulus and undrained shear strength of heavily overconsolidated materials poses a real challenge and researchers and designers have attempted to understand the key factors associated with measuring both parameters as accurately as possible since the start of geomechanics. The investigations carried out within the hard glacial till common to the City of Edmonton indicate that measurement of the deformation modulus and undrained shear strength is relatively consistent when using insitu testing methods when compared to the laboratory results.

6 CONCLUSIONS

A series of laboratory and in-situ investigations have been carried out as part of the City of Edmonton's Light Rail Transit expansion, in order to quantify the ground characteristics. The results from these investigations were compared to previously conducted laboratory work and in-situ tests. The results of the current investigation indicate that the undrained shear strength measured using a pressuremeter in pre-bored borehole was slightly less than that measured in a number of previously carried out triaxial tests. As expected, when the undrained shear strengths of the soils are examined, it appears that the laboratory tests tend to overestimate the strength, likely due to the inability of the test methods to capture the macrostructure of the soil mass. Instead, triaxial tests document the performance of small scale, intact samples and ultimately provide a glimpse into strength of the soil's microstructure or fabric.

When the modulus of deformation is considered, the moduli measured in the laboratory is considerably less than those observed in the field due either to sample disturbance during recovery and preparation and/or due to the high stress relaxation of the heavily overconsolidated materials. Interestingly, the results of the previous in-situ investigations that lacked sophisticated computer systems and dedicated data logging equipment compared quite well with the findings of the recent, state of the art field investigations.

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