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Estimating Soil Properties Using Deformations Associated with Deep Excavations

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ABSTRACT
Deformations were monitored for a recently completed deep tied-back excavation in the City of Edmonton. A 2D finite element model was used to back analyse the soil properties that provided the best agreement with the measured deformations. These values were then compared with the findings of previous laboratory, in-situ and back calculated parameters. These best-fit parameters will be used to estimate the surface settlement expected for the tunnels associated with the proposed extension of the City of Edmonton's North LRT.

RÉSUMÉ
Les déformations d'une excavation profonde à murs de soutènement ancrés récemment complétée à Edmonton ont été mesurées. Un modèle bidimensionnel d'analyse par éléments finis a été utilisé pour faire une rétro-analyse des propriétés du sol qui s'accordent le mieux avec les déformations mesurées. Ces valeurs ont ensuite été comparées avec les paramètres trouvés auparavant en laboratoire, sur le site et par calcul à rebours. Les paramètres qui s'approchent le mieux des données mesurées seront utilisés pour estimer les tassements de surface attendus avec l'extension nord proposée du train léger (LRT) de la ville d'Edmonton.

1 INTRODUCTION

Since the late 1970's the City of Edmonton has extended its Light Rail Transit (LRT) system to provide efficient and environmentally friendly transportation to the growing population. Currently, an extension of the LRT is being designed to provide access from the existing downtown Churchill Station to the Northern Alberta Institute of Technology (NAIT). The proposed extension includes construction of approximately 750 m of twin-tunnels (Figure 1). A portion of these tunnels has been incorporated into the foundation of the Epcor tower currently under construction.

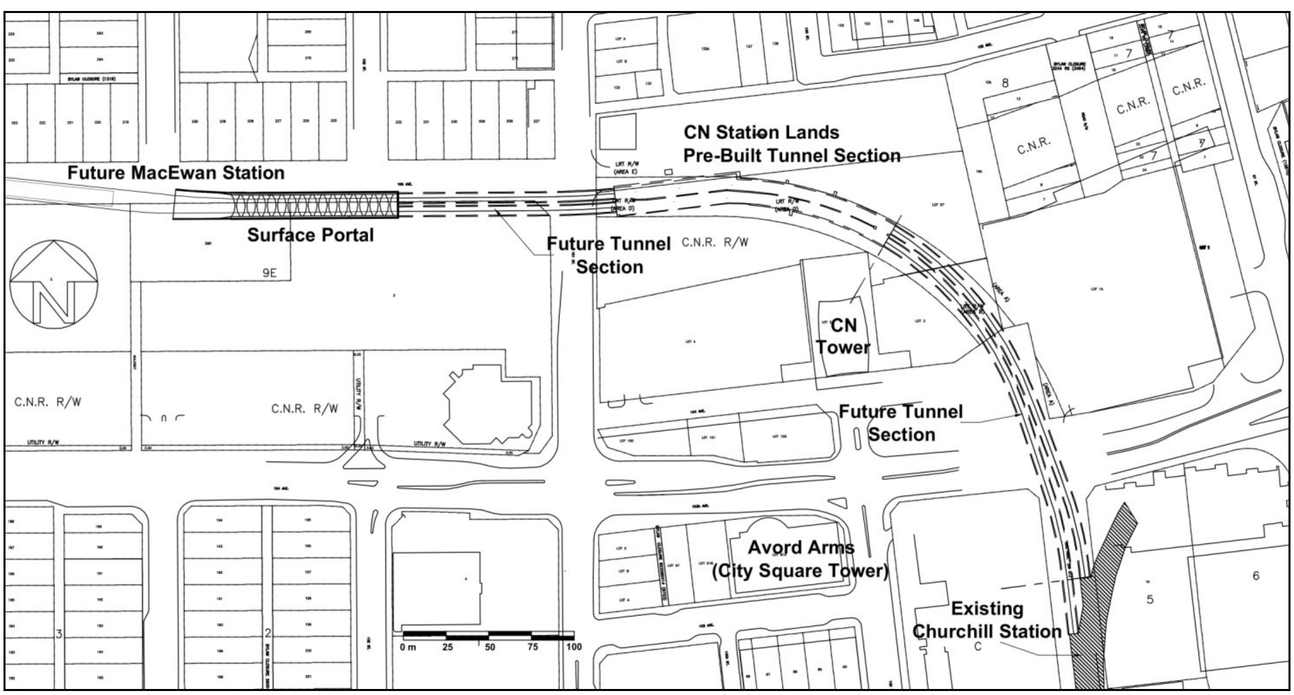

Figure 1: Proposed Edmonton North LRT Extension Tunnel from Churchill Station to MacEwan Station

Site investigations using traditional auger drilling and sampling, as well as CPTs were carried out to evaluate the soil conditions along the tunnel alignment. The properties of the soils and historic experience from previous LRT projects were used to establish the soil parameters for the preliminary design of the tunnels. The excavation for the Epcor building foundation also exposed the sequence of soils that will be encountered by the tunnel and provided an opportunity to document the performance of these soils in exposed temporary excavations.

The Epcor foundation excavation required the construction of a deep tied-back retaining system (Figure 2). Two slope inclinometers were used to monitor the performance of the retaining system. The deformation of this retaining system was also used to constrain the in-situ soil properties. A series of two dimensional finite element analyses were carried out to match the measured deformations. This paper summarizes the results from these analyses and compares the soil properties used in the finite element analyses with the values historically associated with these soils.

2 GEOTECHNCIAL SETTING

2.1 Soil stratigraphy

The site investigation for the North LRT alignment (Soliman and McRoberts, 2009) from Churchill Station to the portal at Station 600+464 revealed the following stratigraphic units:

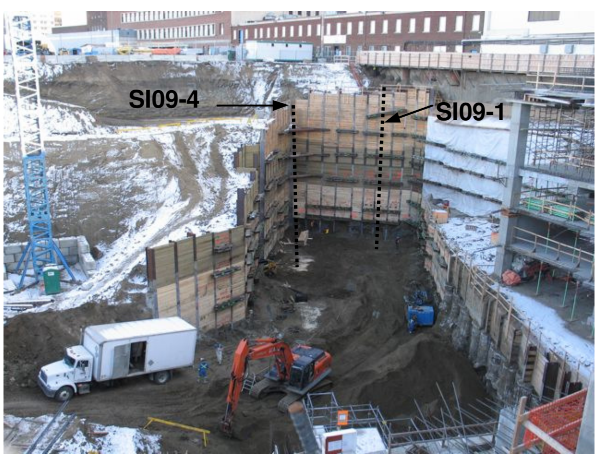

Figure 2: Location of the Slope Inclinometers used to monitor the Soldier-Pile wall and used in the back analyses

Fill: Fill consisting of reworked native soils including clay, glacial till, sand and silt materials including fragments of construction debris (brick, concrete, asphaltic concrete). The Fill material is generally compact to dense with an average thickness of about 3 metres.

Lacustrine Clay: The Fill is underlain by a glaciolacustrine silt and clay known as the Lake Edmonton Clay (LEC) and is typically about 5 metres thick. The silt and clay is generally firm to stiff.

Glacial Till: The Glacial Till (GT) deposit is approximately 15 to 16 metres thick and generally very stiff and dense. Zones of Intra-Till Sand with varying fines content are randomly encountered within the Glacial Till. Cobbles and boulders are also present within the soil matrix. The interface between the GT and the underlying sand and gravel formation was very sharp and planar and dipped slightly towards the West. It is clear in Figure 3 that there are three distinct joint sets within the GT, two vertical and one horizontal. When excavated by a backhoe, the GT would fail preferentially along these joint sets forming blocks as shown in Figure 4.

Saskatchewan Sand and Gravel: Below the GT, an approximately 10 to 11 metre thick layer of pre-glacial sand and gravel (SSG) deposits is encountered. These sediments are generally compact to very dense and are characterized by relatively uniform gradations. The silt content for these granular materials varies from 1% to 99%, with an average of 36%, therefore limiting the number of reliable options for ground conditioning. As shown in Figure 3, the SSG is capable of standing vertical following excavation and in some cases it was left as a vertical cut for periods greater than several months.

Bedrock: Edmonton Formation Bedrock consisting of soft sedimentary deposits of claystone, sandstone and siltstone with varying thickness is encountered below the SSG deposits.

Groundwater: Perched groundwater was found in the Intra-till sand lenses. No significant seepage was encountered during the excavation of the Epcor foundations. Groundwater occurs within the SSG formation well below the tunnel invert and perched above the surface of the shale bedrock.

Figure 3: Interface between the GT and SSG at the Station Lands. Notice the vertical and horizontal joints in the till. These joint surfaces are iron stained.

Figure 4: Blocky nature of glacial till when excavated with a backhoe

2.2 Laboratory and in-situ strength

The proposed alignment for the North LRT between Churchill Station and the Epcor building will be driven in two major stratigraphic units: (1) hard Glacial Till and (2) pre-glacial Saskatchewan Sands and Gravel (fine to medium grained dense sand). DeJong and Harris, 1971 presented data related to a series of triaxial tests carried out on block samples of the GT recovered from within the CN Tower excavation in the mid 1960's. The strength parameters estimated from these laboratory tests were reported to be ϕ'=24° and c'=14 kPa.

Medeiros, 1979 carried out a series of triaxial tests on both the GT and the SSG materials recovered from boreholes drilled approximately 300 and 650 m south and southwest of the station lands. Block samples were recovered from within the excavations for the Churchill and Central LRT Stations during their construction. The samples were preserved in the field and hand trimmed prior to testing. In each case, the samples were probed using one of four stress paths as well as in several plane strain experiments. Based on the various laboratory tests, the Mohr-Coulomb failure envelope for the samples of the GT was estimated to be ϕ'=40° and c'=0 kPa while for the plane strain condition tests ϕ'=46.5° and c'=0 kPa. In every sample tested except one, the material behaved as an overconsolidated material with respect to volume change. However, when the stress strain profiles were examined, every sample regardless of the degree of overconsolidation responded as though it were a strain hardening material.

The SSG was recovered by Medeiros (1979) using block samples cut from the open excavations of the abovementioned LRT stations. Because the SSG was not fully saturated, the samples could not be frozen and there was significant difficulty preparing them for testing. As a result, only triaxial tests could be carried out on the relatively undisturbed SSG samples. As with the GT, the SSG samples were subjected to four different triaxial stress paths.

When the stress-strain plots were examined, it was clear that like the GT, the SSG material responded as a strain hardening material regardless of the stress path chosen. The change in volume however displayed the characteristics of an overconsolidated material by first contracting and dilating with increased shear strain. Medeiros found that the stress-strain relationships were highly dependent on the stress path taken for the sand samples. It was shown that during triaxial extension, the shear strength of the soil was mobilized much sooner than those samples tested in triaxial compression. .

Whittebolle, 1983 also carried out a series of high quality triaxial tests on relatively undisturbed samples of the GT recovered from below the floor of a structure located approximately 1100 m southwest of the Epcor site. The samples were cored using an air rotary system specifically designed for recovery of the till material. Due to the blocky nature of the soil, the core barrel had to have a diameter of 100 mm or less to minimize the risk that the samples may pass through joints within the till. This resulted in fairly small triaxial samples when prepared for testing. Both unconsolidated undrained (UU) and consolidated drained (CD) triaxial tests were carried out on the recovered samples.

The results of the triaxial testing by Whittebolle (open cit.) indicated that there was a significant degree of scatter associated with the laboratory testing. The Mohr-Coulomb strength parameters of the GT were measured to range from ϕ'=31.5 to 33.5° and c'=15 to 25 kPa. Whittebolle (open cit.) found that the GT tended to exhibit two angles of friction depending on the confining stress applied to the sample. At lower stress levels, the effective friction angle was slightly higher due to the dilative nature of the material. Beyond a confining stress of approximately 100 kPa the friction angle reduced until a stress of around 400 kPa was reached, at which time the angle became relatively stable.

Morgenstern and Thomson, 1971, showed that the method of sampling played a significant role in terms of estimation of the ultimate yield strength and the appropriate failure envelope. Their experiments indicated that the undrained shear strength of samples recovered from cut block samples, which are typically thought to provide the most reliable data, actually resulted in some of the lowest values measured in their investigation. They found that when using a drill tool designed specifically to core soils into a Shelby tube (pitcher sampler), the strength of the soil tended to be higher than all other methods used. Additional UC tests carried out by AMEC, 2009 for the design of the NLRT tunnel have been added to the original model for comparison. The results of the strength testing carried out by Morgenstern and Thomson, 1971 and Soliman and McRoberts, 2009 are shown below in Figure 5.

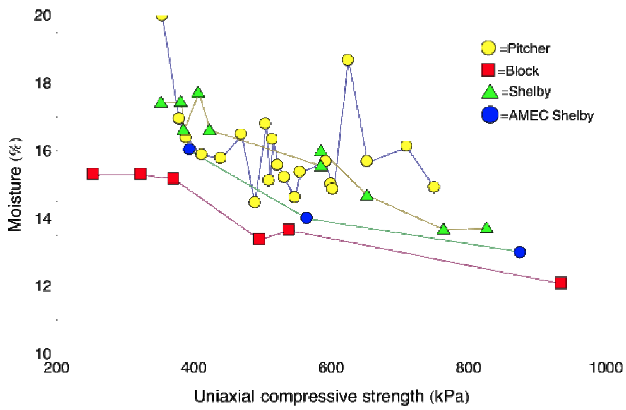

Figure 5: Comparison of Undisturbed Samples Unconfined Compressive Strength

Recently a series of static CPTu tests were carried out along the proposed LRT extension alignment. The shear strength calculated from the measured tip resistance varied considerably from testhole to testhole. Based on the measured tip resistances, the estimated N60 in the GT ranged from around 20 to greater than 50 blows/300 mm of penetration. In areas of the till with reduced skin friction, the calculated undrained shear strength varied from around 250 to greater than 2,000 kPa, though typically the estimated undrained shear strength is around 500 kPa. In CPTu holes 09-01 and 09-02, sections of the GT had to be bored out prior to readvancement of the piezocone indicating effective refusal. The bored sections within the GT occurred between elevations 654 to 658 m.

For the SSG the measured tip resistance varied from 10 to greater than 80 MPa, though was typically around 50 MPa. The calculated N60 values ranged from 15 to greater than 50 blows/300 mm of penetration. Isolated undrained shear strength values were measured within the SSG and were found to be approximately 500 to 1,750 kPa. As with the GT, the SSG material had to be bored out in CPT09-01 and 09-02 once refusal had been achieved. Refusal occurred between elevations 643 to 648 m. This range of elevations corresponds with the proposed invert of the LRT extension. Refusal also occurred in CPT09-01 at elevation 640 to 642 m. The results of CPT09-02 and the stratigraphy identified in the nearest borehole (AMEC BH09-14) are shown in Figure 6.

2.3 Laboratory and in-situ Deformation Modulus

DeJong and Harris (open cit.) estimated the deformation modulus from oedometer tests carried out on the recovered block samples of GT. The deformation modulus calculated from a total of six consolidation tests carried out on the recovered block samples was reported to range from 10 to 21 MPa. They also reported that for three unconsolidated undrained (UU) triaxial tests, the tangent modulus of elasticity ranged from 8 to 10 MPa, while for two consolidated undrained (CU) triaxial tests, the modulus of elasticity was measured as 82 and 96 MPa.

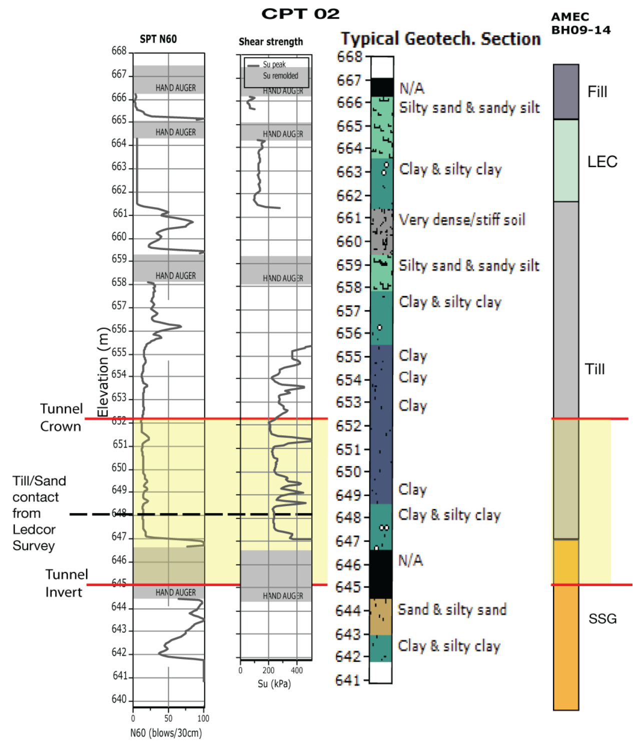

Figure 6: CPT09-02 and Borehole 09-14 Profile

Medeiros (open cit.) reported the elastic modulus from conventional triaxial tests as well as from cyclic triaxial tests. He found that the modulus of elasticity of the GT was highly dependent on the confining stress and therefore it could not be well defined from the conventional triaxial tests. However, Medeiros did find that when subjected to cyclic loading, the modulus was not dependent on the confining stress and was therefore independent of the stress path. Based on three cyclic triaxial tests carried out on the GT, the modulus of elasticity was found to range from 142 to 164 MPa.

In addition to the laboratory testing, DeJong and Harris also carried out a full scale monitoring program in order to back calculate the deformation modulus of the GT based on real settlement data. These experiments involved monitoring the settlements that occurred during the construction and following completion of the CN Tower adjacent to the proposed alignment (see Figure 1). They found that the settlements were approximately 1 mm per floor of structure when founded in the GT. Based on the measured settlements of the structure and the assumed combination of dead and live loads, the back calculated deformation modulus was found to vary with respect to the applied load in excess of the overburden pressure as shown below in Figure 7. The derived moduli from the CN Tower tended to stabilize as the applied load became relatively high. From this stabilization, DeJong and Harris estimated the deformation modulus from the full scale construction experiments to be around 490 MPa for the GT.

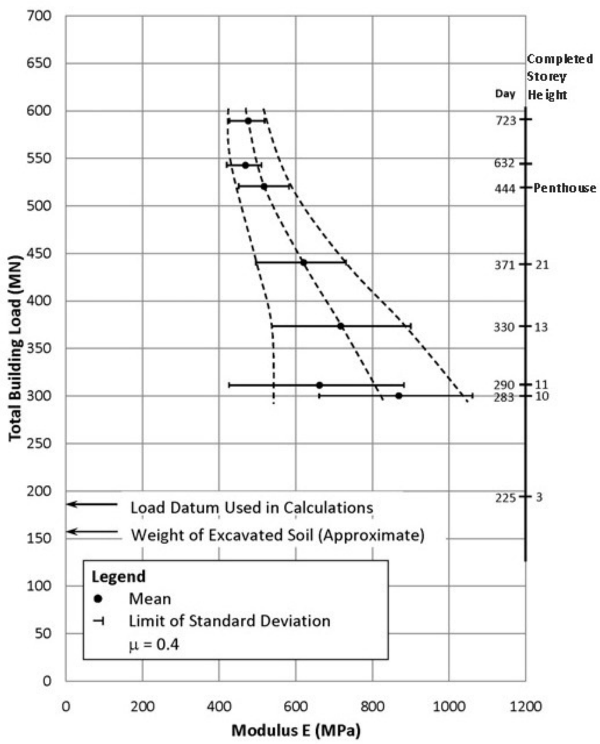

Figure 7: Derived deformation modulus from CN Tower settlement monitoring data (DeJong and Harris, 1971)

DeJong and Harris as well as Medeiros suggest that the deformations measured during full scale excavations within the SSG should be considerably less than those estimated by using the results of triaxial tests.

In-situ pressuremeter testing was carried out following completion of the CN Tower by Eisenstein and Morrison, 1973 as part of a study to establish the likely deformation modulus of the GT and the SSG formations. Eisenstein and Morrison (open cit.) recorded the deformation modulus within the GT and the SSG formations adjacent to the completed CN Tower foundation as well as a site approximately 600 m south of the site at the now Telus building. They found that in the eight tests within the till and SSG layers the deformation modulus ranges from around 125 to 240 MPa but is typically around 200 MPa. They did not find that the variation between the two formations was significant when compared to the test depth with the exception of the three tests carried out near the surface of the till layer. In these three upper tests, the deformation modulus ranges from around 55 to 110 MPa but is typically around 70 MPa.

Additional in-situ testing was recently carried out as part of the LRT tunnel design and included the advancement of two cone penetration tests (CPTu) with the small strain shear modulus measured by analysing the shear wave velocities at depth. Adjustment of the small strain shear modulus was made by dividing the calculated dynamic elastic modulus by a factor of 3 as suggested by Mayne (2001). This correction does not address the fact that the measured shear modulus is dynamic however it provides an approximate method for estimating the elastic modulus in terms of the strains that are expected for large excavations.

Upon examination of the results of the estimated elastic moduli calculated from the dynamic CPTu data it was seen that within the LEC formation, the modulus ranges from 40 to 100 MPa though is typically around 45 MPa. The modulus of the GT range from 110 to 350 MPa with a mean of 250 MPa, and the modulus for the SSG ranged from 130 to 500 MPa with a mean of 250 MPa. The results from the various in-situ modulus testing are summarized below in Table 3.

A statistical analysis of the measured modulus values results in a mean of 197 MPa and 307 for the GT and SSG, respectively. A histogram showing the frequency of measurement of both the laboratory and in-situ moduli for both the GT and SSG is shown in Figure 8.

Table 3: Young's Modulus (MPa) Measured in In-Situ Tests, after Eisenstein & Morrison, 1972 and AMEC, 2009

Material Type	Pressuremeter		CPTu	
	CN1	AGT2	CPT09-01	CPT09-02
LEC	NA	NA	40 (min) 100 (max)	40 (min) 100 (max)
Upper GT	55 (min) 110 (max)	62	NA	NA
GT	125 (min) 140 (max)	NA	195 (min) 350 (max)	110 (min) 300 (max)
SSG	135 (min) 240 (max)	NA	130 (min) 500 (max)	170 (min) 390 (max)

2.4 Summary of Results

When the results of the various modulus calculation methods are compared as in Figure 9, it appears that the information obtained from the in-situ pressuremeter testing gives the most consistent results with the field results of DeJong and Harris, providing a reasonable upper bound to the likely in-situ modulus.

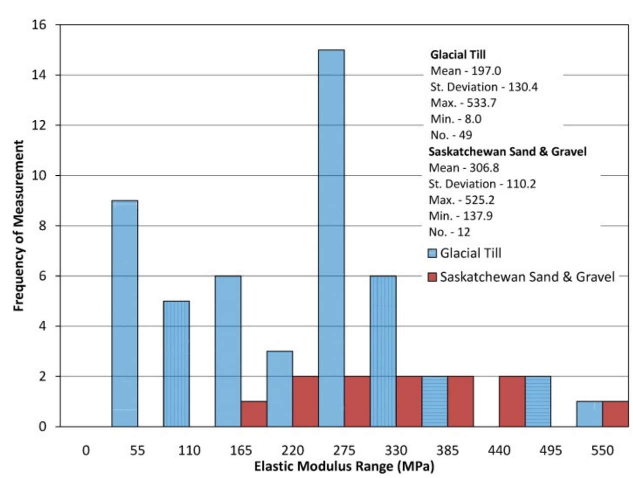

Figure 8: Frequency of Measured Elastic Moduli for GT and SSG Formations

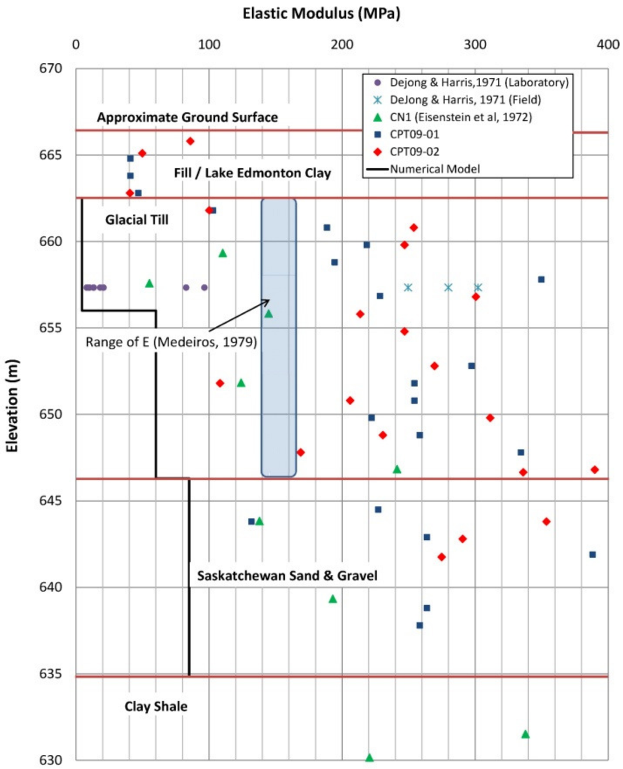

Figure 9: Comparison of deformation modulus from various in-situ and laboratory scale tests as well as back analysis

The results of the various laboratory testing did not provide very reliable estimates of the modulus. Conversely, with the exception of the lower range of strength parameters provided by DeJong and Harris, the Mohr-Coulomb strength parameters were relatively consistent for the majority of the laboratory tests. The lab testing did reveal that the smaller the scale of the experiment, the lower the predicted modulus. This was indicated by the very low end values reported by the oedometer tests carried out on recovered block samples.

3 BACK ANALYSIS OF RETENTION SYSTEM

Numerical modeling of the existing North LRT excavation and support systems were carried out to estimate the in-situ soil parameters near the eastern leg of the North LRT tunnels. The numerical model considered the existing retaining structure installed along the northern section of the current excavation. The lateral displacement results obtained from the numerical model were then compared to the monitoring data of the slope inclinometer installed during the construction of the soldier pile and tieback support system. The location of the inclinometers relative to the support system is shown in Figure 2.

The numerical model was constructed using the commercially available, two-dimensional finite element analysis program known as Phase^2 supplied by RocScience Inc. The program is designed specifically for modelling of various excavations within geological profiles. The model calculated the stresses and displacements for the profile considering the preselected boundary conditions and soil profile parameters. An image of the North Wall support system model is shown in Figure 10. The displacements calculated for the retaining wall were compared to the measurements recorded in the slope inclinometer SI09-4 installed within the north wall. The inclinometer is attached to the steel reinforcement placed within the bored piles as part of the existing soldier-pile wall retaining system. The soldier piles consist of structural I-beams (W530x74) and concrete bored piles measuring 750 mm in diameter spaced at 2 m on centre.

The model was divided into 17 stages to represent the various construction phases of the excavation. The geostatic stresses and the assumed field stresses were applied prior to commencement of the construction activities. Geostatic stress were used to confirm that the model was performing as designed by allowing the calculation of the vertical and horizontal stresses at the interface of each strata using the known thickness, unit weight and the horizontal stress ratio of each layer within the model. Following the application of gravity, the soldier pile wall was installed to the as-built elevations. Excavation then proceeded at 1.5 m intervals in each stage thereafter. When the elevation of a tieback was encountered in a stage, the excavation was extended an additional 1.5 m below the tieback elevation and the support element was installed. Each tieback was installed to lengths indicated by the design drawings provided to AECOM and pre-tensioned to 80% of the design working load.

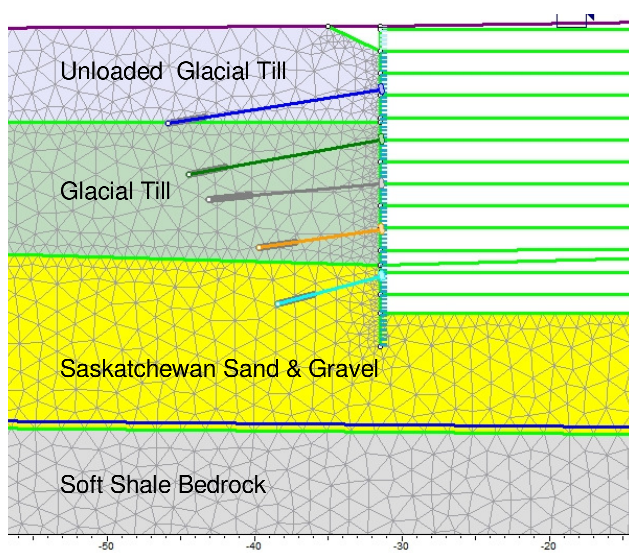

Figure 10: North support system model in Phase^2

The results of the numerical model were viewed in a post processor program included with Phase^2. The displacements were calculated at each element located along the length of the soldier pile wall. The total lateral movement of the wall was then subtracted from the initial lateral displacements calculated during the application of gravity to obtain the movements resulting from the construction activities alone. These displacements were then compared to the measured SI data. Figure 11 shows the measured SI displacements compared to those obtained from the numerical model using the soil parameters in Table 4.

A match to the measured displacements in the upper unrestrained section of the wall could not be made using uniform properties for the GT. In each model that was run, the calculated displacements at the top of the excavation were considerably less than those measured in the inclinometer. To provide reasonable agreement with the measured displacement, the modulus of upper zone of the GT was reduced. The modulus for the upper portion of the GT labelled "Unloaded Clay Till" is not considered a material property but a property that reflects the contractors' construction technique and the rebound that occurred prior to starting the shoring wall construction. Once these considerations had been employed in the numerical model, the calculated displacements more closely matched those measured during the construction process.

Table 4: Soil Parameters for Retaining System Back Analysis

Material Name	Unit Weight (kN/m^3)	Young's Modulus (MPa)	Poisson's Ratio	Peak Parameters Tensile Strength (kPa)	c' (kPa)	ϕ' (°)	In Plane K_o
Unloaded GT	21.0	4.5	0.35	0	5	30	0.85
GT	21.0	60	0.35	0	50	40	0.95
SSG	20.0	85	0.35	0	10	35	1.0
Soft Shale	23.0	400	0.3	0	100	25	1.8
Stiff Shale	23.0	700	0.3	0	400	25	1.8

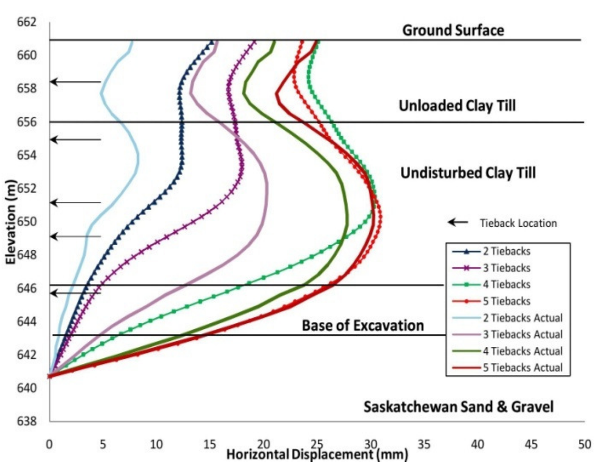

Figure 11: Comparison of the SI 09-4 results with those from the numerical model for the North Support System model. (Model displacements are the lines without symbols)

When the results of the model are compared with the monitoring results for SI09-4, a maximum displacement of 31 mm is estimated to occur at Elev. 650.3 m while the maximum measured displacement as of January 26, 2010 was approximately 30 mm at Elev. 650.4. The results of the numerical model also indicate that the soil response in the till is generally elastic in nature. None of the elements in the modelled had yielded either in tension or shear. Hence the only soil parameters that control the displacements in the Till are Young's Modulus, Poisson's ratio and K_o. There is minor yielding in the sand but given the very high in-situ strength of this deposit the 35 degree friction angle is considered too loo.

4 DISCUSSION OF RESULTS

The construction sequence for the Epcor building results in an open cut. This cut was exposed for approximately 2 years before the construction of the tied-back soldier-pile wall for the NLRT. The results of the back analysis of the retaining system for the NLRT excavation indicate a lower range of the possible moduli of elasticity; this is likely due to the degree of disturbance that had occurred on the site prior to installation of the support structure. As part of the site preparation, the overlying Fill and LEC deposits were removed before the installation of the soldier pile wall. It is assumed that this resulted in a significant degree of unloading and softening of the underlying GT. Based on the deflections measured in the inclinometer, the softened till zone was estimated to extend roughly 6 m below ground surface. Even with the softened modulus in the upper 6 m of the wall, the observed shape still was not captured as well as the underlying lower till and SSG formations.

As with the findings of DeJong and Harris, the numerical model indicated that the majority of the displacements observed in the support structure were elastic in nature. Local yielding in the model occurred near the top of the wall where the larger lateral displacements resulted in tensile failures of the softened GT. Additional yielding was observed at the base of the excavation in the SSG. There was no evidence of yielding in the SSG at the base of the excavation and it is likely that the shear strength of this very dense and stiff sand is underestimated in the model.

Eisenstein and Thomson (1978) indicate that the majority of displacements measured within one of the first LRT tunnels advanced in the City of Edmonton, occurred almost immediately following installation of the rib and lagging support. At the test locations, monitoring was continued for a total of 36 days following completion of a section, but it is reported that the majority of the displacements occurred within the first 3 days of completion.

5 CONCLUSIONS

A brief summary of previous laboratory, in-situ and full scale displacement monitoring programs previously carried out in the City of Edmonton has been provided.

A review of the results shows there is an inconsistency between the moduli measured in the laboratory and those observed in larger full scale excavations. The moduli of elasticity estimated from the smaller scale laboratory experiments tend to overestimate the expected deformations when compared to the actual measured values.

At present there are no methods to associate the measured moduli with the expected full scale deformation modulus. It would appear that in-situ testing that results in strains of a similar order of magnitude tend to provide a reasonably reliable range of moduli while in-situ tests such as the seismic CPTu may not provide consistent results with depth through the depth of the formation.

The lateral displacements associated with a deep excavation were used assess large scale in-situ soil properties. A two dimensional numerical model was used to back analyze the measured displacements. The back analysis indicated that ground behaviour is essentially elastic, and the likely moduli of the native materials are somewhat lower than those reported by others for in-situ (pressuremeter and dynamic CPTu) as well as for actual displacement monitoring data. The modulus of elasticity measured during a series of triaxial tests was considerably lower than all of the in-situ or full scale estimates conducted throughout the years. It is thought that the lower values of the elastic modulus in the numerical model are likely due to the high degree of unloading that had occurred at the site prior to the commencement of the displacement monitoring program. Additional laboratory experiments and an in-situ testing program have been implemented to reconcile these differences.

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