Highway 66 Embankment on Soft Ground – Design, Construction and Long-Term Monitoring: A Case Study

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ABSTRACT
Highway embankment construction in Northern Ontario often involves the placement of fill over soft ground comprised of thick cohesive deposits. The embankments must satisfy relatively stringent post-construction settlement criteria. Consequently, the understanding of soil compressibility properties is critical in the design of such embankments to ensure long-term performance.

A case study of an approximately 3.5 m high embankment associated with the realignment of Highway 66 near Virginiatown, Ontario is examined herein. The realigned Highway 66 was constructed under Ministry of Transportation of Ontario (MTO) Contract 2015-5121 and was opened to motorists in the Fall of 2017. The design of the embankment involved sub-excavation of peat, installation of prefabricated geosynthetic (wick) drains through an up to about 17 m thick deposit of clayey silt to clay, and staged embankment construction with surcharging to achieve the settlement performance criterion of the highway. The settlement and pore water pressure of the cohesive deposits are being monitored and indicate that the embankment settled in excess of 2 m during construction. Cone penetration testing (CPT), in-situ field vane tests and soil sampling within/near the footprint of the embankment were also carried out near the end of construction to allow for a comparison between compression parameters (primary and secondary) estimated from standard testing of the clay during the design phase of the project and higher complexity testing completed post-construction. The data from the on-going field instrumentation monitoring program and laboratory long-term consolidation testing to assess secondary compression (creep) is also discussed as an area of further research.

RÉSUMÉ
La construction de remblais pour les routes du nord de l'Ontario implique souvent la mise en place de remblais sur un sol meuble constitué de dépôts cohérents épais. Ces remblais doivent satisfaire des critères de tassement rigoureux après la construction. Par conséquent, une compréhension des propriétés de compressibilité du sol est essentielle dans la conception de tels remblais afin d'assurer leur performance à long terme.

Une étude de cas d'un remblai d'une épaisseur d'environ 3,5 m associé au réalignement de la route 66 près de Virginiatown, en Ontario, fait l'objet du présent article. La route 66 a été réaménagée en vertu du contrat 5121 du ministère des Transports de l'Ontario (MTO) et a été ouverte aux automobilistes à l'automne 2017. La conception du remblai impliquait l'excavation de la tourbe et l'installation de drains verticaux préfabriqués géo-synthétiques (wick drains) jusqu'à une profondeur d'environ 17 m dans une couche d'argile et de silt argileux, ainsi que la mise en place par étape du remblai avec surcharge afin d'atteindre le critère de tassement de l'autoroute. Le tassement et la pression d'eau interstitielle des dépôts cohérents ont été surveillés et indiquent un tassement de plus de 2 m pendant la construction. Des essais de pénétration au cône (CPT), des essais au scissomètre de chantier, et des prélèvements de sol à l'intérieur du remblai ont également été effectués vers la fin de la construction. Ceci a permis une comparaison entre les paramètres de compression (primaire et secondaire) de l'argile utilisés pendant la phase de conception aux paramètres obtenus au moyen de tests plus complexes effectués après la construction. La collecte des données du programme de surveillance des instruments sur le terrain, ainsi que les essais de consolidation à long terme en laboratoire en cours sont également discutés pour évaluer la compression secondaire (fluage).

1 INTRODUCTION

King's Highway 66 is a two-lane Trans-Canada Highway in Northeastern Ontario connecting Ontario and Quebec. A 3.4 km long realignment of Highway 66 from approximately 11.0 km east of the junction of Highway 66 and Highway 624 easterly was constructed due to the risk of surface subsidence associated with the abandoned Kerr-Chesterville underground mine located beneath the footprint of the existing highway alignment.

Low-lying swamplands with standing water are commonly encountered during construction of MTO highway embankments in Northeastern Ontario. The subsurface conditions encountered at these locations typically consist of peat underlain by thick deposits of clay. The clay deposits in this area, which were deposited

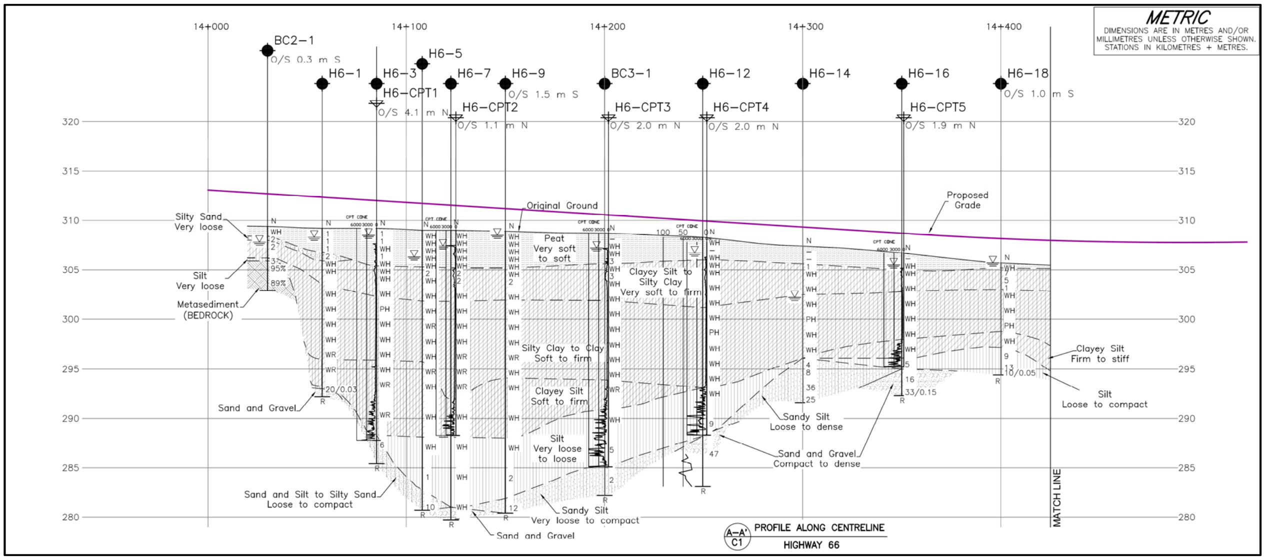

Figure 1. Interpreted soil stratigraphy at Swamp Crossing H6/H7

seasonally along the lakebed of the prehistoric Lake Ojibway, are generally comprised of stratified sediments with varying degrees of plasticity (i.e., varved/layered deposits with silt to clay laminae).

The preferred highway realignment required fill placement over several such swampland crossings to construct the highway embankment. In particular, Swamp Crossing H6/H7 discussed herein, was approximately 500 m in length and encountered up to about a 4 m thick deposit of peat underlain by an approximately 17 m thick varved cohesive deposit as shown on Figure 1.

A geotechnical subsurface investigation was carried out to complete detailed design of the highway embankment for the new two-lane highway over this swamp crossing. Subsequent to the detailed design, the Highway 66 realignment was completed in the Fall of 2017 under MTO Contract 2015-5121 and has since been opened to traffic.

The design and construction of MTO highway embankments are required to satisfy embankment performance requirements. The post-construction settlement criterion selected for this class of highway is 200 mm over its 20-year design life. In order to comply with this performance requirement, various settlement mitigation measures are often considered. At Swamp Crossing H6/H7, several embankment design alternatives were compared based on advantages, disadvantages, costs, risks and consequences. The selected preferred option involved sub-excavation of the peat deposit; installation of wick drains through the cohesive deposit; staged construction; and surcharging.

A geotechnical instrumentation and monitoring program was developed as an integral component of the detailed embankment design to monitor the magnitude and time-rate of settlement during construction. For this project, staged construction with waiting/delay periods were specified in the Contract and excess pore water pressure generation in response to fill placement and dissipation required close monitoring to avoid embankment slope failure during construction. Significant magnitudes of settlement, on the order of 1.5 m, were predicted at some sections of the swamp crossing where fill heights of up to 3.5 m above the original grade were required. Monitoring of the settlements and excess pore water pressures during the surcharge period were critical in confirming that the estimated settlements were progressing at a rate which would ensure that post-construction settlements were within MTO's performance criteria and would not cause delays to the overall Contract schedule.

Following the successful completion of the embankment, the MTO has initiated a post-construction investigation program including field investigation, geotechnical laboratory testing and a long-term instrumentation and monitoring program in order to further evaluate the short-term and long-term settlement behaviour. The purpose of the post-construction program is to re-evaluate the predicted primary consolidation settlement and to further evaluate the secondary compression settlement. The data collected will be analyzed and utilized to: i) provide insight into the prediction of time-dependent settlement, which has significant schedule implications, and; ii) to assess the long-term behaviour of the foundation soils.

2 EMBANKMENT DESIGN

The embankment design associated with the wick drain foundation treatment area involved a detailed stability and a settlement assessment for each stage of construction at critical sections along Swamp Crossing H6/H7.

2.1 Stability

Stability analyses, combining both total stress and effective stress conditions, were performed using the limit equilibrium method. The stability analyses were performed to check that the geometry and proposed rate of construction satisfied the target minimum Factor of Safety of 1.3 at each stage of embankment construction.

The stability analyses were carried out assuming a 1.25H:1V side slope profile for the rock fill embankment and a 2H:1V side slope profile for the granular surcharge load placed on top of the rock fill embankment.

Given the presence of thick and soft cohesive deposits and stability issues associated with the proposed embankment geometry and rate of construction, 1.5 m high by 5 m wide stability berms (dimensions governed by the limited right-of-way) at the toes of the embankment were included in the design to maintain stability.

The impact of excess pore water pressure development on the stability of the embankments at each stage of construction was assessed as part of the effective stress analyses. The analyses involved the generation of a two-dimensional field of total pore water pressures throughout the foundation soil deposits at critical time periods. In addition, different rates of construction (i.e., delay periods between construction stages) were also considered in the analyses to evaluate the effect of excess pore water pressure dissipation on stability.

2.2 Settlement

The sources of settlement in the analyses considered: i) immediate settlement of the granular drainage blanket and of the replacement fill materials in sub-excavation areas; ii) immediate settlement of the native granular soils; iii) primary time-dependent consolidation of the cohesive deposits, and; iv) secondary time-dependent (creep) compression of the cohesive deposits. The self-weight compression of the embankment rock fill materials was also considered, but not included in the criterion used to assess the required duration of embankment surcharging.

Where settlements of the foundation soils were estimated to be greater than 1 m, the analyses assumed that an additional 0.5 m thick rock fill top-up would be placed prior to placement of the surcharge load to compensate for these large settlements.

To estimate the rate of excess pore water pressure dissipation and consolidation, analyses were carried out to assess the effect of different wick drain spacings on the response of the foundation soil deposits to the proposed embankment fills.

The analyses employed the analytical solutions for assessing the degree of consolidation by radial (or horizontal) drainage proposed by Baron (1948), including the extended solutions of Hansbo (1979) developed specifically to assess the use of wick drains for the consolidation of compressible cohesive deposits.

The extended solutions by Hansbo (1979) permit including the effects of the wick drain well resistance / discharge capacity and the effects of smear of the soil along the wick drain (due to installation) on the rate of excess pore water pressure dissipation / consolidation.

The secondary compression settlements were calculated using appropriate values of $C_{\alpha(\varepsilon)}$ for both the over-consolidated and normally-consolidated portions of the deposits. $C_{\alpha(\varepsilon)}$ was assessed from results of standard (24 hour) incrementally loaded (IL) consolidation tests and results of index testing using the empirical correlation provided by Mesri (1973). A $C_{\alpha(\varepsilon)}$ of 1.0% was selected for the normally-consolidated portion of the stratum and a $C_{\alpha(\varepsilon)}$ of 0.2% for the over-consolidated portion of the stratum.

The coefficients of consolidation in the horizontal direction (c_h) were assessed primarily from the results of 33 pore water pressure dissipation tests carried out as part of the CPT testing. A value of c_h was also assessed from the results of a laboratory consolidation test performed on a vertically trimmed specimen from a Shelby tube sample. Based on the field and laboratory data, a c_h value of 2.0×10^{-2} cm^2/sec was assigned to the upper clayey silt to silty clay deposit (above Elevation 303 m) as well as to the lower clayey silt deposit (below Elevation 295 m), and a c_h value of 5.0×10^{-3} cm^2/sec was assigned to the middle silty clay to clay deposit (between Elevations 303 m and 295 m).

The horizontal permeability of the cohesive deposits immediately adjacent to the wick drain is generally less than the permeability measured or estimated for the overall cohesive deposits as a result of localized disturbance / smearing of the soil caused by insertion of the steel mandrel into the subsurface during installation of the wick drains. Based on published information in literature and previous project experience in Northern Ontario, (Dittrich et al., 2010) a smear ratio (k_h/k_s) of 5 was employed for the design of the wick drain foundation system.

2.3 Geotechnical Instrumentation and Monitoring Program

A specification for the supply of geotechnical instrumentation, consisting of 18 settlement plates (SPs) and 24 vibrating wire piezometers (VWPs), and a monitoring program was prepared as part of the design phase. The purpose of the monitoring program was to confirm that, over the duration of construction, the magnitude of excess pore water pressures and settlements measured in the field were accurately represented by the design model. The SPs and VWPs were specified to be installed following peat sub-excavation, preparation of a drainage blanket, and wick drain installation.

3 EMBANKMENT CONSTRUCTION

Construction activities at Swamp Crossing H6/H7 commenced in April 2016 with the sub-excavation of surficial peat deposits. The thickness of the peat deposits ranged from about 0.1 m to as much as 4 m in some areas. The peat sub-excavation and backfilling operations were carried out in accordance with OPSS 209 (Construction Specification for Embankments over Swamps). Backfilling included the placement of a 0.5 m thick granular filter blanket designed for the wick drain installation and drainage above the original ground surface.

The selected wick drain for the site was a MebraDrain MD-88. Over 11,000 wick drains with embedment ranging from about 7 m to 22.5 m were installed at an equilateral triangular grid spacing of 1.5 m for a total quantity of 175,000 linear metres.

Table 1 summarizes the embankment construction restrictions and waiting/delay periods specified in the

Contract as well as the actual hold times that occurred during construction.

Table 1. Embankment construction staging details

Stage No.	Fill/Lift Thickness (m)	Hold Time Before Next Stage (days)	Actual Hold Time Before Next Stage (days)
1	0.5	60[1]	18 to 71
2[2]	1.0	60	23 to 43
3	Up to 4.0[3]	30	140
4	2.0	44	80

[1] Fill placement on the drainage blanket was restricted between December 1 and June 1.

[2] Stage 2 includes construction of stability toe berms (1.5 m high by 5 m wide).

[3] Fill thickness includes an additional 0.5 m of rock fill top-up prior to surcharge load to compensate for settlement of fill during construction between Stages 1 and 3.

Table 1 (Note 1) reflects the requirements in OPSS 220 (Construction Specification for Wick Drain Installation) that precludes wick drains installation in frozen ground. OPSS 206 (Construction Specification for Grading) also imposes restrictions for fill placement in winter. Wait times for fill Stages 1 to 3 were established based on the excess pore water pressure development and rate of dissipation estimated at the design stage to maintain embankment stability. The surcharge waiting/delay period was assessed based on the time required to achieve the post-construction settlement criterion of 200 mm over a 20-year period following completion of construction (i.e., removal of surcharge).

Due to delays elsewhere in the contract, an anticipated wick drain installation start date of August 17, 2016 was estimated. With a production rate of 8000 m/day using two crews and weekend shifts, the wick drain installation was scheduled to be completed in 22 days. Factoring the baseline time restriction (10 days) and the 60 day wait/delay period for Stage 1 outlined in Table 1, insufficient time would remain for the placement of rock fill during Stage 2 before the December 1, 2016 deadline. Stage 3 rock fill and Stage 4 surcharge would not have been possible before the December 1 limit and therefore a contractual delay was projected. Figure 2 illustrates the wick drain installation activity.

Figure 2. Wick drain installation at Station 14+650

With the objective of placing the embankment surcharge before winter shutdown, the designer was requested by MTO to revisit the waiting/delay periods specified in the contract documents. The stability analysis was refined to consider that some dissipation would occur in advance of instrument installation and during fill placement over the length of the swamp. It was determined that the waiting/delay periods for Stages 1 and 2 could be reduced provided that the measured excess pore water pressures did not exceed the Review and Alert Levels (i.e., acceptable levels) established during preparation of the monitoring program.

Instrumentation installation/baselining was completed on October 7, 2016. Monitoring of the instruments occurred throughout construction at intervals specified in the Contract. The stability of the embankment in response to additional loading was assessed based on the measured pore water pressures prior to each stage. In addition, the readings were assessed in advance of surcharge removal to check that the excess pore water pressure response and magnitude / time-rate of settlement were similar to the design estimates.

Stage 2 rock fill placement commenced on October 11, 2016 and was completed on October 31, 2016. A total of 17 days elapsed following Stage 1 filling to enable excess pore water pressures to dissipate to acceptable levels. Figure 3 illustrates the Stage 2 rock fill placement.

Stage 3 rock fill placement was completed between November 23 and December 2, 2016. A total of 22 days elapsed after Stage 2 filling to enable the excess pore water pressures to sufficiently dissipate.

Stage 4 surcharge placement was deferred until the Spring of 2017 (April 2017), corresponding to a Stage 3 hold time of approximately 140 days. The Contract required a wait/delay period of 44 days with the surcharge in place; however, based on a reassessment of the measured settlement and rate of excess pore water pressure dissipation with the revised rate of staged construction, it was determined that following the originally planned 2 m surcharge with a wait/delay period of 44 days would not achieve the target post-construction settlement criterion.

Figure 3. Rock fill placement during Stage 2

Several alternatives were considered to achieve the post-construction settlement criterion including:

i) additional wait/delay time with the surcharge;
ii) placement of additional surcharge material (i.e., higher surcharge);
iii) construction of a lightweight fill (e.g., expanded polystyrene) core within the final geometry of the embankment; and,
iv) a combination of these mitigation measures.

It was concluded that partial removal of the embankment fill and replacement with expanded polystyrene core would have large material and time cost implications. Therefore, the selected preferred option was to increase the height of the granular surcharge and extend the surcharge wait period. It was recommended to place an additional 3 m of granular fill (1 m of granular fill to compensate for the settlement during embankment construction plus 2 m of granular fill above the proposed final grade). Surcharge removal commenced in late July 2017 (80 day hold time) as illustrated on Figure 4.

Figure 4. Removal of granular surcharge material

4 POST-CONSTRUCTION MONITORING

Typically, during surcharge removal, the SPs are decommissioned and the VWPs are no longer monitored; however, the MTO requested additional monitoring and assessment be carried out to compare measured field properties to estimates from design and to further evaluate the post-construction embankment settlement behaviour. Consequently, select SPs were maintained during surcharge removal to permit additional settlement readings and a post-construction monitoring program was developed to allow remote acquisition of pore water pressure and settlement data following surcharge removal.

Three locations along the swamp crossing were targeted for post-construction monitoring as follows: i) Station 14+090, corresponding to the largest measured settlement (i.e., critical section); ii) Station 14+180 with similar clay thickness to the critical section, but a lower final embankment height, and; iii) Station 14+270 with thinner clay and a lower embankment height.

An additional six VWPs were installed to measure pore water pressure throughout the clay deposit to help identify the end of primary consolidation. Three vibrating wire inline extensometers (VWIXs), with five anchors and four transducers each, were also installed after surcharge removal, along the shoulder of the new embankment. The bottom anchor at each location was installed within the cohesionless soils below the clay deposit to act as a fixed point, against which settlements could be measured. Total magnitudes of settlement for each station were calculated by summing the post-construction settlement measured by the VWIXs with the settlement measured during construction by adjacent SPs. Following installation of the post-construction instrumentation in November 2017, the pore water pressure and settlement readings from the site have been acquired remotely.

The results of the monitoring compared to the settlement estimates (updated/revised to reflect the actual rate of embankment fill placement) developed during design are shown for Station 14+090 on Figure 5. Note that, for clarity and presentation purposes, only the initial 1000 days of the predicted settlement are shown on the plots. The settlement monitoring data suggests that the cohesive deposit was near the completion of primary consolidation at the time of the additional field investigation.

5 SUMMARY OF FIELD INVESTIGATION

During the detailed design of the project, 49 boreholes and CPTs were advanced within Swamp Crossing H6/H7. In-situ shear strength testing was carried out within the cohesive deposits using a MTO ‘N’-size vane and a calibrated torque wrench. Collection of soil samples for consolidation and high complexity laboratory testing during design was carried out using 75 mm diameter, 600 mm long, Shelby Tube samples obtained by manual push of a standard sampler.

As part of the post-construction assessment of the swamp crossing, an additional 9 boreholes and 6 CPTs were advanced. The boreholes were advanced to collect samples for additional laboratory testing and to complete in-situ shear strength tests at the three monitoring array sections. At each monitoring array section one borehole and one CPT was advanced as far as permissible, limited by property constraints, beyond the toe of the embankment; whereas, two boreholes and one CPT were advanced through the embankment shoulder. 75 mm diameter Shelby Tubes samples for the post-construction assessment were obtained using a hydraulic piston sampler. In-situ shear strength carried out for the post-construction assessment followed similar procedures to the testing completed during the detailed design phase.

6 SUMMARY OF LABORATORY TESTING

Both strength and compressibility testing were carried out as part of the high complexity testing lab program, but only the compressibility testing will be discussed herein.

Six incrementally loaded (IL) 24 hour consolidation tests (ASTM D2435/D2435M–11), were carried out during the detailed design phase of the project. Subsequently, as part of the post-construction assessment of the swamp crossing, one X-ray diffraction test (XRD), an additional ten standard IL consolidation tests, three long-term IL consolidation tests, and five constant rate of strain (CRS) consolidation tests (ASTM4186/D4186M–12) were completed. For consistency and comparison purposes, the new samples were obtained at similar elevations compared to those samples obtained during the detailed design stage. The new samples were also recovered from below the new highway embankment and beyond the toes of the embankment in order to assess the effect of embankment loading on soil properties.

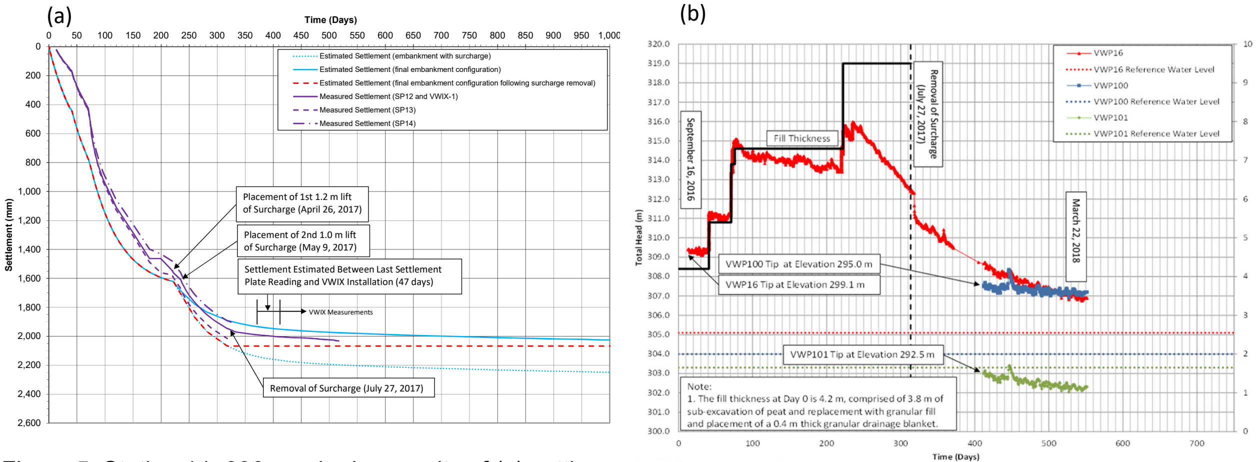

Figure 5. Station 14+090 monitoring results of (a) settlement; (b) pore water pressure

The additional laboratory testing was carried out to: i) identify the mineralogy of the clayey soils; ii) compare disturbance impacts from sampling techniques, and; iii) assess if additional and more complex testing (i.e., CRS and long-term IL consolidation tests) would provide improved/more refined results.

The Specimen Quality Designation (SQD) system, a term coined by Terzaghi et al. (1996) based on the method proposed by Andresen and Kolstad (1977), was used to evaluate the quality of samples selected for the consolidation tests. The SQD of consolidation samples is dependent on various factors such as, sampling procedures, transportation, storage, and specimen preparation. The sample quality of recovered samples ranged from A (best) to E (worst); however, on average, the sample quality was A or B. Inappreciable difference was observed between samples obtained using manual sampling methods compared to those obtained using a hydraulic piston sampler. For the assessment of this swamp crossing, only specimens with an SQD of A, B or C were considered (i.e., less than 4% axial strain at the estimated in-situ effective stress).

6.1 Index Classification Testing

The qualitative soil composition determined from the XRD test carried out on Borehole 1B, Sample 10 revealed that the primary clay minerals present are illite and chlorite. The results of water content and Atterberg limits tests are presented in Table 2. Previous testing carried out by others on cohesive deposits in Northeastern Ontario (Quigley et. al, 1972) yielded similar results.

Table 2. Summary of water content and Atterberg limits test results

Material	w_P (%)	w_L (%)	w_n (%)	PI [1] (%)	LI [2] (%)
Clayey Silt to Clay	15 to 27	23 to 73	12 to 93	6 to 46	0.5 to 3.5
Average:	21	43	48	22	1.4

[1] PI denotes Plasticity Index.

[2] LI denotes Liquidity Index.

6.2 Incrementally Loaded Consolidation Tests

In general, the standard IL consolidation tests were carried out on specimens approximately 25 mm thick. Given the irregularly stratified or varved composition of the samples, an attempt was made to select relatively homogeneous samples; however, at the time of extrusion, the silty and clayey layers/laminae in some samples were difficult to identify without disturbing or drying the samples. As shown on Figure 6, the orientation and thickness of the layers/laminae observed on the samples varied considerably.

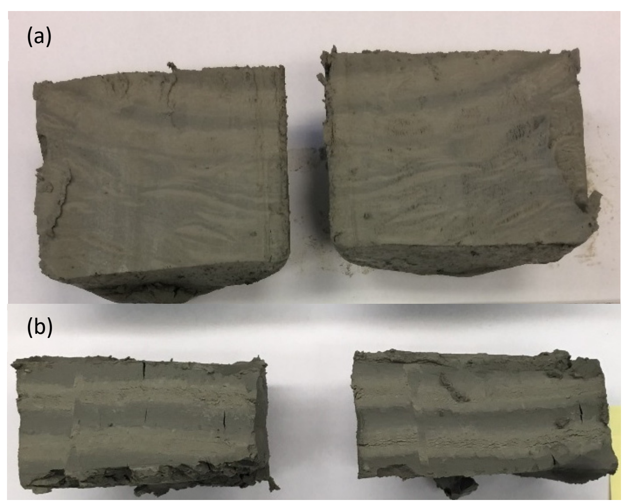

Figure 6. Silty and clayey layers/laminae observed on specimens recovered from (a) Borehole 1B Sample 10, and; (b) Borehole 3B Sample 6

For the standard consolidation testing, a load increment ratio (LIR) of one was used, with a duration of approximately 24 hours, which was sufficient to reach the end of primary consolidation at each loading increment.

However, for the long-term testing, an approach proposed by Watabe et al. (2012) was generally implemented. Specifically, for the long-term consolidation testing, a LIR of one with a load increment duration of 24 hours was used until the estimated in-situ effective stress was reached, at which point the loading was maintained for a period of seven days in an attempt to reduce specimen disturbance. The loading was then increased to coincide with the final in-situ effective stress corresponding to the final embankment geometry (surcharge loading was not considered) and maintained for a period of approximately 100 days.

The end of primary void ratio (e_{EOP}) and the corresponding length of time for each loading increment were determined using the root time method proposed by Taylor (1942).

An example of the various void ratio versus log stress ($e – \log\sigma'_v$) curves from the IL consolidation testing carried out on samples recovered at Station 14+090 are presented on Figure 7. The wide range of the measured initial void ratio (i.e., varying from about 0.6 to 1.95) associated with specimens obtained from similar elevations can be attributed to the irregularly stratified/varved nature of the recovered samples (i.e., some of the specimens were more silty and some were more clayey).

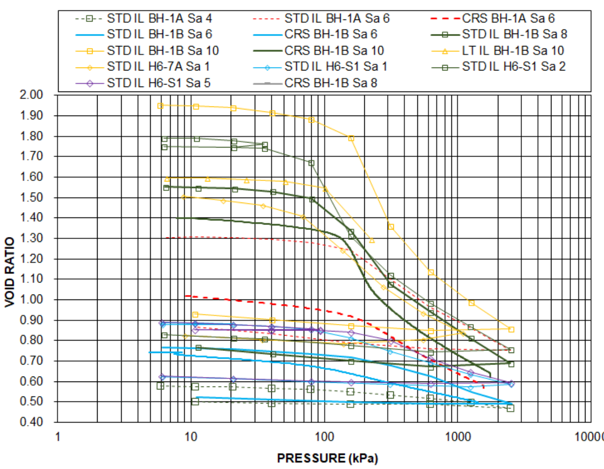

Figure 7. Station 14+090: summary of $e – \log\sigma'_v$ curves

6.3 Constant Rate of Strain Consolidation Tests

CRS consolidation tests were also carried out on specimens approximately 25 mm in thickness. The testing was completed using a strain rate selected to be near the typical end of primary strain rate for clays proposed by Mesri et al. (2002) while maintaining conformance with ASTM4186/D4186M–12. The results of the $e – \log\sigma'_v$ curves were adjusted to the specimen specific end of primary void ratio using the method proposed by Mesri et al. (2002). Examples of the CRS consolidation test $e – \log\sigma'_v$ curves compared to those obtained from standard and long-term IL consolidation tests completed on specimens from the same Shelby tube sample are shown on Figure 8. The unmodified $e – \log\sigma'_v$ curves are also presented to highlight the secondary compression effects during each loading increment.

In the opinion of the authors, the $e – \log\sigma'_v$ (corrected to end of primary void ratio) of the CRS and IL consolidation testing provide similar results. The variability of the initial void ratio from specimens taken from the same Shelby tube samples suggest difficulties with obtaining repeatable results from cohesive deposits that are not homogenous in

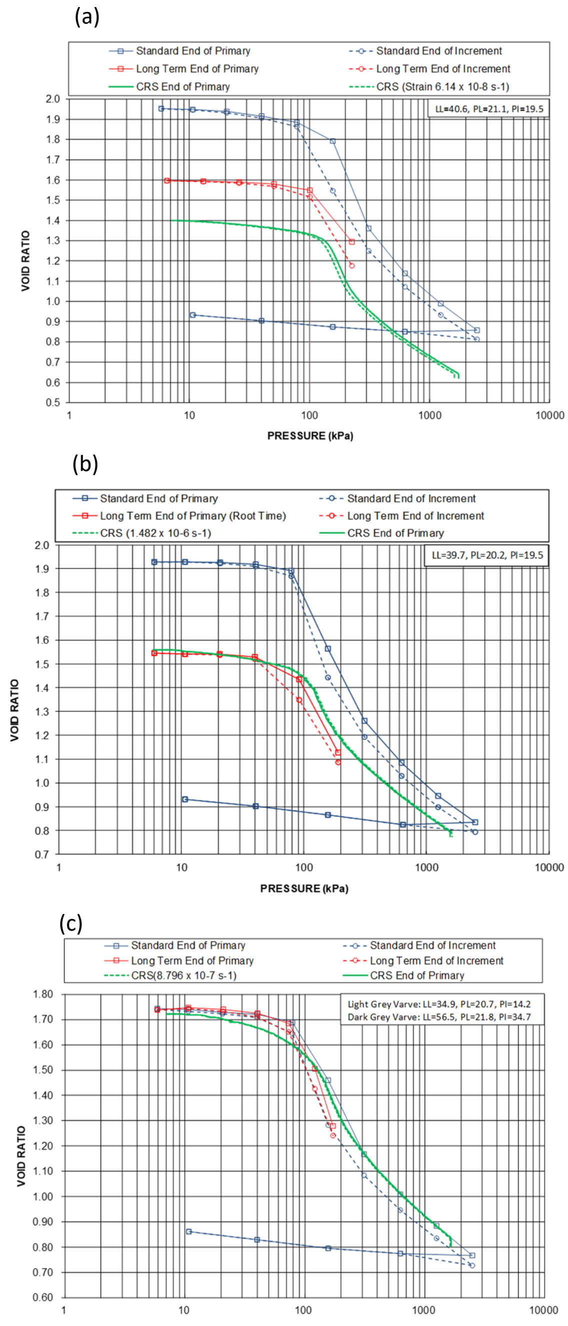

Figure 8. Comparison of CRS as well as standard and LT IL consolidation tests carried out on (a) Borehole 1B Sample 10; (b) Borehole 2B Sample 9, and; (c) Borehole 3B Sample 6

nature (i.e., deposits that are stratified or varved) and require thoughtful interpretation in selecting soil parameters during design. However, the slopes of the $e – \log\sigma'_v$ curves, by load increment, and the interpreted preconsolidation pressure (σ'_{pc}) of the specimens are generally consistent regardless of the initial void ratio. Therefore, provided that the average in-situ initial void ratio is not based on a single consolidation test, but rather comprehensively estimated through moisture content testing, and if the specimen is representative of the bulk sample, the impact of thin layers/laminae on estimating the magnitude of settlement for this type of cohesive deposit is expected to be minimal.

7 RESULTS AND DISCUSSION

Deformation properties interpreted from laboratory testing were used to establish design lines to be used in the settlement analysis for the swamp crossing. A summary of the geotechnical soil parameters and design lines for Swamp Crossing H6/H7 is presented on Figure 11.

7.1 Preconsolidation Pressure

The σ'_{pc} of the clay was estimated from the consolidation tests using the method proposed by Oikawa (1987), which has been suggested by Umar and Sadrekarimi (2016) to provide improved accuracy as compared to alternative methods. The undrained shear strength (s_u) and σ'_{pc} were subsequently correlated from the in-situ shear strength testing and laboratory consolidation testing using Equation 1 (Mesri, 1975).

$$\left(\frac{s_u}{\sigma'_{pc}}\right)_{field} = 0.22 \qquad [1]$$

The initial void ratio (e_o) of the cohesive deposit was determined from the moisture content and specific gravity laboratory testing carried out on specimens of the cohesive deposit. The compression index (C_c) values provided on Figure 11 correspond to the linear portion of the e – logσ'_v curves at stresses immediately beyond σ'_{pc} (i.e., generally corresponding to a stress range between about 150 kPa and 300 kPa). The two empirical correlations of estimating C_c based on the water content and liquid limit or e_o are presented on Figure 11 and are considered to be similar to the results of the consolidation testing. The two empirical correlations are shown in Equation 2 (Koppula, 1986) and Equation 3 (Azzouz, 1976).

$$C_c = 0.009w_n + 0.005w_L \qquad [2]$$

$$C_c = 0.75(e_o - 0.50) \qquad [3]$$

7.2 Void Ratio and Compression Index

The void ratio versus compression index (e – C_c) plot presented on Figure 9 shows the results from the consolidation testing carried out at this site. The scatter may be attributed to the stratified/varved nature of the cohesive deposits encountered at the site and would make a site-specific correlation difficult. The results from the IL consolidation testing are consistent with the observations noted during the consolidation testing (see Section 6.3), where e_o varied significantly based on specimens tested from a given Shelby tube sample; however, C_c and σ'_{pc} remained relatively constant for a given load increment. The results of the CRS consolidation testing were interpreted over each recorded time interval and the C_c ranged greatly (i.e., from about 0.2 to 50) above a void ratio of one. In general, the bulk of the CRS consolidation data was consistent with the results from the IL consolidation testing.

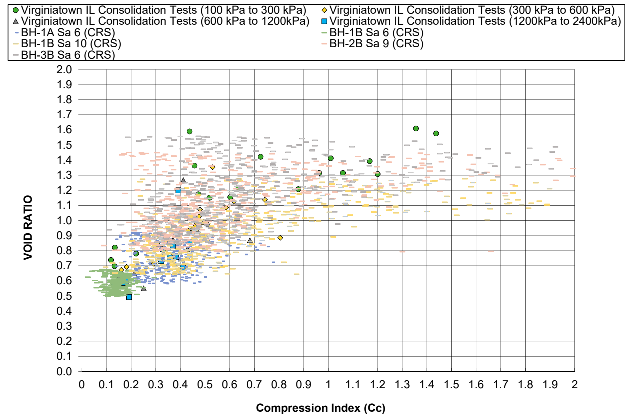

Figure 9. Consolidation testing results, relationship between void ratio and compression index (C_c)

7.3 Secondary Compression Index

Each C_c value estimated from Equation 2 and Equation 3 were further related using Equation 4 (Mesri and Castro, 1987) to estimate a corresponding secondary compression index (C_α).

$$\frac{C_\alpha}{C_c} = 0.04 \pm 0.01 \qquad [4]$$

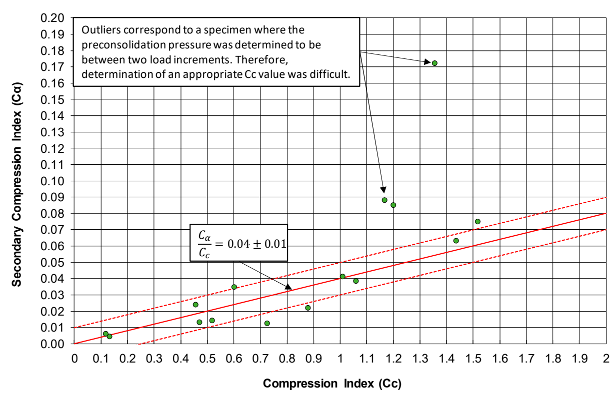

Figure 10. IL consolidation testing, relationship between secondary compression index (C_α) and compression index (C_c)

One C_α from each IL consolidation test was interpreted from the displacement versus log time readings at the load increment corresponding closely to the in-situ final effective stress due to the embankment loading at final grade. For the purposes of this study, surcharge effects were not considered. A summary of the C_α values interpreted from the IL consolidation tests is shown on Figure 10. The three outliers on Figure 10 demonstrate that variation beyond the predicted ratio is observed where portions of the e – logσ'_v curve slope are highly non-linear. For example, the three C_α values, which are associated with specimens where the load increment was near the σ'_{pc}, were calculated to be substantially higher than predicted. This suggests that the interpretation of $\frac{C_\alpha}{C_C}$ from IL consolidation testing carried out with a LIR of 1 may provide misleading results for the loading increment near σ'$_{pc}$; therefore, judgement is required.

7.4 Horizontal Coefficient of Consolidation

The horizontal coefficient of consolidation (c_h) is a critical parameter in the design a wick drain foundation systems. The rate of excess pore water pressure dissipation and consolidation is governed in the horizontal direction (as opposed to the vertical direction) which represents the shortest drainage path as a result of the wick drains. Consequently, the c_h was assessed from 33 and 57 pore water pressure dissipation tests which were carried out as part of the CPT testing during design and as part of the post-construction field program, respectively. The formulations proposed by Teh (1987) and Houlsby and Teh (1988) were used to estimate the c_h values. A summary of the estimated average c_h values is provided in Table 3 and shown on Figure 11, which also includes the original design lines used in the average degree of horizontal consolidation calculations.

Table 3. Summary of average coefficients of consolidation in the horizontal direction (c_h)

Elevation (m)	Material Type	c_h (cm^2/sec) Design	c_h (cm^2/sec) Research
Above 303	Clayey Silt to Silty Clay	4.1×10^{-2}	7.6×10^{-3}
303 to 295	Silty Clay to Clay	1.7×10^{-2}	7.6×10^{-3}
Below 295	Clayey Silt	3.7×10^{-2}	8.0×10^{-2}

The c_h estimated in the upper clayey silt to silty clay deposit and in the underlying silty clay to clay deposit during the post-construction field program was about 5.3 times and 2.2 times lower when compared to the c_h estimated during detail design in the respective deposits. The reduction in the average c_h can likely be attributed to the increase in the vertical effective stress as a result of the loading stress imposed by the new embankment. The loading stress is highest immediately below the base of the sub-excavated zone and this correlated well with large decrease in c_h above Elevation 303 m. Based on Figure 11, it is evident that with depth, as the loading stress dissipates, the difference between the c_h measured during the post-construction field program and during the original design becomes smaller.

Conversely, in the lower clayey silt deposit, the c_h estimated during the post-construction field program was about 2.2 times larger. However, based on the limited amount of tests in the lower cohesive deposit, and considering the low loading stress at depth, it is assumed that the average c_h estimated at depth was not affected by embankment construction.

There was no appreciable difference between the c_h estimated from pore water pressure dissipation tests carried out at the toes of the new embankment and near the centerline of the new embankment.

In addition, the similarity between the time-rate of settlement based on the measured in-situ data and the model established during the design stage (as shown on Figure 5), indicates that c_h values estimated from the CPT pore water pressure dissipation tests correlate well with the in-situ c_h values.

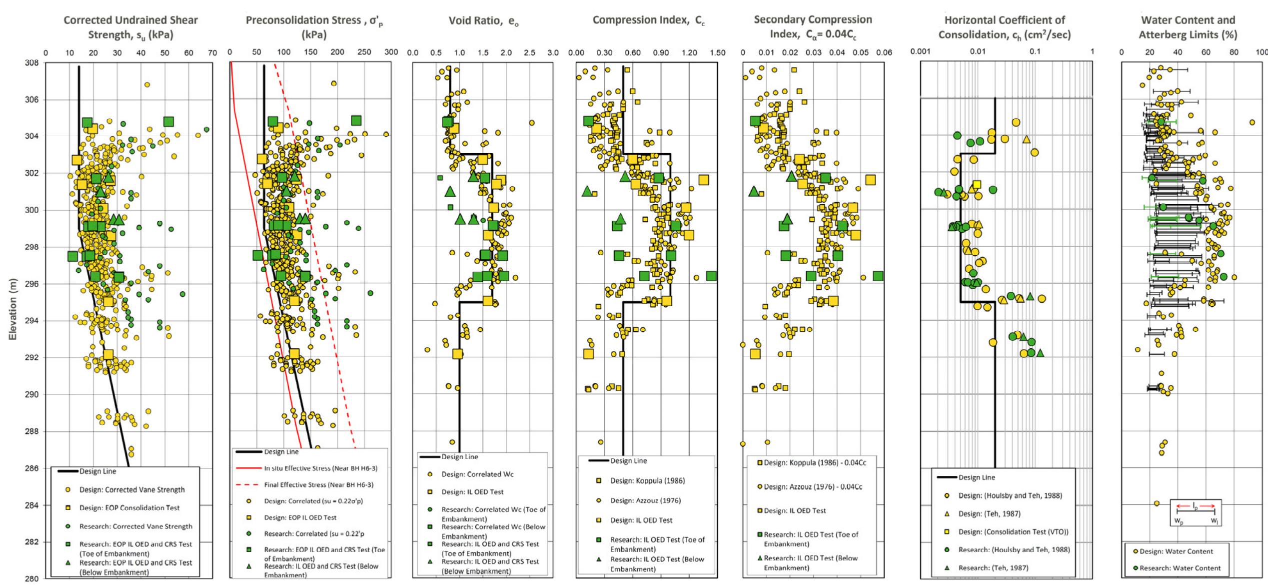

Figure 11. Swamp Crossing H6/H7: summary of soil parameters and design lines

8 CONCLUSIONS

A case study of an approximately 3.5 m high embankment constructed on soft compressible soils in Northeastern Ontario, which has settled more than 2 m during construction, has been examined. The results of laboratory testing and field monitoring have been presented to review the suitability of estimating compressibility parameters using standard testing procedures (IL consolidation tests). As part of the review, relevant compressibility parameters were determined from standard IL, long-term IL and CRS consolidation tests. Following correction of the e – logσ'_v curves to end of primary consolidation, the following observations were noted during a review of the laboratory parameters:

i) The difference in SQD between samples obtained using a standard manual sampler and hydraulic piston sampler was negligible.
ii) e_o measured on specimens recovered from the same Shelby tube sample varied significantly, which caused difficulty in obtaining repeatable results from a varved/stratified deposit and suggests that great care must be taken in assessing the composite behaviour of non-homogenous clayey soils.
iii) σ'_{pc} values interpreted from the e – logσ'v curves remained relatively consistent between the standard IL, long-term IL and CRS consolidation tests. The variability in e_o did not have a significant impact on the estimate of σ'_{pc}.
iv) C_c values interpreted over the same stress ranges beyond the σ'_{pc} were generally similar amongst the standard IL and CRS consolidation tests, regardless of the initial e_o; however, some discrepancy was noticed with the long-term IL tests, which might be attributed to impacts on the soil properties due to the significantly longer duration of the test.
v) C_α values interpreted from the IL consolidation tests at stresses corresponding to the final embankment geometry generally correlated well with Equation 4, proposed by Mesri and Castro (1987). However, in areas where the final stress was near the σ'_{pc}, Equation 4 yielded substantially different results as compared to the laboratory interpretations. In the authors' opinion, this is likely attributed to the variability of C_c near σ'_{pc}, which presents difficulties in comparing C_α over the same change in void ratio on tests carried out using a LIR of one. CRS C_α values were not included as part of the comparison as sustained loading was only carried a relatively large load upon the completion of the testing.

Overall, the settlement of the Highway 66 embankment through Swamp Crossing H6/H7 has been relatively close to predictions to date using standard IL consolidation testing and design procedures.

CRS consolidation testing resulted in similar estimates of primary consolidation parameters, with the additional data points along the e – logσ'_v curve allowing for a more objective selection of σ'_{pc}; however, assumptions with regards to the rate of strain, are required to produce the end of primary e – logσ'_v curves.

Long-term IL consolidation testing also resulted in similar estimates of primary consolidation parameters; however, given the longer duration of the test, limited value may be gained in selecting this method of testing to estimate primary consolidation settlement.

Considering the varved/stratified nature of clayey soils encountered frequently in Northeastern Ontario, difficulties were experienced in obtaining samples with a similar e_o. However, the various types of consolidation tests resulted in similar estimates of compressibility parameters. These results suggest that increasing the number of standard IL consolidation tests might be more beneficial in understanding the behavior of bulk samples rather than carrying out fewer, but more complex, tests.

Monitoring at the site is on-going and will be carried out over several years. Future studies will include: i) a comparison on the c_v results from the various testing methods ii) a comparison of different settlement estimation methods to the measured field data; and, ii) a long-term assessment of the secondary compression (creep) behavior observed on-site.

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