Performance of Embankments on Soft Ground at Highway 11 Underpass at Black Creek/Robins Road in Northeastern Ontario

Keli Shi, Jason Lee
Thurber Engineering Ltd, Oakville, Ontario, Canada
Tony Sangiuliano, Brady Lin
Ministry of Transportation Ontario, Downsview, Ontario, Canada

ABSTRACT
This paper presents a case study that describes design, construction and post construction investigation and monitoring of approach embankments at Highway 11 Underpass at Black Creek/Robins Road located in Northeastern Ontario. The approach embankments approximately 10 m in height were constructed on up to 35 m of soft to firm clay deposit using a combination of ground improvement techniques including wick drains, surcharge, stabilizing berm and light-weight fill (Expanded Polystyrene - EPS blocks). Subsurface conditions at the site are comprised of deposits of slightly over-consolidated to normally consolidated glaciolacustrine silty clays. The soft to firm and highly compressible soils posed short term stability and long-term settlement challenges.

This paper provides engineering design and construction background applied to ensure short term and long term performance of the approach embankments. The paper presents findings of data collected to examine the soil properties and behaviour following construction of the embankments. A comparison of the soil properties of the foundation clays before and after embankment construction has been conducted to better understand the effects of ground improvement and to enable more accurate predictions of strength gain and magnitude and time rate of settlements.

RÉSUMÉ
Cet article présente une étude bref de la conception, construction et surveillance après construction pour les remblais adjacent au pont de Black Creek/ Robins Road a Route 11 au Nord-est Ontario. Les remblais, dont environ 11 mètres en hauteur, soutiennent par la couche argiles molles à maximum 35 m en profondeur. Les remblais ont construit par amélioration des sols comme drains verticaux, surcharge, et remblais légers (EPS). Ce dépôt de l'argile silteuse glaciolacustrine est presque normalement consolidé et la condition molle et compressible pose des complexités à propos de la stabilité et le tassement pendant de temps immédiate et éloigné.

Cet article résume les méthodes des conception et construction utilisent pour vérifier la performance des remblais adjacent au pont. On présente une description sommaire des données géotechniques et un résumé du comportement en construction. On évalue les changements de sol pendant le chargement et discuter des effets des modes amélioration de sol.

1 INTRODUCTION

The Highway 11 Black Creek Road / Robins Road Interchange located in Northeastern Ontario was part of the Ministry of Transportation (MTO) Highway 11 four-laning project from north of Burk's Falls to south of Sundridge. The interchange was constructed under two separate contracts. The first contract (Contract 2007-5188) was an advanced contract that included peat sub-excavation, wick drains, instrumentation monitoring, embankment construction and surcharging in order to address embankment stability and settlement challenges associated with proposed embankment fill heights in the order of 10 metres and deep deposits of up to 35 metres of compressible silty clay. In the second contract (Contract 2008-5113), the Hwy 11 underpass structure, a two-span structure that carries Black Creek/Robins Road over Highway 11 and founded on driven steel H-piles to refusal was constructed.

The presence of a deep deposit of soft to firm silty clay up to 35 metres in thickness provided a number of engineering challenges. A combination of foundation improvement and staged construction was required to enable the approach embankments to be built to satisfy short term and long term stability and to satisfy MTO post construction settlement tolerances. The design included sub-excavation of organics, installation of wick drains, staged embankment construction, stabilizing berms, surcharge and preload, and substitution using Expanded Polystyrene (EPS) blocks. An advanced contract was also part of the strategy to ensure embankment stability and settlement performance requirements.

The original foundation investigation for this project consisted of exploratory boreholes, field vane testing and CPTU tests. Laboratory analyses consisted of index property tests and one-dimensional consolidation tests.

A unique part of the project included a post construction foundation investigation and settlement monitoring program. The foundation investigation program included advancement of boreholes, CPTUs and field vanes. Laboratory analyses included index property tests, incremental loading consolidation tests, controlled rate of strain consolidation tests, scan electronic microscopy and x-ray diffraction tests.

The purpose of the post construction program is to provide insight into the prediction of time-dependent settlement and to assess the long-term behavior of the foundation soils. A comparison of the soil properties of the foundation clays from a strength and compressibility characteristics perspective before and after embankment construction has been conducted to better understand the effects of ground improvement and to enable more accurate predictions of strength gain and magnitude and time rate of settlements.

2 INVESTIGATION PROCEDURE

Original Program

The original foundation investigation program carried out in the entire interchange area prior to the first contract included 40 exploratory boreholes, 12 dynamic cone penetration tests (DCPT) and 16 cone penetration tests with pore pressure measurements (CPTU).

Laboratory analyses during the original program included index property tests and incremental loading consolidation tests.

Post Construction Program

Figure 1 illustrates the scope of the post construction foundation investigation carried out in December 2012 and June 2013 along with the boreholes and CPTUs carried out during the original program. The post construction foundation investigation program included three (3) exploratory boreholes and three (3) cone penetration tests with pore pressure measurements (CPTU) from the paved roadway surface of the approach embankments.

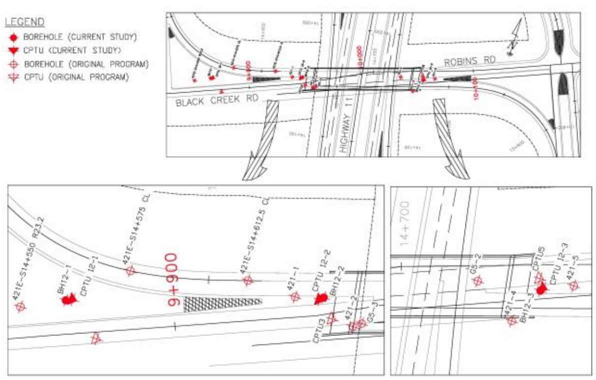

Figure 1 – Borehole and CPTU Locations

As part of the post construction investigative effort, a total of 26 consolidation tests were carried out, among which 16 were Incremental Loading (IL) type as per ASTM D2435 and 10 were Controlled Rate of Strain (CRS) type as per ASTM D4186.

Figure 2 illustrates an instrumentation plan of the long-term settlement monitoring (5 years) currently in progress. The instrumentation mainly consisted of pavement surface markers installed within 50 m of structure abutments.

The long-term settlement monitoring program included installation and monitoring of 10 survey targets on the asphalt pavement surface located at 1, 3, 5, 7, 10, 15, 20, 30, 40 and 50m behind each abutment. Monitoring of the survey targets is to last for 5 years starting in the spring of 2013.

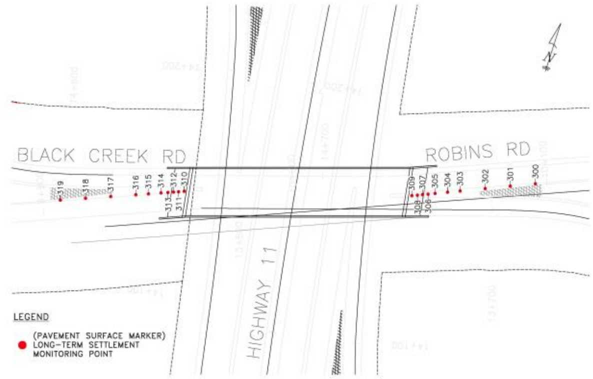

Figure 2 – Long-term Settlement Monitoring Program

3 SUBSURFACE CONDITIONS

In general, the site consisted of topsoil, peat and roadbed fills at the ground surface underlain by compressible glacio-lacustrine or fluvial deposit including silt and sand, silty clay, clayey silt and silt, which was in turn underlain by broadly graded generally very dense sand till containing cobbles and boulders. The overburden is underlain by bedrock consisting of Precambrian metamorphic granitic gneiss. Groundwater level was within 2m of the ground surface, and artesian condition (<1.5m above ground surface) was encountered within the sand till deposit. Figure 3 provides a subsurface model of the site along the approach embankments to the Highway 11 Black Creek Road (West Approach)/Robins Road (East Approach) Underpass.

A glacio-lacustrine silty clay was the predominant

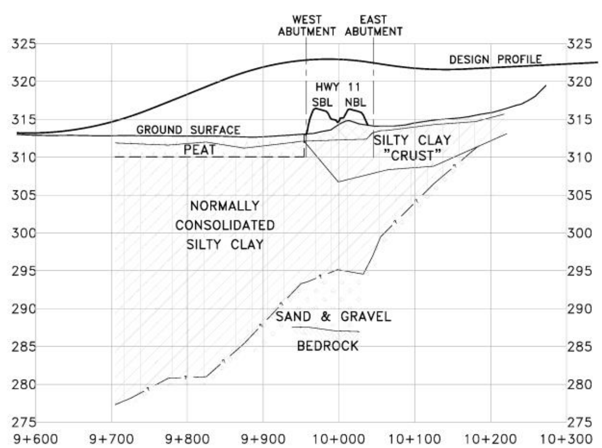

Figure 3 – Subsurface Model

deposit at the site. The thickness of this deposit increased from northeast to southwest, reaching a maximum of 36.6m (EL. 275.6m) at Black Creek Road. The upper 3 to 8 m of this deposit consisted of a crust with physical and mechanical properties distinct from the underlying deposit.

Generally brown and grading to grey at depth, natural water contents of the clay increased with depth, ranging from 22% to 41%. Pre-construction undrained shear strength was more than 100 kPa near the ground surface and decreased with depth to values in the order of 40 kPa, indicating firm to very stiff consistency.

Below the crust, the deposit consisted of an unoxidized grey silty clay ranging from low to high plasticity but generally intermediate plasticity. Silt fraction ranged 40% to 70% and the clay fraction ranged from 20% to 65%. Natural moisture contents were high, generally higher than the Liquid Limit (Liquidity Index greater than 1).

The in-situ soils were slightly over-consolidated to normally consolidated with over-consolidation pressure ranging from 15 to 64 kPa in excess of the overburden pressure. The undrained shear strengths decreased with depth from 100 to 40 kPa in the crust and increased over depth from 40 to 70 kPa below the crust with a S_u/σ'_v ratio of 0.2 to 0.3. The sensitivity of the soil ranged from low to highly sensitive but generally medium with an average value of 4.

Pre-construction consolidation pressures and the compressibility characteristics of the soils were evaluated by eleven Oedometer tests and several CPTU tests. Pre-consolidation pressures ranged typically from 15 to 64 kPa above the overburden pressure in the top 6.4m below which the clay was normally consolidated. The compression index ratio ranged from 0.07 to 0.36, averaging 0.17; the vertical coefficient of consolidation ranged from 25 to 100 m^2/year, mostly between 25 and 50 m^2/year; the horizontal coefficient of consolidation ranged from 30 to 500 m^2/year, with the higher values (greater than 200 m^2/year) measured in silt/sand lenses; the coefficient of secondary consolidation measured from time dependent Oedometer tests for approximately 48 hours at constant stress ranged from 0.001 to 0.015, averaging 0.003.

Increase in silt content was noted at the bottom 1 to 4m of the clay deposit in some of the investigation boreholes and the soils were described as clayey silt and silt. The cohesive clayey silt was typically stiff to very stiff and cohesionless silt was compact with the SPT “N” values ranging from 2 to 15. Natural water content ranged from 22% to 41%. Based on Atterberg Limits test conducted on one soil sample, the soil was CL-ML according to the Modified Unified Soil Classification System.

Cohesionless sand till with thickness up to 10.1m was encountered underlying the silty clay and clayey silt. The sand till predominately consisted of silty sand or gravelly sand, with variable amount of silt, gravel, cobbles and boulders. The sand till was dense to very dense and the measured natural water content typically ranged from 10% to 15%.

Underlying bedrock was Precambrian metamorphic bedrock, consisting of granitic gneiss. Both TCR and SCR ranged from 97% to 100% and RQD from 74% to 100%. The bedrock was strong to very strong. The bedrock surface varied from El. 276.1 to 289.0m where proved by coring.

4 EMBANKMENT AND FOUNDATION DESIGN

Geotechnical design of the embankments and foundation was carried out in 2005 and 2006 with extensive analyses to achieve two main objectives, i.e. embankment stability during construction and post-construction embankment settlement meeting MTO requirements. Locally produced rock fill was the main building material for the embankments at the project site.

In-situ shear strengths of the foundation clay did not permit single-stage construction of the approach embankments. Through multiple rounds of slope stability analysis coupled with consolidation (excess pore pressure dissipation) analysis, the final embankment design consisted of up to 5.5m high and 16m wide stabilizing berm and staged construction in two to three stages with waiting time between stages up to three-month in duration. Prefabricated vertical drains (wick drains) spaced at 1.5m in a triangular grid pattern were required for 90% to 98% excess pore pressure dissipation during waiting time to enable sufficient strength gain in the foundation clay.

The main challenge for the geotechnical design of this project was the requirement to develop a cost-effective scheme to mitigate post-construction embankment settlement behind abutments and to meet MTO’s Settlement Criteria (25 to 50mm within 50m of bridge abutments). An iterative approach was adopted to optimize the maximum height of surcharge (temporary fill placed above design road grade) and the minimum thickness of EPS fill (replacement of conventional fill below pavement structure) to achieve sufficient over-consolidation in the foundation clay and hence to limit the long-term embankment settlement. Figure 4 shows the final embankment design which included 2m surcharge and up to 3m EPS in the West Abutment area, and 3.5m surcharge and up to 1m EPS in the East Abutment area.

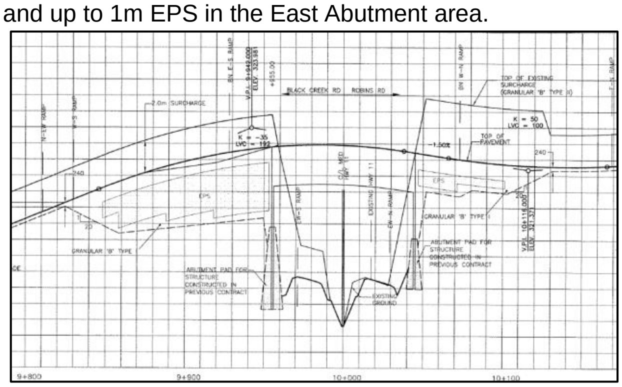

Figure 4 – Final Design of Approach Embankments

The bridge was a two-span bridge founded on steel HP310x110 piles driven to bedrock.

5 RESULTS AND DISCUSSION

Construction of the embankments and Underpass structure was carried out between 2007 and 2011. The entire interchange project was completed in the fall of 2011. The maximum monitored foundation settlements during embankment construction were approximately 0.9m in the east abutment area, and 1.5 to 2m along the west approach, generally increasing from the west abutment towards the west. Figures 5 and 6 illustrate the site conditions during approach construction and after bridge completion, respectively.

Figure 5 – Looking West at West Approach during Surcharging

The post-construction research program was initiated in late 2012. The program consisted mainly of long term settlement monitoring and characterization of the strength and compressibility properties of the consolidated foundation clay.

Figure 6 - Looking North at Underpass after Completion

5.1 Settlement Monitoring

Figures 7 and 8 illustrate the results of settlement monitoring for the underpass bridge approaches up to the end of 2017.

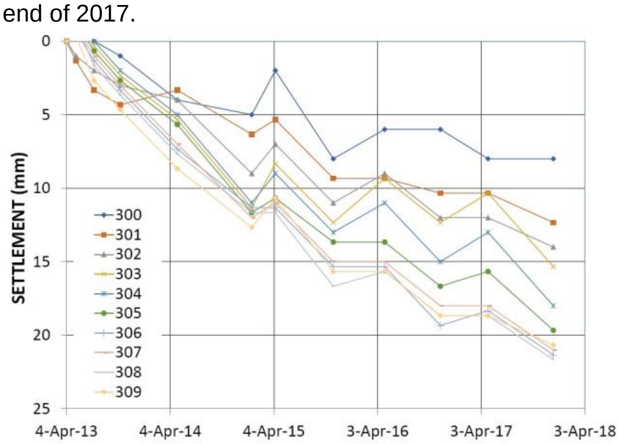

Figure 7 – Robins Road (East Approach) Settlement

At the Robins Road (East Approach), survey target 300 was located farthest from the east abutment and 309 immediately behind the east abutment. At the Black Creek Road (West Approach), survey target 319 was located farthest from the west abutment and 310 immediately behind the west abutment.

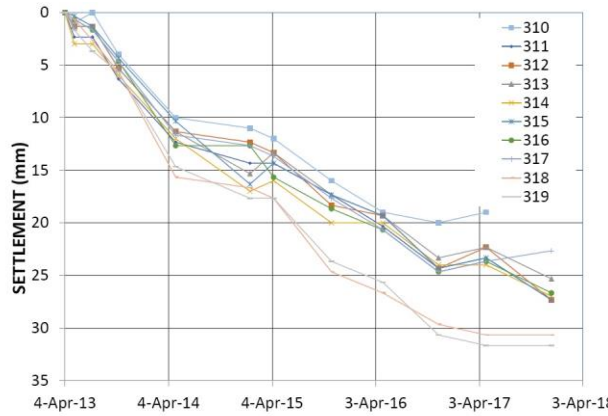

Figure 8 – Black Creek Road (West Approach) Settlement

The settlement data indicates long term embankment settlements up to 20 mm at the east abutment and 30 mm at the west abutment over a 4.5-year period. The magnitudes of the monitored settlements include both embankment fill compression and secondary consolidation of the foundation. It is worth noting that the settlements behind the west abutment, where 2m surcharge and 3m EPS were used, have shown signs of stabilization since late 2016.

The approach settlements were generally lower further back from the abutments. At the east approach, the settlement decreases consistently towards the east as the clay pinches out in thickness. At the west approach, the settlement pattern is likely associated with higher over-consolidation achieved in the foundation clay under the 2D stress conditions further back from the abutment.

5.2 Pre- and Post-Construction Soil Properties

Field and laboratory testing data gathered from the original investigation program and the post-construction program was compiled and reduced to enable a parametric comparison of the clay properties before and after the embankment construction.

CPTU Results

Three CPTU soundings (CPTU2, CPTU3 and CPTU5) conducted in the approach embankment areas during the original program were used in comparison with the three more recent CPTU soundings (CPTU12-1, CPTU12-2 and CPTU12-3) conducted in the post-construction program.

CPT data interpretation methodology and equations proposed in *Lunne, Robertson and Powell (1997)* were used as a guide in reducing the raw data and in correlating engineering parameters in combination with some recently developed equations published in *Robertson (2010)*.

Figure 9 shows increase in uncorrected tip resistance (q_c) and sleeve friction (f_s) in the foundation clay in the west approach area behind the abutment. The average uncorrected tip resistance between Elevations 308m and 295m has increased from 700kPa to 1900kPa, suggesting much higher undrained shear strength (S_u) and pre-consolidation pressure (P_c') after construction. Similarly, the average sleeve friction at the same elevations has increased substantially from about 8kPa to 50kPa, suggesting much lower sensitivity likely due to lower liquidity indices (LI) following consolidation.

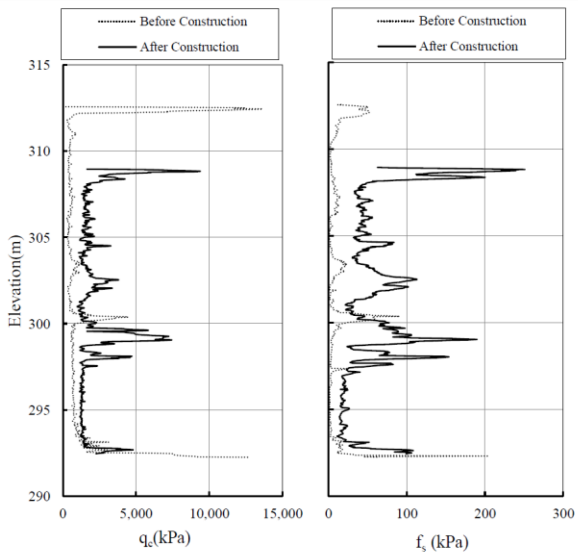

Figure 9 – CPTU (West Abutment)

Post-construction CPTU data clearly illustrated foundation settlement at depth as evidenced by downward shifts in elevations of the stiffer silty interlayers (i.e. spikes in both q_c and f_s plots). In addition, the number of stiffer interlayers appear to have increased after consolidation potentially associated with more pronounced strengthening realized in the higher silt content zones.

Undrained Strength and Pre-consolidation Pressure

Selected engineering parameters from the CPTU data interpretation were further calibrated by field and lab test data, including undisturbed and remoulded S_u and P_c'.

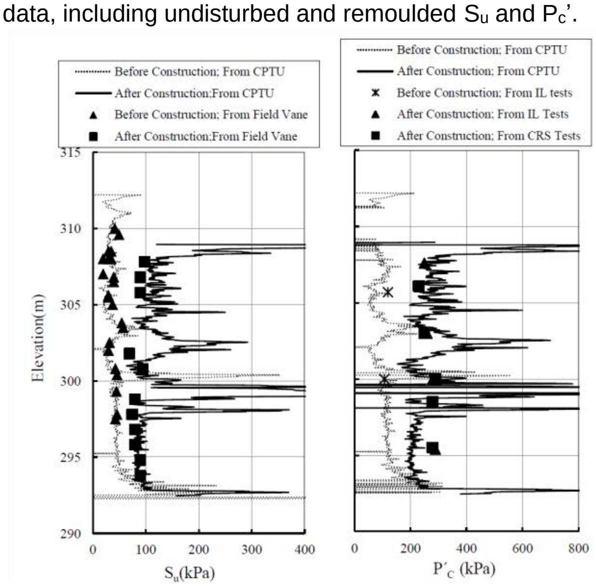

Figure 10 – Shear Strength (East Abutment)

Figure 10 illustrates increase in S_u and P_c' in the foundation clay in the east approach area behind the abutment. The CPTU profiles were calibrated against field vane strengths and IL/CRS consolidation test results.

Figure 10 indicates a significant jump in average S_u between Elevations 308m and 295m from approximately 50kPa to 100kPa. In the meantime, the average P_c' at the same elevations has increased from 100-150 kPa to 200-300 kPa. Significant hardening near the top of the clay deposit was evident while the shear strengths in the lower portion of the deposit have hardly increased. The percentage increase in shear strengths generally trends lower with depth.

Pre- and Post-Construction Compressibility

One-dimensional consolidation tests were performed on the undisturbed Shelby tube samples collected in the original program and the post-construction program. The pre-construction samples had been subjected to IL tests only, while the post-construction samples were tested using both IL and CRS procedures.

Figures 11 and 12 show graphical comparison of the pre-construction and post-construction consolidation curves of the samples recovered from similar depths within the foundation clay.

The values of P_c' were interpreted based on a combination of in-situ overburden pressure (σ'_{vo}), S_u, and interpretative methods including Casagrande (1936) and strain energy density method.

Figure 11 shows that the post-construction consolidation tests started at much lower initial void ratios. However, the recompression, normal compression and unloading behaviour of the pre- and post-construction samples resembles each other, which is characteristic of unstructured glacio-lacustrine clays. Clay contents of the tested clay samples ranged between 20% and 30% and plasticity indices between 10% and 15%.

Leroueil (1983) indicated that the strain rates at the end of each loading increment in a 24-hr oedometer test typically range from 2E-8 to 1E-6 per minute. Qu et al.

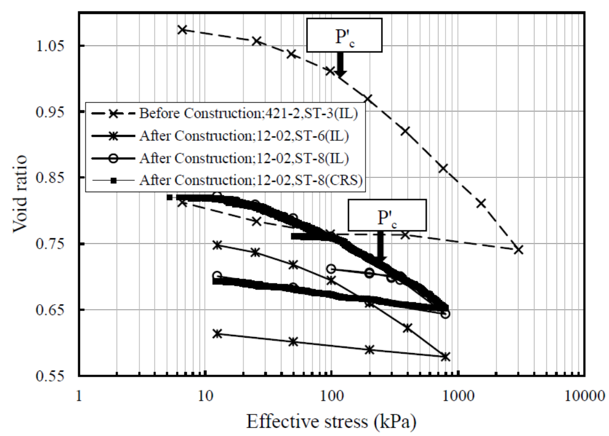

Figure 11 – Consolidation Tests
(West Abutment 12m &15m depth)

(2014) suggested that the 24-hr strain rate may be estimated using equation [1]:

$$\dot{\varepsilon}_{oed}^{vp} = \frac{C_{\alpha e}}{\ln(10)\,(1+e_0)} \times \frac{1}{24hr} \qquad [1]$$

$C_{\alpha e}$ (or C_α) of the oedometer samples ranged typically between 0.003 and 0.004 in the current study. The average 24-hr strain rates of the IL tests shown on Figure 11 are estimated to be approximately 5E-7 to 7E-7 per minute. The CRS test was conducted at strain rates ranging from 1E-5 to 1E-4 per minute with an average of 3E-5, which is about 50 times faster than the average 24-hr strain rate. The consolidation curves of IL and CRS tests performed on the samples trimmed from the same Shelby tube indicated identical stress-strain behaviour, suggesting very low rate sensitivity of the clay samples. Further study will be required to confirm this observation of the subject clay.

IL consolidation tests performed on the undisturbed samples collected at 10m depth in the east abutment area are illustrated in Figure 12. Similar to the samples from the west abutment area, the consolidation behaviours of the pre- and post-construction samples are fairly repeatable in terms of the values of C_c and C_r. The post-construction sample shows lower compressibility at stress levels near P_c', most likely due to the effect of consolidation.

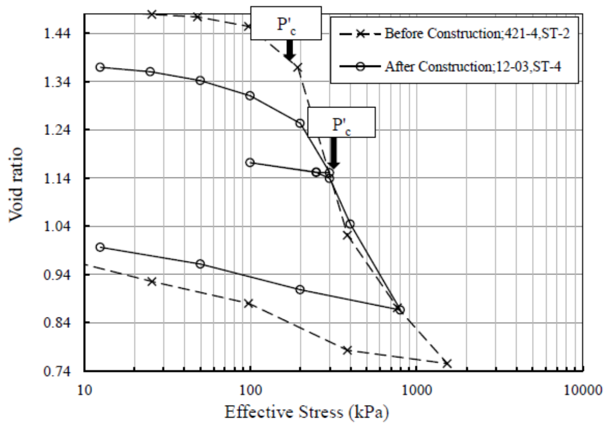

Figure 12 – Consolidation Tests
(East Abutment 10m depth)

Normal compression indices (C_c) interpreted from IL and CRS consolidation tests performed on the undisturbed clay samples collected from both approach embankment areas and various depths were plotted against initial void ratios (e_0) as shown on Figure 13.

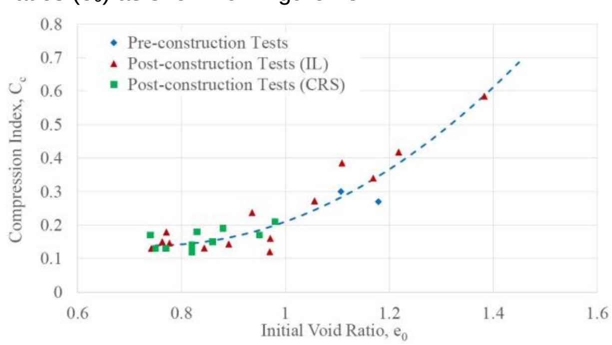

Figure 13 – Compression Index vs. Initial Void Ratio

The plot reveals that the compression index stays relatively constant under an initial void ratio of 0.9 to 1.0 and then increases essentially linearly at higher initial void ratios. The increase in compressibility with void ratio is believed to be a function of clay fraction.

As one of the main objectives of current study, the coefficient of secondary consolidation (C_α) was closely examined with respect to the loading stress and in relation to the compression index.

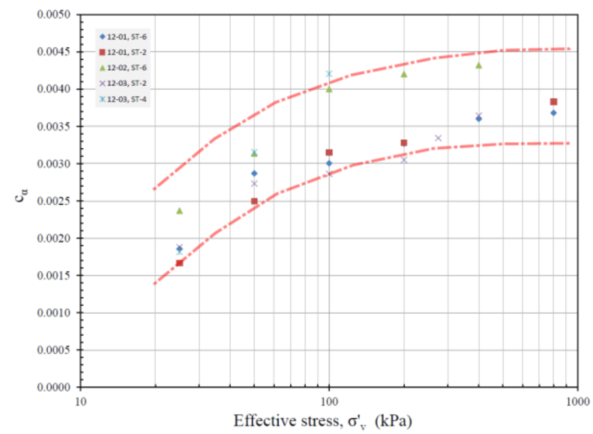

Figure 14 – Cα vs. Vertical Stress σ'_{v0}

For each individual load increment in an IL consolidation test or at a pre-selected stress level in a CRS consolidation test, C_α value was interpreted in *e-log(t)* space following the end of primary consolidation (EOP). All C_α values were then plotted against corresponding effective stresses as shown on Figure 14. Most of the data points were generally contained within a narrow band of 0.001 to 0.0012 in difference between the upper bound and lower bound C_α values.

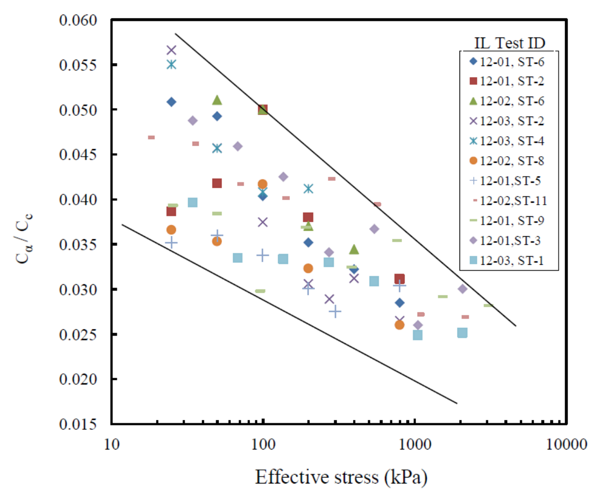

Figure 15 – C_α/C_c (C_α/C_r) vs. Vertical Stress σ'_{v0}

The trend of the data band shows that the C_α value increases quickly on the recompression branch and plateaus into the virgin compression line, mirroring the C_r to C_c transition in the *e-log(p')* space. This observation suggests that it might be overly conservative to assume a constant C_α/C_c ratio in predicting secondary consolidation settlement.

Further, C_α/C_r and C_α/C_c ratios were assessed in both recompression zone and virgin compression zone. The ratios were again plotted with respective effective stresses at which C_c (or C_r) and C_α were estimated. Figure 15 depicts the trend of C_α/C_r or C_α/C_c ratios with the effective stress and indicates that the ratio is a function of effective stress rather than a constant and generally decreases with increasing stress.

The degree of scatter in the C_α/C_r or C_α/C_c ratios tends to decrease at higher stress levels. The scatter is also found to be wider in the over-consolidated range than in the normally consolidated range, which is potentially related to the variation of over-consolidation ratios (OCR), and the clay plasticity as influenced by mineralogy and clay fraction.

6 SUMMARY AND CONCLUSIONS

Embankments up to 12m (final grade plus surcharge) in height were built over 35m deep soft to firm clay with the use of wick drains, stabilizing berms and staged construction to maintain foundation stability during construction. Surcharging and substitution of conventional fill by EPS blocks were part of the geotechnical design to mitigate long-term embankment settlements. In the current study, the effects of these mitigative measures on the foundation clay properties are examined to potentially facilitate more efficient and optimal design of similar highway projects in the future.

Through a comparison of pre- and post-construction clay properties, the following observations were made:

1) Undrained shear strengths of the foundation clay have increased significantly after consolidation under the embankment loading. An average S_u'/σ'_v ratio of 0.25 was used to predict the strength gain in the clay with dissipation of excess pore pressure. An average increase of 200kPa in vertical effective stress due to embankment loading was estimated in the foundation clay between Elevation 308m and 295m, suggesting an average S_u increase of 50kPa. This agrees well with the post-construction field vane and CPTU data.
2) The average pre-consolidation pressure in the foundation clay between Elevation 308m and 295m has increased from 100-150kPa to 200-300kPa. The magnitudes of increase in P_c' are generally in agreement with the stress difference between the initial over-consolidation and the effective stress increase in the clay under the maximum embankment loading. The percentage increase generally trends lower with depth.
3) Stress- and time-dependent compressibility of the clay, i.e. C_c, C_r and C_α, remain essentially unchanged before and after construction. This observation is generally in line with the characteristics of an unstructured clay.
4) At an initial void ratio of 1.0 or lower, the subject clay tends to maintain a practically constant compressibility in response to stress change. The clay compressibility increases approximately linearly at initial void ratios greater than 1.0. It is postulated that the silt fraction governs the compressibility at low void ratios while at high void ratios the clay fraction takes the control.
5) Coefficient of secondary compression C_α tends to increase with effective stress and seemingly approaches an asymptotic value at higher stress levels.
6) C_α is usually taken as a constant percentage of C_c in the geotechnical design. The current study suggests that the ratio of C_α to C_c (or C_r) may decrease with increasing effective stress.

A close examination of the findings related to coefficient of secondary compression implies that the current geotechnical practice in assuming a constant C_α/C_c ratio irrespective of the range of working stresses may be somewhat conservative in predicting long-term settlement of embankments on over-consolidated clays.

A primary objective of this post-construction research program is to advance the understanding and future design of the long-term performance of embankments on compressible clay foundations by studying rheological behaviour (i.e. creep or secondary compression) or strain rate dependency of subject clay with application of advanced consolidation devices like CRS apparatus. Further studies including in-depth numerical modelling of the long-term foundation deformation under constant embankment loading could be conducted within a constitutive framework integrated with rheological model of the subject clay.

References

Lunne, T., Robertson, P.K. and Powell, J. J. M. 1997. Cone Penetration Testing in Geotechnical Practice, Blackie Academic and Professional.

Robertson, P.K. and Cabal, K.L. 2010. Guide to Cone Penetration Testing for Geotechnical Engineering

Sully J.P., Robertson P.K., Campanella R.G. and Woeller D.J. 1999. An Approach to Evaluation of Field CPTU Dissipation Data in Overconsolidated Fine-grained Soils. CGJ, Vol. 36: 369-381

Mayne, P.W. 2007. NCHRP Project 20-05, Topic 37-14, Cone Penetration Testing State-of-Practice

Mayne, P.W. and Mitchell, J.K. 1987. Profiling of Overconsolidation Ratio in Clays by Field Vane. CGJ, Vol. 25: 150-157

Mesri, G. and Feng, T.W. 1991. Surcharging to Reduce Secondary Settlements, Geo-Coast 1991, Yokohama, Japan, 3/14: 359-364

Hansbo, S. 1960. Consolidation of Clay with Special Reference to Influence of Vertical Sand Drains. Swedish Geotechnical Institute, Proceedings No. 18

Hansbo, S. 1979. Consolidation of Clay using Band-shaped Prefabricated Drains. Ground Engineering, 12(5): 16-25

Barron, R.A. 1947. Consolidation of Fine-grained Soils by Drain Wells. ASCE, 113: 718-742

Mesri, G., 1973. Coefficient of Secondary Compression. Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 99, SM 1, 123-137.

Mesri, G. and Godlewski, P.M., 1977. Time and Stress Compressibility Interrelationship. Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 103, GT 5, 417-430.

Boone, S.J. 2010. A Critical Reappraisal of "Preconsolidation Pressure" Interpretation Using the Oedometer Test. CGJ, Vol. 47:281-296

Qu, G., Hinchberger, S., Shi, K. and Dittrich P. 2014. An EVP Approach for Embankment Surcharge Design. CGC Proceedings GeoRegina 2014, Paper #232

Leroueil, S. 1987. Tenth Canadian Geotechnical Colloquium: Recent Developments in Consolidation of Natural Clays. CGJ, Vol. 25: 85-107

Watabe Y., Udaka K., Nakatani Y. and Leroueil S. 2012. Long-term Consolidation Behaviour Interpreted with Isotache Concept for Worldwide Clays. The Japanese Geotechnical Society, Soils and Foundations, 52(3): 449-464

Hamouche K.K., Leroueil S., Roy M. and Lutenegger A.J. 1995. In situ Evaluation of K_0 in Eastern Canada Clays. CGJ, Vol. 32: 677-688