A Case Study for Ground Improvement using Surcharge and Wick Drains for the Canadian Port-of-Entry Building Foundations of Gordie Howe International Bridge Project

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ABSTRACT
The Gordie Howe International Bridge project is one of the largest public-private partnership (P3) infrastructure projects in North America. The site lies within the physiographic region of Southwestern Ontario and the Eastern Detroit Lowland known as the St. Clair Clay Plains. The site contains more than 20 m of soft soil comprising lake-deposited lacustrine, laminated and varved silty clay deposits overlying glacial till, and limestone and dolostone bedrock. Ground improvement using surcharge and wick drains has been used to consolidate the soft clay deposit to meet the stringent 100-year design requirements for the building foundations. This paper presents a detailed case study of the ground improvement design considerations, including field investigations and laboratory testing to characterize the soil properties, and the development of the design requirements for the wick drains and surcharge thickness and duration. A geotechnical instrumentation and monitoring plan was developed to provide the basis for collecting the detailed in-situ monitoring data required for the back-analysis to confirm that the end of the surcharge period had been reached. The post surcharge investigation including Cone Penetration Testing, borehole investigations, and laboratory tests to verify the settlement design criteria had been met prior to constructing the foundations. This case study provides insights into the foundation design approach used for this P3 infrastructure project.

RÉSUMÉ
Le projet du pont international Gordie Howe est l'un des plus importants projets d'infrastructure en partenariat public-privé (P3) en Amérique du Nord. Le site se trouve dans la région physiographique du sud-ouest de l'Ontario et des basses terres de l'est de Détroit connues sous le nom de plaines argileuses de St. Clair. Le site contient plus de 20 m de sol meuble composé de dépôts lacustres lacustres, de dépôts d'argile limoneuse stratifiés et varvés recouvrant du till glaciaire et un substrat rocheux de calcaire et de dolomie. L'amélioration du sol à l'aide de surcharges et de drains à mèche a été utilisée pour consolider le dépôt d'argile molle afin de répondre aux exigences de conception strictes de 100 ans pour les fondations du bâtiment. Cet article présente une étude de cas détaillée des considérations de conception d'amélioration du sol, y compris des enquêtes sur le terrain et des essais en laboratoire pour caractériser les propriétés du sol, et le développement des exigences de conception pour les drains à mèche et l'épaisseur et la durée de la surcharge. Un plan d'instrumentation et de surveillance géotechnique a été élaboré pour fournir la base de la collecte des données de surveillance in situ détaillées nécessaires à la rétroanalyse pour confirmer que la fin de la période de surcharge avait été atteinte. L'enquête post-surcharge, y compris les tests de pénétration au cône, les enquêtes sur les trous de forage et les tests en laboratoire pour vérifier que les critères de conception des tassements avaient été respectés avant la construction des fondations. Cette étude de cas donne un aperçu de l'approche de conception des fondations utilisée pour les grands projets d'infrastructure en PPP.

1 INTRODUCTION

The Gordie Howe International Bridge is a new bridge across the Detroit River to connect Windsor, Ontario, and Detroit, Michigan, as seen in Figure 1. The project is currently under construction, and it includes four components: the Main Bridge and Approach Spans, Canadian Port of Entry (POE), US POE, and Michigan Interchange.

The Canadian POE has a footprint of about 0.526 km^2 (130 acres). It includes the inbound border inspection facilities for passenger and commercial vehicles, outbound inspection facilities, maintenance facilities, and toll collection facilities. The Canadian POE contains eleven (11) structures including six (6) buildings, i.e., the Main Building, Primary Inspection Booths and Canopy, Secondary/Tertiary Traffic Examination Building, Client Processing Centre Building, Large Scale Imaging Building, Toll Service Operations Center and Toll Booths, and Maintenance Building, Salt Barn, and Hazmat Canopy.

The site is within the physiographic regions of Southwestern Ontario and the Eastern Detroit Lowland, known as the St. Clair Clay Plains. The unconsolidated sediments above the bedrock were formed during the late Wisconsinan stage of glaciation (Dreimanis and Karrow, 1972). The overburden comprises glaciolacustrine and glacial till and till-like soils in the Windsor and Detroit metropolitan area (Hedec, 1998). The site is generally flat

with little natural relief in Windsor (Chapman and Putnam, 1984).

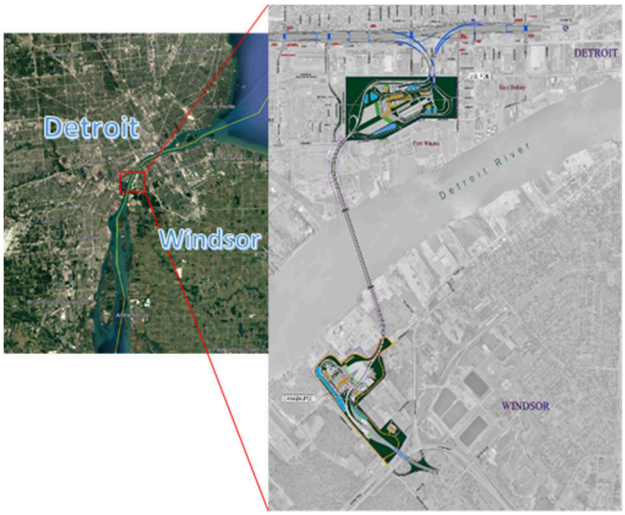

Figure 1. Site Location for Gordie Howe Internation Bridge

The soil at the site contains a layer of fill overlying a grey clay deposit, which rests directly on the dolomitic limestone bedrock of the Devonian Detroit River Group. The thickness of the clay ranges from 21 to 25m. Silt pockets and sandy silt to silty sand lenses with varying thicknesses are present within this deposit.

In some areas, a relatively thin stratum (1 to 2 m thickness) of dense or hard glacial till deposit overlies the bedrock. The glacial till layer is locally termed 'hardpan' in the Detroit area. Artesian groundwater pressures are often encountered within the glacial till layer or the shallow fractured bedrock, and it contains elevated levels of hydrogen sulfide (H_2S) and methane (CH_4) gasses.

From conventional consolidation theory, it is well known that significant settlement will occur when additional loading is applied to the thick clay deposit (Terzaghi et al. 1996). Following an option assessment at the tender stage, a number of options were considered to mitigate this settlement, including the use of deep pile foundations, sub excavation and replacement of soil with light-weight fill, and ground improvement using wick drains and surcharge. Given that the design of the Canadian POE included an overall grade raise of several metres, the preferred option selected for the building structures was ground improvement with wick drains and surcharge, as this also addressed the requirement to improve the subgrade conditions for the pavement structures. This paper focuses ground improvement design of the Main Building.

2 GEOTECHNICAL INVESTIGATION AND SUBSURFACE SOIL CONDITION

2.1 Geotechnical Investigation

An extensive geotechnical investigation was carried out prior to the ground improvement detailed design for the entire Canadian POE. In summary, the investigation program consisted of 141 geotechnical boreholes and 49 cone penetration test (CPT) holes. Standard penetration tests (SPT) and field vane tests were conducted in the boreholes. Undisturbed samples were collected using thin-wall Shelby tubes and samples were selected for advanced laboratory tests including 1-D consolidation, creep, triaxial, and unconfined compression tests. A total of 58 vibrating wire piezometers (VWP) and 38 monitoring wells were installed in selected geotechnical boreholes to collect the site hydrogeological information, which was used in the detailed design.

2.2 Subsurface Condition

The historical land use within much of the site was residential lands. During the early site preparation work, the topsoil was stripped and replaced with sand and gravel fill to a depth of about 0.8m. The general ground surface elevation within the footprint of the Main Building at the time of the investigation was about 177.8mASL. A thick clay deposit was encountered below the granular fill typically comprising a layer of overconsolidated silty clay (CL-ML) at the surface of the clay deposit gradually transitioning to lean clay (CL) with increasing depth. A fat clay (CH) was occasionally encountered within the upper lean clay portion from the elevation of about 174 mASL metres above sea level (mASL) to 165 mASL.

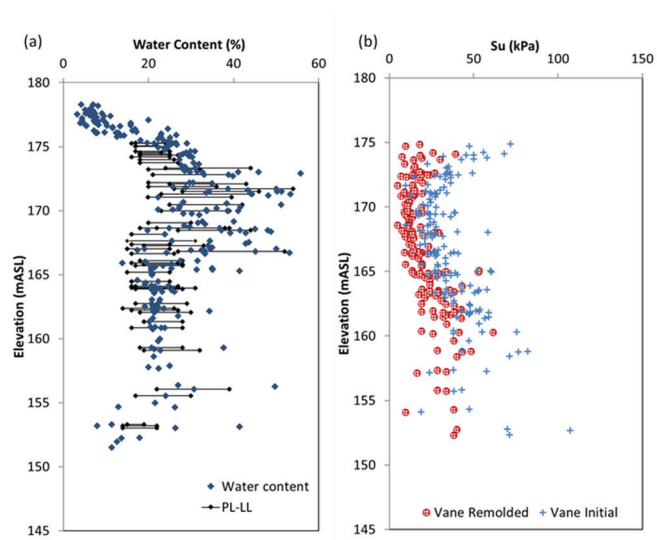

Figure 2. (a) water content and Atterberg limits vs. elevation at Main Building; (b) in situ vane test results vs. elevation at Main Building

Figure 2(a) shows the water content and Atterberg limits of the soil samples collected in the Main Building area. The clay deposit was initially divided into three different layers for the purpose of settlement analysis, i.e., desiccated clay, upper clay, and lower clay. Later following further investigation and testing, the upper clay was further divided into two sublayers, i.e., slightly over consolidated upper clay and normally consolidated upper clay, based on the back analysis from additional field monitoring data. The

field vane test results support the decision to sub-divide the upper clay layer, as seen in Figure 2(b).

Figure 3 shows the compression index and swell index plotted against elevation for the clay deposit on-site.

Groundwater levels are one of key design parameters used in the ground improvement and foundation design. A seasonal groundwater level fluctuation was observed on the site. From the 2019 investigation, the groundwater level was between 176.2 to 176.7mASL from February to March. During the early works, the groundwater level was between 172.4 mASL and 173.6mASL. A design groundwater level of 176 mASL was considered to be representative of the current conditions and was selected for the settlement analysis.

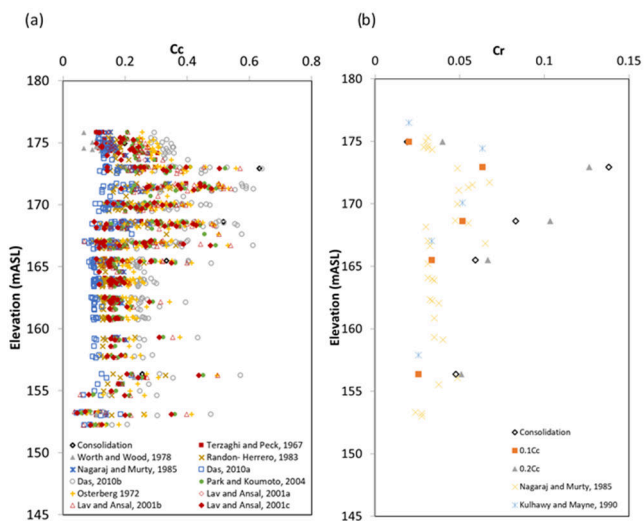

Figure 3. Compression Index and Swell Index for the Clay Deposit

3 GROUND IMPROVEMENT DESIGN

The basic principle of ground improvement using surcharge and wick drain (or prefabricated vertical drains (PVDs)) is to reduce the void ratio of soil by accelerating the consolidation process, thus reducing the compressibility and long-term settlement and increasing the strength of the soils. Wick drains have been used successfully to improve the soft clay (Bergado et al. 2002; Chu et al. 2004; Bo et al. 2005) and swamp and peat (Dittrich et al. 2010; Mesri et al. 2017). Barron (1948) proposed an analytical solution to predict the degree of consolidation resulting from the use of sand drains. Later Hansbo (1981) modified the analytical solution by considering a smear zone and well resistance of wick drains.

3.1 Wick drains

Wick drains are typically installed in two commonly used patterns, i.e., a square or triangular pattern. The design parameters for the wick drains include the ratio of the diameter of smear zone to the diameter of installation mandrel, d_s/d_m, the ratio of undisturbed to smear zone permeability, k_h/k_s, spacing, and the installation depth.

Many laboratory and field studies were carried out to develop the range of design ratios (Dittrich et al. 2010, Han 2015). The d_s/d_m and k_h/k_s ratio varies from 1.5 to 5 and 1 to 10, respectively. The actual value depends on both in-situ installation vibration and the soil behavior.

3.2 Design Considerations and Parameters

Ground improvement is required to consolidate the soft clay deposit to meet the stringent 100-year design requirements for the building foundations at both the ultimate limit state (ULS) and service limit state (SLS). All the building structures were designed to be supported using spread footings and strip footings. The degree of ground improvement was initially selected to provide sufficient strength gain for ULS capacity, and to minimize the post-construction settlement to within 50mm and differential settlement to less than 1/300 rotation for the long term. Based on the back-analysis during the waiting period, which will be discussed later, raft foundations were occasionally required to accommodate the structural loading changes during detailed design.

According to the project agreement, strength gain was one of the ground improvement requirements. The strength increase due to the surcharge was calculated using the following equation for normally consolidated clay (Skempton 1957) to estimate the global slope stability of the temporary staged construction conditions and ULS capacity:

$$\Delta S_u = (0.11 + 0.0037 I_p)\Delta\sigma_z \quad [1]$$

It was determined that there would be less significant strength gain at the desiccated clay layer due to the high over-consolidation ratio (OCR ≥ 4). Therefore, no change in the undrained shear strength was assumed for the desiccated clay layer to determine the ULS capacity and the global stability during the construction period. The strength gain of the upper and lower clay was only considered in the design. Also, permanent grade raises are needed at all building locations based on the site grading requirements. Therefore, engineered fill was added on-site to increase the bearing capacity at ULS. For this reason, the bearing capacity at SLS, which is settlement governed, is more critical than the capacity at ULS.

For the wick drain design, a sufficient thickness of clay barrier was required between the wick drain tips and the artesian groundwater zone (152.3mASL) to prevent artesian groundwater from entering the wicks and coming

to the surface. The wick drain parameters used for the initial design are shown in Table 1. The d_s/d_m and k_h/k_s ratio was updated in the later construction stage based on additional investigations and back-analysis results.

Table 1. Wick Drain Design Parameters

Parameters	Value
Installation Pattern	Triangular
Installation Tip Elevation (mASL)	156 (Main Building Area) 157(Maintenance Building) 158 (All other places)
Spacing (m)	1.0 m (Main Building Area) 1.5m (All other places)
d_s/d_m	3 for original design
k_h/k_s	3 for original design

3.3 Consolidation Analysis and Surcharge Design

The surcharge height is a critical design factor for the ground improvement program. It was determined based on the allowable post-construction settlement of the building structures. The combination of wick drain numbers /spacing and surcharge height should provide sufficient consolidation to the clay deposit within the scheduled surcharge period to meet the post-construction settlement criteria. The surcharge design for the Main Building is presented in the following sections.

The settlement analysis was carried out using Settle3D software by Rocscience inc. Consolidation tests were conducted on the selected Shelby tube samples to obtain the initial design parameters. The design parameters for the settlement analysis are shown in Table 2 for the Main Building.

Table 2. Analysis Parameters at Main Building

Soil Type	Parameter	Values for Main Building
Existing Fill	Layer Thickness	Average 1.2 m
	E_s	62500 kPa
	γ	20.5 kN/m^3
	γ_{sat}	21 kN/m^3
Desiccated Clay	Layer Thickness	Approximately 3 m
	γ	19.0 kN/m^3
	C_c	0.199 (top) to 0.5 (bottom)
	C_r	0.018 (top) to 0.1 (bottom)
	OCR	4 (top) to 1 (bottom)
	e_0	0.5 (top) to 1 (bottom)
	C_v	0.001(cm^2/s)
	C_h:C_v ratio	2
Upper Clay	Layer Thickness	Approximately 10.2 m
	γ	18.0 kN/m^3
	C_c	0.5 (top) to 0.255 (bottom)
	C_r	0.1 (top) to 0.03 (bottom)
	OCR	1
	e_0	1 (top) to 0.58 (bottom)
	C_v	0.001(cm^2/s)
	C_h:C_v ratio	2
	C_a/Cc	0.03 (original ground); 0.01(post-surcharge ground)
Lower Clay	Layer Thickness	Approximately 9.2 m
	γ	21.0 kN/m^3
	C_c	0.255
	C_r	0.04
	OCR	1
	e_0	0.56 (top) to 0.64 (bottom)
	C_v	0.002 (cm^2/s)
	C_h:C_v ratio	3
	C_α/Cc	0.03 (original ground); 0.01(post-surcharge ground)
Lower Silt Layer	Layer Thickness	Approximately 1.2m
	γ	20.0 kN/m^3
	C_c	0.15
	C_r	0.015
	OCR	3.5
	e_0	0.6

List of symbols:
E_s-elastic modulus; γ -bulk unit weight; γ_{sat}-saturated unit weight C_c - compression index; C_r-recompression index; OCR-over consolidation ratio; e_0-void ratio, C_v- vertical coefficient of consolidation, C_h-Horizontal coefficient of consolidation; C_a-secondary compression index.

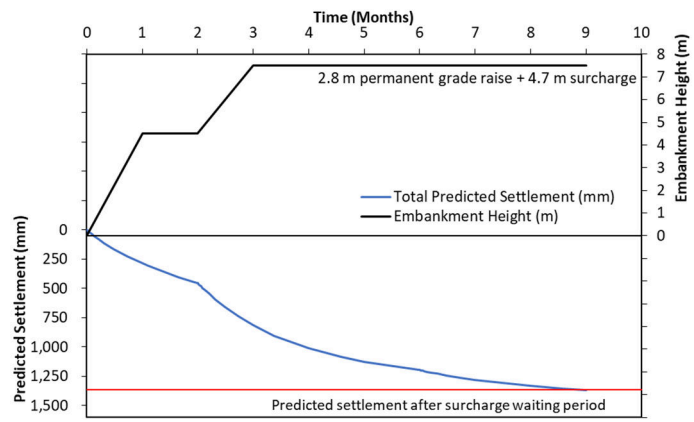

Figure 4. Settlement Prediction with Time for Staged Construction of the Main Building Embankment

From the initial settlement analysis, a surcharge height of 7.5m (2.8 permanent grade raise +4.7m surcharge) within the Main Building area was recommended. The top of the embankment was designed to be 185.3mASL. Based on the settlement model, the fill would need to be compacted to a minimum unit weight of 20.3 kN/m^3. A surcharge period of 6 months was required. That is, 6 months after completing the embankment construction, the surcharge could be removed to the permanent grade. From the stability analysis, a two-stage construction sequence was required to maintain minimum global safety (i.e, FOS >1.3). The required waiting period between the two stages was a minimum of 45 days to allow for sufficient strength gain as the clay consolidated, which was estimated based on Eq.1. The predicted settlement is shown in Figure 4, with the predicted maximum settlement at the end of the surcharge period of 1360mm.

The amount of settlement is one of the key factors monitored on-site as a criterion to determine the progress of consolidation and the end of the surcharge.

3.4 Geotechnical Instrumentation and Monitoring Plan

A geotechnical instrumentation and monitoring plan is very important for monitoring the consolidation progress within the building footprint, protecting any existing adjacent structures or utilities from the proposed surcharge consolidation, and ensuring the stability of embankment fills. The geotechnical instrumentation used on-site includes:

- settlement plates (SP) to measure the ground settlement;
- vibrating wire piezometers (VWP) to measure porewater pressure, and;
- slope inclinometers (SI) to measure the horizontal movement of the ground.

Figure 5 shows the plan view of the geotechnical instrumentation plan within the Main Building area

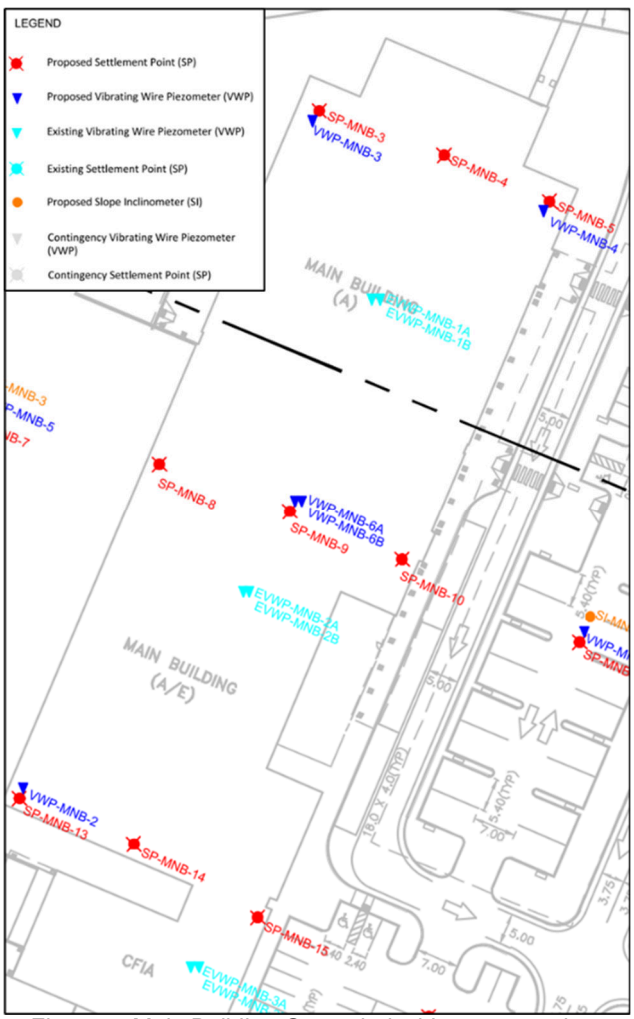

Figure 5. Main Building Geotechnical Instrumentation Plan

4 BACK ANALYSIS AND POST-SURCHARGE INVESTIGATION

To refine the foundation design and assess the requirements of the design changes during the surcharge consolidation period, the monitoring data were used to back-analyze the design parameters. The foundation options and bearing capacities were also updated based on the back-analysis of the field monitoring data.

4.1 Field Monitoring and Settlement Analysis

The Main Building embankment construction commenced on December 03, 2019 and it was completed to 185.0 mASL on March 23, 2020. In the initial design, the engineered fill for the permanent grade raise and surcharge fill was designed to be 20.3 kN/m^3. However, the as-built unit weight of the fill was 18.9 kN/m^3 and 18.7 kN/m^3 for engineered fill and surcharge fill, respectively. Therefore, an additional compensation fill was added and completed on June 10, 2020, to bring the embankment top elevation to 186.0 mASL. Furthermore, an additional 'cap' fill was required at the south portion of the Main Building Embankment to accelerate the consolidation process and reduce the predicted post-construction differential settlement. This cap was completed on August 13, 2020, and the final height of the embankment top at the south portion of the Main Building was 187.65mASL.

A global stability analysis was carried out for a section through the south slope of the Main Building surcharge area using the estimated strength gain calculated using Eq. 1. The final height of the embankment was 9.85m and the overall global factor of safety satisfied the project requirement. From this analysis, it was determined that a prolonged waiting period (48 hours) was required between each fill lift, and the frequency of monitoring was also increased.

Overall, the total consolidation period was extended to 12 months from the start of the embankment construction to removal of the surcharge, instead of the 9 months that was estimated in the original design.

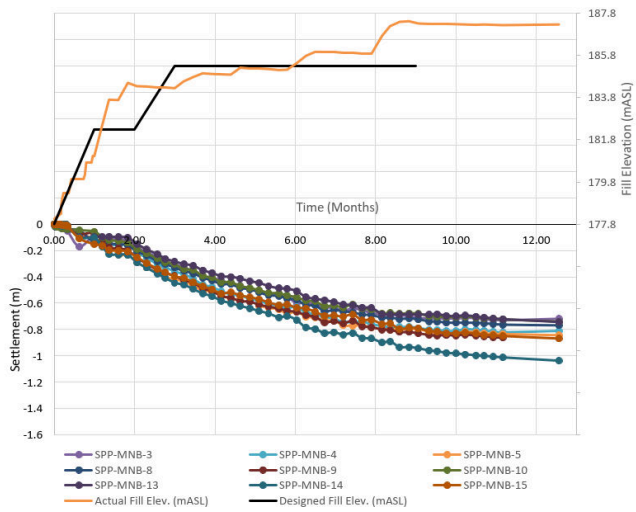

Figure 6. Surcharge Height and Settlement Monitoring Data

Figure 6 shows the actual fill height and settlement monitoring data obtained during construction. It is noted that a maximum settlement of 1050mm was observed on site, which is less than the maximum settlement estimated in the original design (1360mm).

Table 3. Updated Design Parameters in Back-analysis

Design Parameters	Initial Design	Back-analyses
GWT	176 mASL	177 mASL
d_s/d_m	3	4
k_h/k_s	3	4
OCR (Desiccated Clay)	4 (top) to 1 (bottom)	10 (top) to 4 (bottom)
OCR (Upper clay)	1	• 4 (top) to 1.3(bottom) for overconsolidated upper clay • 1 for normally consolidated upper clay
C_h:C_v Ratio	2 (upper clay) 3 (lower clay)	1 (upper clay) 2 (lower clay)

Therefore, a back-analysis was carried out to adjust the original design parameters. The updated parameters are shown in Table 3. During the consolidation period, it was found that the groundwater table rose to an elevation of 177 mASL, as seen in Figure 7. The ratio of d_s/d_m and k_h/k_s increased from 3 to 4 in the back analysis. Based on the CPT results, the upper clay layer was further divided into two sublayers for the back analysis: slightly overconsolidated clay and normally consolidated clay. The results of the back-analyses are shown in Figure 8.

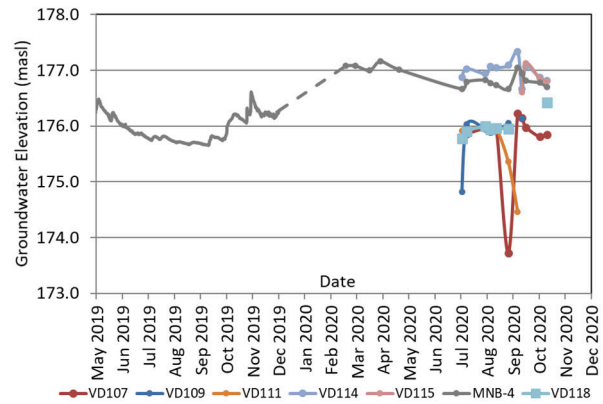

Figure 7. Water Level Data from Monitoring Wells and Vertical Drains

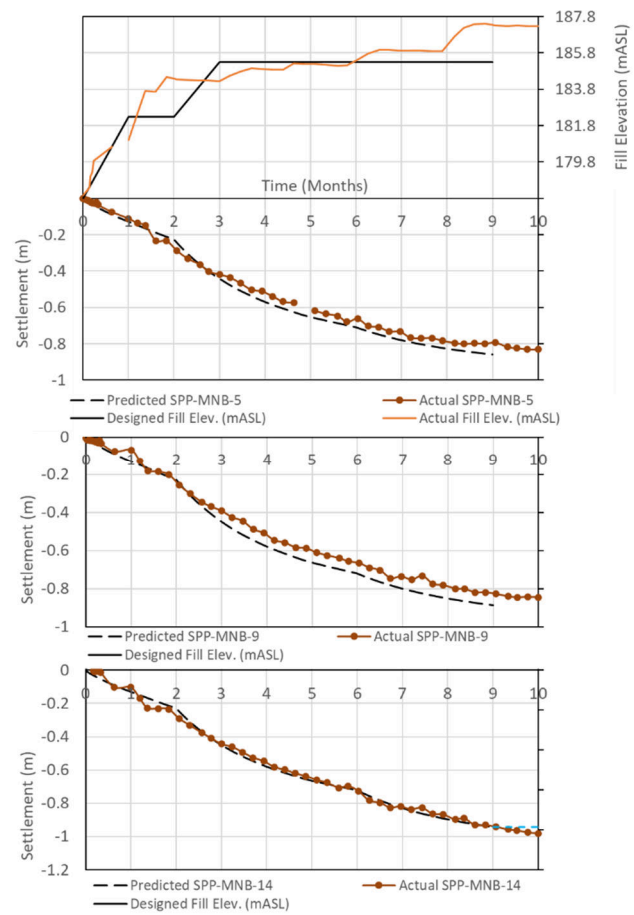

Figure 8 Back-Analysis Prediction at North, Center, and South Part of Embankment

As noted above, the back-analysis prediction more closely followed the general trend of the settlement. The $\sqrt{time}$ Method was used to estimate the degree of consolidation. Under the project construction schedule, this method is considered to be more representative of the

actual response of the ground. From the back-analysis, the consolidation that occurred on-site was over 90% on average, and this was also confirmed based on the VWP monitoring data, as seen in Figure 9. All the VWP data reached either baseline or within 10% of the baseline hydrostatic pressure, except VWP-MNB-5 (installed at the toe of the embankment and possible malfunction) and VWP-MNB-6B (installed at 156mASL and affected by the artesian zone).

4.2 Post-Surcharge Investigation and Surcharge Removal

The Main Building is the largest building within the Canadian POE, and a supplementary in-situ investigation was carried out after surcharge placement, including a CPT and borehole investigation, to evaluate the post-surcharge strength gain in the clay prior to providing recommendations on the timing of the surcharge removal.

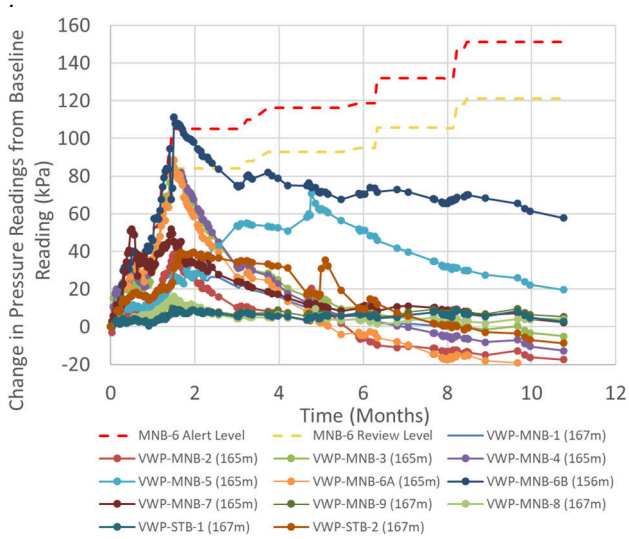

Figure 9 VWP Monitoring Data at Main Building Area

Table 4. Summary of the Investigations for Main Building Surcharge Embankment

Testing	ID	Location	Notes
CPTs	MNB-CPT-1new	North	40% to 100% increase in S_u
	MNB-CPT-3new		
	MNB-CPT-4new	South	Minor to moderate increase
	CPT20-MNB-06		
	CPT20-MNB-07		
	CPT20-MNB-08		
Borehole	MNB-12	South	Significant increase in S_u
	MNB-13		

Table 4 presents a summary of the post-surcharge investigation results and recommendations for the Main Building ground improvement. Six (6) CPTs and two (2) borehole investigations were carried out within the Main Building footprint. The results showed that 40% to 100% increase in undrained shear strength on the north side of the Main Building (MNB-CPT-1 New, MNB-CPT-3 New). A minor to moderate increase in undrained shear strength was observed within the south side of the Main Building (MNB-CPT-4new, CPT20-MNB-06, CPT20-MNB-07, and CPT20-MNB-08). The undrained shear strength, S_u, from the interpretation of CPT tests, field vane tests, and UCS are plotted in Figure 10.

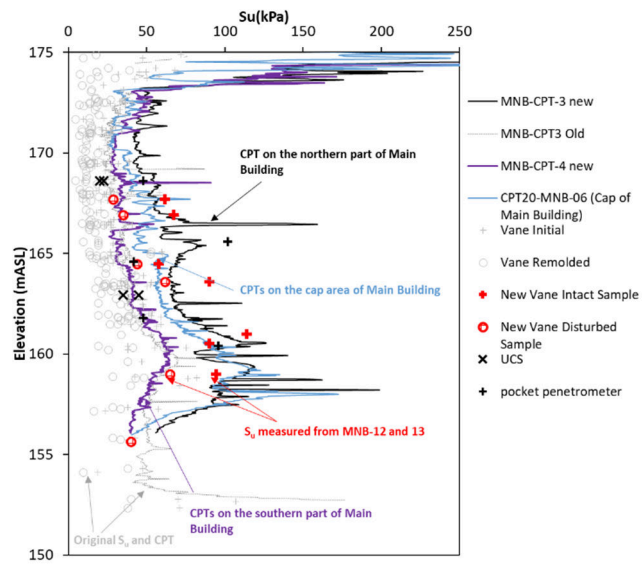

Figure 10 Summary of Undrained Shear Strength from CPT and Borehole Investigations

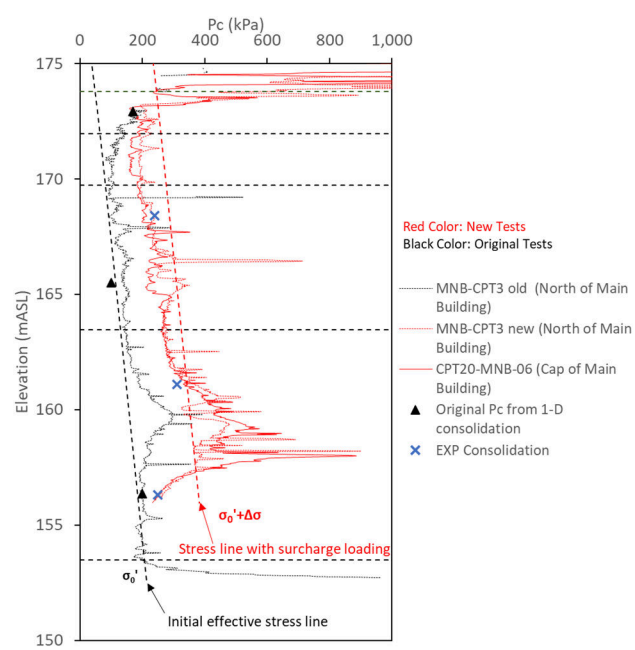

Figure 11 Pre-consolidation P_c from CPTs and 1-D Consolidation

It can be seen that the field vane tests and pocket penetrometer tests indicate a similar strength gain occurred at the south portion of the Main Building as it at the north side. However, the CPT results (CPT-4 new and CPT-MNB-06) show a minor to moderate strength gain, especially at the upper clay level (about 174 mASL to 162 mASL). Therefore, two (2) boreholes were drilled to take undisturbed samples, and three (3) consolidation tests were carried out to determine the post-surcharge pre-consolidation pressures. The results are shown in Figure 11 with the estimated pre-consolidation pressure from CPT tests. It is noted that the pre-consolidation pressure, P_c, measured by 1-D consolidation tests, is consistent with the estimated values from CPT20-MNB-06, indicating a sufficient strength gaining due to ground improvement.

In addition, physical samples of the soil column were extruded and inspected by an AECOM geotechnical engineer to verify the strength gain of ground improvement. Figure 12 shows photos of the extruded samples. Some disturbed sand and silt seams were present within the clay deposit, and the clayey zones were observed to have consolidated successfully.

Figure 12 Photos for Sample Inspection

5 FIELD OBSERVATION AND DISCUSSION

5.1 Wick Drain Design Parameters

The ratio of diameter of smear zone to diameter of installation mandrel, d_s/d_m, and the ratio of undisturbed to smear zone permeability, k_h/k_s, are two key parameters required for the design of wick drains. From the back analysis, d_s/d_m and k_h/k_s of 4 was more representative for the clay deposits in Windsor and Detroit area.

5.2 Site Drainage Effects

Good site drainage conditions are required for the wick drain and surcharge to be effective. A dish-like geometry will naturally be formed at the centre of the large surcharge mound area, and accumulated discharged water can cause back pressure to build up and slow down the consolidation progress.

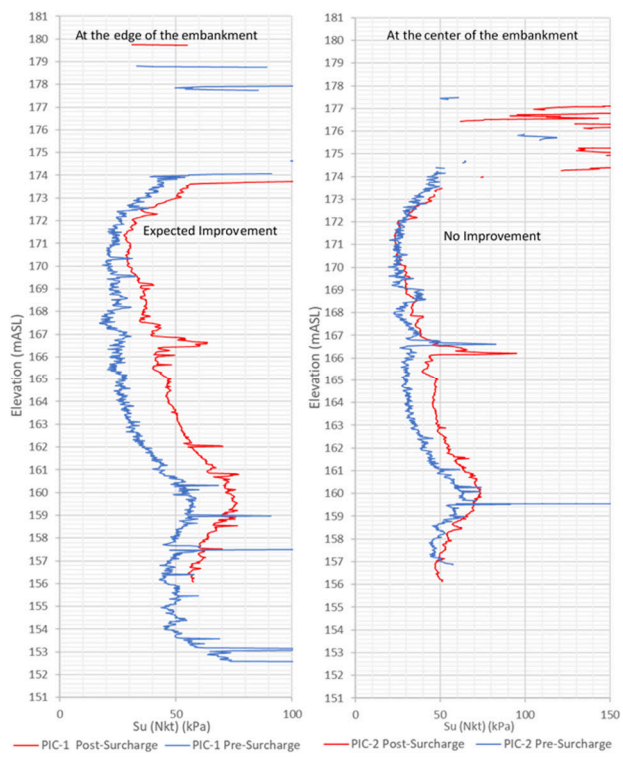

Figure 13 Estimated S_u from the CPTs at the Primary Inspection Canopy Area

Figure 12 shows the post-surcharge CPTs at the Primary Inspection Canopy area. As noted, there is more significant ground improvement taking place at the edge of the embankment compared with the centre. This difference is attributed to back pressure building up at the centre of the surcharge embankment. A CPT (PIC-1 CPT) was carried out near the west edge of the primary inspection canopy embankment, which was at the west boundary of the project site. Another CPT (PIC-2 CPT) was carried at the centre of the embankment, near the center of the project site. Some vertical wells were installed at the centre of the surcharge mound to pump out the trapped water and reduce the back-pressure build-up. It could be concluded that good drainage, including installing vertical wells to prevent back-pressure and/or horizontal drains connecting vertical drains before placing surcharge, is important for

improving consolidation and reducing the surcharge period.

5.3 Back-Analysis for Design Changes

A back-analysis was carried out for every building embankment to confirm that sufficient consolidation had taken place prior to surcharge removal. The structural designs changed several times during the surcharge period at various locations for different buildings. The back-analysis during the surcharge period allowed the project team to make critical adjustments to the foundation designs. For example, some of the spread footings had to be replaced with a raft foundation for several building structures to accommodate the new structural design loading and meet the design requirements for total and differential settlement. Therefore, an interactive design process, including back-analysis, is very important for design-build projects like the Gordie Howe International Bridge project.

6 CONCLUSIONS

This paper presents a case study of the ground improvement design using wick drains and surcharge for the Gordie Howe International Bridge project. The Main Building design was highlighted and discussed in detail. The original design predicted a maximum settlement of 1360mm. However, the settlement plate monitoring shows that the maximum settlement was about 1050mm. According to the field monitoring data, some of the original design parameters, including wick drain smear zone ratios, horizontal to vertical hydraulic conductivity, and soil layer division, were adjusted based on the back-analysis to improve the foundation design. A post-surcharge investigation, including CPT and borehole investigation, was carried out to verify the strength gain achieved by the ground improvement.

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