Ground Improvement - County Rd 2/34 and Hwy 401 Underpass

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ABSTRACT
The design and construction of the approach embankments related to a bridge replacement at the County Rd 2/34 and Highway 401 underpass located in Lancaster, Ontario will be presented in this paper and is demonstrative of the Ministry of Transportation's (MTO) commitment to assess new technologies and applications that support ministry initiatives and operational needs. The existing structure, built in 1963, has been replaced with a four lane, two-span structure with a shift in the alignment immediately to west of the existing alignment with the new embankments overlapping the existing east embankment slopes. The new embankments are about 8 m in height. The new bridge abutments and pier are supported on H-Pile foundations driven to bedrock.

The subsurface conditions consist of a surficial layer of fill and/or topsoil (with some localized peat) underlain by a 9 to 10 m thick compressible and sensitive clay deposit underlain by relatively thin deposits of glacial till and/or sand and gravel over limestone bedrock. The historical embankment loading has led to settlement of the embankments in the order of 1.5 m since the original construction.

This paper describes the engineering process that culminated with the need for a ground modification alternative to mitigate settlement and embankment stability concerns and the preparation of the contract package that included a performance-based Ground Improvement Special Provision with the requirements for the design, construction and monitoring by a specialty Contractor as a design-build package. The Contractor's design included Controlled Modulus Columns (CMC) and wick drains complemented with a monitoring program consisting of multi point borehole extensometers, settlement plates, vibrating wire piezometers and shape accelerometer arrays.

RÉSUMÉ
La conception et la construction des remblais d'approche relatifs au remplacement d'un pont au passage inférieur à l'intersection de la route de comté 2/34 et de l'autoroute 401 situés à Lancaster, en Ontario, démontrent l'engagement du ministère des Transports (MTO) à évaluer les nouvelles technologies et applications qui appuient ses initiatives et ses besoins opérationnels. La structure existante, construite en 1963, sera remplacée par une structure à quatre voies et à deux travées avec un décalage de l'alignement vers l'ouest de la structure existante (immédiatement adjacent à l'alignement existant) avec les nouveaux remblais chevauchant la pente du remblai est existant. Le nouveau remblai élargi à une hauteur d'environ 8 m et une largeur variante de 15 à 20 m à la crête pour accommoder ce décalage. Les nouvelles culées et piliers du pont sont soutenus par des pieux battus de type 'H'.

Les conditions souterraines à l'emplacement de l'alignement proposé du passage inférieur de la route de comté 2/34 consistent en une couche superficielle de remblai et/ou de terre végétale (avec tourbe à certains endroits), reposant sur un épais dépôt d'argile compressible et sensible. L'argile repose sur des dépôts relativement minces de till et/ou de sable et gravier sur un socle rocheux constitué de calcaire. La charge induite par le remblai d'approche existant sur le dépôt d'argile sensible et compressible profond a engendré de très grands tassements depuis la construction originale du remblai. Ces tassements ont mené à un inclinement des culées de l'ouvrage existant vers les remblais d'approche. Pour la composante conception-construction du contrat 2018-4018, l'entrepreneur a sélectionné une combinaison de plusieurs méthodes. Afin de satisfaire aux exigences de performance strictes en matière de tassement et de stabilité globale (statique et sismique) dans le contrat de construction, les colonnes à module contrôlé (CMC) et les drains verticaux ont été utilisés sous les nouveaux remblais d'approche nord et sud.

Cet article décrit le processus d'ingénierie qui a conduit à la nécessité d'une alternative d'amélioration des sols et à la préparation d'un contrat incluant une disposition spéciale d'amélioration du sol basée sur la performance avec les exigences pour la conception, la construction, ainsi que la surveillance et les études avant et après la construction. Le programme de surveillance de l'entrepreneur comprenait des extensomètres de forage sur plusieurs points, des plaques de tassement, des piézomètres à corde vibrante et des réseaux d'accéléromètres de forme. En outre des sujets énumérés ci-haut, cet article traite également de la construction et de la supervision effectuées pendant la construction pour le projet.

1 INTRODUCTION

The presence of a thick layer of clay provided foundation engineering challenges for the approach embankments related to the replacement of the existing County Road 2/34 Bridge over Highway 401. The existing embankment loading over the 9 to 10 m deep sensitive and compressible clay deposit resulted in very large settlements of the embankments since the original construction. These settlements were greatest within 50 m behind the abutments and resulted in the abutments of the existing structure tilting backwards towards the approach embankments.

The new underpass structure is a two-span (40 m and 38.5 m in length) slab-on-girder bridge with integral abutments supported by driven piles approximately 15 m in length driven to bedrock.

The bridge and related approach embankments were constructed under Ministry of Transportation of Ontario (MTO) Contract 2018-4008 and has been operational as of August 2021.

The alignment of the new underpass structure was shifted to the west abutting the existing alignment. The horizontal alignment was selected to permit traffic to be maintained on the existing bridge during construction and presented foundation engineering challenges due to the new embankment loading on the existing embankment. The new embankments required placement of new fill about 8 m in height and about 15 to 20 m in width at the crest.

Embankment settlement and stability analyses were carried out during detailed design to assess the optimum embankment mitigation strategy to ensure embankment stability during construction and to also ensure long term settlement and stability performance following construction.

A suite of alternatives (such as EPS embankment, wick drains and surcharge, rigid inclusions, deep soil mixing) was compared during the detailed design of the approach embankment. Following a comprehensive review of the alternatives and driven by MTO values and a commitment to innovative solutions, the MTO selected ground improvement administered via an alternative contract delivery model for the design and construction of the approach embankments. A design build model was used with performance criteria and a warranty for the approach embankments within a conventional design bid build contract. The Design-Build Contractor chose a combination of Controlled Modulus Columns (CMC) and wick drains to mitigate settlement issues under the new embankments.

2 SITE DESCRIPTION

The existing two-span pile supported bridge carries two lanes of County Road 2/34 traffic over the four-lane Highway 401 near Lancaster, Ontario. The original structure was about 67 m long and 12 m wide. The structure was built in 1963.

The existing approach embankments are approximately 7 to 8 m in height and have side slopes at about 2 horizontal to 1 vertical (2H:1V).

3 SITE INVESTIGATION, SUBSURFACE CONDITIONS AND GEOTECHNICAL DESIGN

3.1 Site Investigation

The investigations for this project were carried out in two phases consisting of an initial investigation for the detailed design and then a subsequent investigation for support of the design-build ground improvement procurement.

The initial detailed design investigation included advancing eleven sampled boreholes at the proposed bridge foundation elements and along the approach embankment realignment. The sampling and in-situ testing consisted of Standard Penetration Tests (SPT's), field shear vane tests and Shelby (Thin-Walled) tube sampling using a piston sampler.

Standpipe monitoring wells were installed in two boreholes for measurement of the groundwater levels following completion of drilling. Index and soil classification tests were carried out on 25% of the selected soil samples. Three one-dimensional consolidation tests were performed on relatively undisturbed soils samples obtained with Shelby tubes.

An additional investigation was carried out to support the ground improvement design and procurement. This investigation included drilling two additional boreholes, as well as carrying out in-situ testing, piezocone penetration test (CPT) and further laboratory testing. Bench scale soil mixing testing was also carried out. The results of that testing are not discussed in this paper but can be found in Afroz et al. (2018).

3.2 Subsurface Conditions

The initial investigation indicated that the subsurface conditions consisted of a surficial layer of fill and/or topsoil with some localized peat underlain by a layer of Champlain Sea clay about 9 to 10 m in thickness as shown in Figure 1. The clay is underlain by relatively thin deposits of glacial till and/or sand and gravel over limestone bedrock.

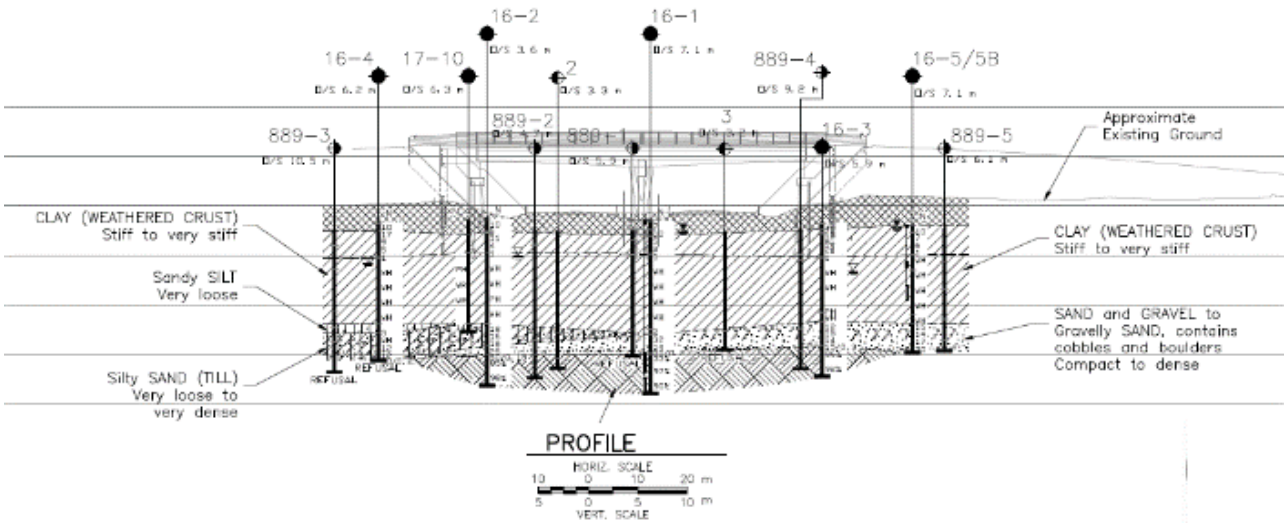

Figure 1. Site stratigraphy along County Road 2/34

The Champlain Sea clay is grey-brown to grey in colour and consists of a 1 to 3 m thick layer of weathered crust underlain by unweathered clay, sensitive and compressible clay. The field vane tests indicated shear strengths ranging from 19 to 65 kPa, indicating a soft to stiff consistency, however the deposit was found to be more generally firm. The results of the laboratory oedometer testing indicated preconsolidation pressures ranging from about 100 to 140 kPa. Water table was noted to be near the ground surface.

3.3 Geotechnical Analysis

Based on the results of the initial investigation and the detailed design geotechnical analyses, it was assessed that the post-paving settlements of the embankment widenings, due to primary consolidation and secondary compression of the clay deposits over a 50-year period, would be about 1000 mm. This would exceed the tolerable limits established by MTO, particularly near the abutments. The new embankment settlements would also be differential to some degree with the existing embankments, which had traffic maintained on them during and after construction posing a potential risk to public safety.

In addition to excessive settlements, the analyses indicated that the factors of safety (FOS) against global instability would be less than acceptable under short term loading conditions and seismic conditions (FOS = 1.3 and 1.1, respectively). The site was assessed to be a seismic Site Class D with a corresponding PGA of 0.374g for a 1 in 2475-year return period.

Based on these results, the site constraints and after several design team discussions, ground improvement was selected as the preferred option by MTO for construction of the new embankment. The advanced laboratory testing consisted of both standard and long-term consolidation testing and constant rate of strain testing. The results of the laboratory oedometer testing, from both phases of investigation, are provided in Table 1.

Table 1. Summary of laboratory oedometer testing

Borehole / Sample Number / Lab	Type of Test	Sample Depth/Elevation (m)	σ_P' (kPa)	σ_{VO}' (kPa)	$\sigma_P' - \sigma_{VO}'$ (kPa)	Cc	Cr	e_o
17-11 / 2 / GAL	IL	4.8/43.3	125	35	90	1.55	0.01	2.3
17-11 / 3 / GAL	LT	6.4/41.7	-	44	-	-	0.02	2.4
17-11 / 4 / GAL	IL	7.9/40.2	130	53	77	1.30	0.03	1.9
17-11 / 2 / UWO	CRS	4.8/43.3	180	35	145	1.41	0.08	1.7
17-11 / 3 / UWO	CRS	6.4/41.7	120	44	76	1.64	0.2	2.4
17-11 / 4 / UWO	CRS	7.9/40.2	190	53	137	1.43	0.09	1.8
16-5B / 1 / GAL	IL	5.7 / 42.4	100	60	40	1.23	0.04	2.2
16-5B / 2 / GAL	IL	7.6 / 40.5	100	70	30	0.81	0.02	1.7
16-3 / 8 / GAL	IL	8.9 / 39.5	140	80	60	2.09	0.02	1.9

GAL Golder Associates laboratory
UWO University of Western Ontario laboratory
σ_P' Apparent preconsolidation pressure
σ_{VO}' Computed existing vertical effective stress
C_c Compression index
C_r Recompression index
e_o Initial void ratio
IL Incremental Loading Oedometer test
LT Long-term Oedometer test
CRS Constant Rate of Strain test

4 CONTRACT DELIVERY MODEL AND PERFORMANCE REQUIREMENTS

4.1 Contract Delivery Model

The Highway 401 and County Rd 2/34 bridge replacement was tendered as a conventional Design Bid Build project, except for the approach embankments to the new bridge. The ground improvement was to be delivered using a Design Build procurement approach. MTO concluded that the Design Build model was better suited for the selection, design, and construction of a preferred ground improvement method, recognizing that local Design Build Contractors had experience with ground modification and with this contract delivery model. Other benefits to this approach were that administering the embankment design within a Design Build model would optimize design and

construction resulting in more economic designs with scheduling advantages. The DB model also transferred accountability to the Design Builder and in doing so reduced the risk of contractual claims.

There were key design interaction issues that had to be considered for development of the ground improvement drawings and the special provision discussed below. Specifically, these were related to the interaction of the ground improvement with the abutment foundation system. Additional analysis was competed during the design to assess the limiting distance for the ground improvement from the pile locations to have predictable lateral resistance from unimproved existing/native soils and to assess the potential settlements at the abutment. Although this analysis is not discussed further in this paper, this interaction between the design bid build structure and the design build ground improvement needs to be considered in the overall design and tender.

4.2 Special Provision

A Ground Improvement Special Provision that specified the requirements for the design, construction, and performance of ground improvement at the approach embankment was included in the Contract Documents. A minimum design life for the approach embankments of 75 years was specified.

The special provision for this project was developed with the intent to provide design build specialty contractors bidding on the project to provide as much latitude as possible for design and construction of the works. The special provision was performance based and the contractor was required to meet the settlement limits tabulated in Table 2.

Table 2. Embankment settlement requirements

Distance from Face of Bridge Abutment (m)	Maximum Total Settlement (mm)	Maximum Differential Settlement Rate
0 – 20	25	100:1
20 – 50	50	100:1
50-75	100	100:1
>75	200	100:1

In addition, the design-build contractor was required to provide improved ground that would result in widened embankments with Factors of Safety greater than 1.3 and 1.1 against global static and seismic instability, respectively.

The specification required a warranty period of three (3) years from the date of substantial completion. During this warranty period, the Contractor was to warrant that the embankments were to meet the performance requirements. For any non-conformances, the Contractor was required to submit a proposal for remediation to the MTO for any noncompliance during the warranty period.

5 DESIGN

5.1 General

The design-build specialty Contractor, Menard, chose a combination of controlled modulus columns (CMC) and wick drains as the preferred option for ground improvement.

Controlled Modulus Columns (CMC) are designed, installed, and controlled according to specifications validated by European authorities.

CMC are vertical semi-rigid inclusions comprised of injected concrete columns, installed with a hollow full-lateral-displacement auger. During auger extraction, the column is developed by grouting under controlled limited pressure through the hollow stem of the auger. Column diameters can vary between 250 and 450 mm as required by the design.

A load transfer platform is provided over the top of the columns to effectively transfer the surface loading from the embankment to the CMCs.

The works must be performed from a stable working platform free of water. The entire process is vibration free and generates minimal to no spoil at the surface.

The CMC solution is meant to provide increased stiffness to the soil mass to control both the total and differential settlements. The imposed embankment loads are transferred between the more compressible soil to the more rigid CMCs.

5.2 Design Calculations

Three categories of calculations were carried out for the design of the ground improvement system: settlement, CMC integrity and embankment slope stability.

Settlement calculations were carried out to assess the performance of the embankment over time and ensure that the design criteria were followed. Integrity calculations to assess the capacity of the individual CMCs to withstand the loading generated by the embankment placement were also completed. Finally, embankment stability calculations were carried out to assess the global stability of the embankment with the ground improvement system in place for both static and seismic conditions.

To study the settlement and the behavior of CMCs under the embankment in detail, axisymmetric finite element models were first carried out in PLAXIS 2D. A representation of axisymmetric modeling, which corresponds to a representative cell of a CMC, is presented in the Figure 2.

Calculations were also carried out considering consolidation to determine the impact of creep on the absolute and residual settlement magnitudes. The soft clay was modeled using the Soft Soil Creep material model to capture the secondary settlement (creep) that would occur over time. Other soil types were modeled using a Mohr-Coulomb model.

The study was carried out for different heights of embankment fills and made it possible to define the spacing between the CMCs and the order of magnitude of the expected settlements.

Based on the results of the axisymmetric models, different finite element models with plane deformations (2D) were prepared to determine the horizontal and vertical displacements as well as the forces in the CMCs. These models were representative of the most critical sections of the improved ground area, from the point of view of the compressible layer thicknesses, the geometry of the embankment, existing or planned structures and the phasing of the works.

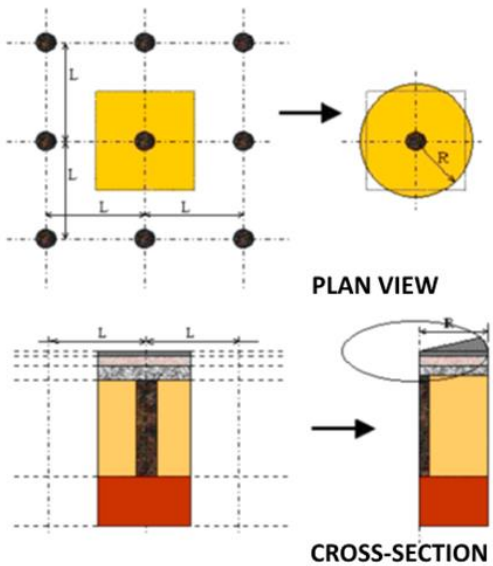

Figure 2. Schematic of an axisymmetric model

5.3 Settlement Analyses

The calibrated parameters from the axisymmetric model were used to develop 2D Plane-Strain models to look at the construction of the new embankment and the associated settlement of both the existing and future roads. The integrity of the Controlled Modulus Columns was also verified based on these outputs.

Plane-strain models were developed at several chainages along the embankments based on the provided MTO cross-sections. Table 3 indicates the estimated settlement magnitudes from those models during construction (assumed to be 6 months) and post-construction for the design life (assumed to be 15 years) for the north embankment.

As can be seen from Table 3, the calculated post-paving settlements of the new embankment and the during-construction settlements of the existing embankment are below the settlement criteria outlined in the specification. Furthermore, the differential settlement rate is less than the specified 100:1 criterion.

Calculations of the CMC axial loads and bending moments were also performed to ensure CMC integrity. CMC integrity calculations were carried out for each loading case assuming a column grout strength of 20 MPa. All CMCs passed this verification except for two (2) individual CMCs in sections 10+100 and 10+125 (9+910 and 9+885). To ensure the integrity of these CMCs, 25 MPa grout was required within these sections in the affected areas.

Table 3. Predicted settlement

North Approach Station	Emb. Height (m)	Estimated Max. Settlement of Existing Road During Construction (mm)	Estimated Max. Settlement of New Road During Construction (mm)	Estimated Max Settlement of New Road During 15-year Pavement Life (mm)
9+950	8	26.6	133.3	21.4
9+940	7.5	12.7	73.3	43.3
9+910	7	20.4	99.2	92.1
9+885	6	19.7	70.9	110
9+850	4	10.2	57.8	80.9
9+835	2.5	15.4	81.0	150

The wick drain spacing was determined based on the consolidation calculations. The maximum wick drain spacing was determined to be 2.2 metres, centre to centre, in a square grid to obtain at least 95% consolidation during the 6-month waiting period.

5.4 Embankment Stability Analyses

Global stability calculations were carried out for the critical case (max embankment loading) to verify the FOS for short term and long-term global stability and for stability under the design seismic conditions. While acceptable FOS's for short- and long-term stability were easily achieved, the pseudo static stability presented a challenge as the existing embankment was not supported by any ground improvement system.

Based on the SLOPE/W model the critical acceleration for a FOS of 1 was determined to be a PGA of 0.33 (instead of the 0.37 required for the site), resulting in an a_c/a_{max} ratio of 0.87. Using the simplified method by Ambraseys and Menu (1988), it was determined that the median displacement will be in the range of 1-5mm which satisfied the specification requirements.

As there are two critical parameters that may vary (undrained shear strength of clay and the effect of ground improvement) a sensitivity analysis of the results was carried out. The results of these calculations are summarized in Table 4 below. Method 1 looks at the situation with no ground improvement effect considered, while Method 2 includes the influence of the ground improvement. The results are shown for a range of undrained shear strengths for the clay. Displacement calculations were carried out using the Ambraseys & Menu method outlined above.

Table 4. Estimated displacement calculations

Undrained Shear Strength of Clay (kPa)	Method 1 Estimated Displacement (mm)	Method 2 Estimated Displacement (mm)
25	73	1 *(< 10mm)*
30	20	0 *(< 10mm)*
35	5	0 *(< 10mm)*

6 INSTRUMENTATION

6.1 Scope

To monitor the performance of the embankment and to learn about the behavior of the ground improvement, instrumentation was installed as illustrated in Figure 3 and as summarized in Table 5

Table 5. Instrumentation quantities

Instrumentation	Quantity
Settlement Plates	24
Surface Settlement Points	48
Shape Accelerometer Array	4
Multi-point Borehole Extensometer	2
Vibrating Wire Piezometers	8
Vibrating Wire Pressure Cells	4

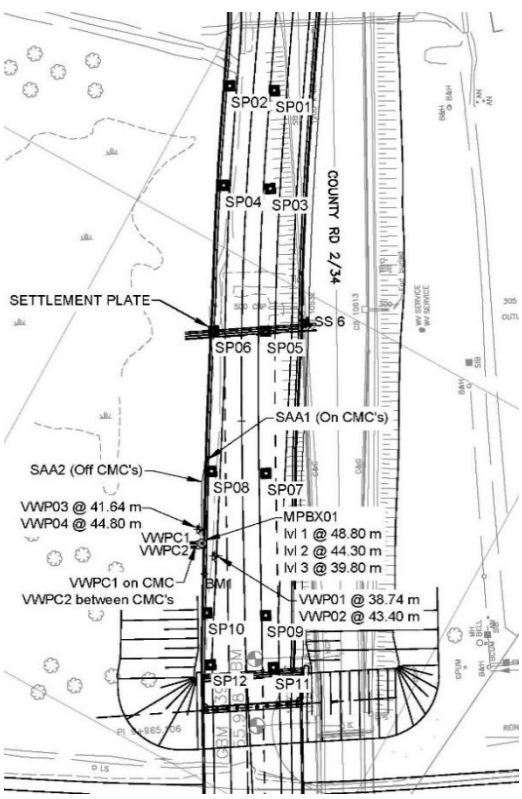

Figure 3. Instrumentation layout - north embankment

6.2 Interpretation of Results

Figure 4 shows the Settlement Plate (SP) data for the north embankment between the monitoring period of September 2019 and December 2020. The settlement at the north approach embankment (SP1 to SP12) ranged from 4 mm of heave to 278 mm of settlement. The south approach embankment (SP13 to SP24, not shown) settlement ranged from 11 mm of heave to 85 mm of settlement.

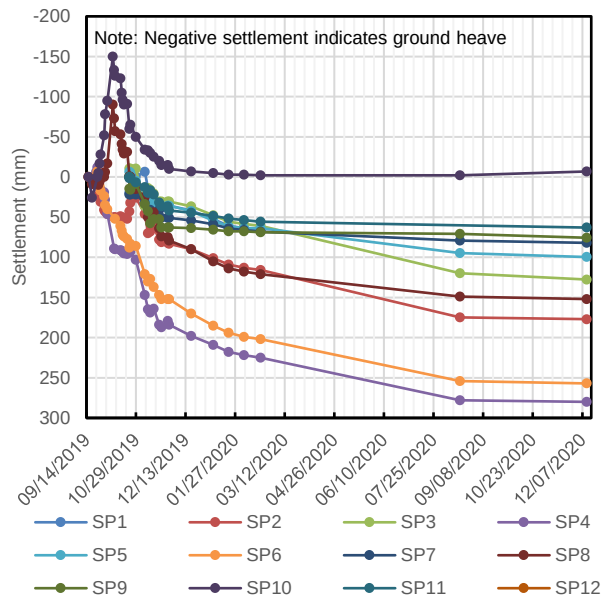

Figure 4. Settlement plate data during construction

Table 6 below presents a comparison of the settlement plate monitoring data with the predicted settlement during construction for both the north and south approach embankments.

Table 6. Predicted versus monitored settlement

Station		Predicted Settlement (mm)	Actual Settlement, April 8, 2021 (mm)
NORTH	9+835	81	86 (SP1), 177 (SP2)
	9+850	58	131 (SP3), 281 (SP4)
	9+885	71	100 (SP5), 259 (SP6)
	9+910	99	84 (SP7), 158 (SP8)
	9+940	73	71 (SP9), SP10 heaved
	9+950	73	63 (SP11), 53 (SP12)
SOUTH	10+050	73	83 (SP13), 85 (SP14)
	10+070	73	22 (SP15), 80 (SP16)
	10+100	99	4 (SP17), 71 (SP18)
	10+125	71	50 (SP19), 56 (SP20)
	10+150	58	SP21 heaved, 31 (SP22)
	10+165	81	SP23 & SP24 heaved

The settlement measured at the settlement plate locations during construction agrees reasonably well with the predicted values. It is noted that during excavation for bridge abutment construction, the contractor stockpiled approximately 500 m^3 of excavated material in in the area off settlement plates SP1 to SP6 leading to greater settlement than predicted during design. It is also noted that SP1 and SP2 are located outside the limits of CMC installation (see Figure 3).

Figure 5 shows the settlement monitoring data for multi-point borehole extensometer MPBX1 which was installed below the north approach embankment. MPBX1 initially indicates heave, which is noted to be a result of the displacement auger and grout injection used during the installation of CMCs. Based on this data a maximum settlement of 131 mm was recorded at MPBX1 at elevation 48.80 m.

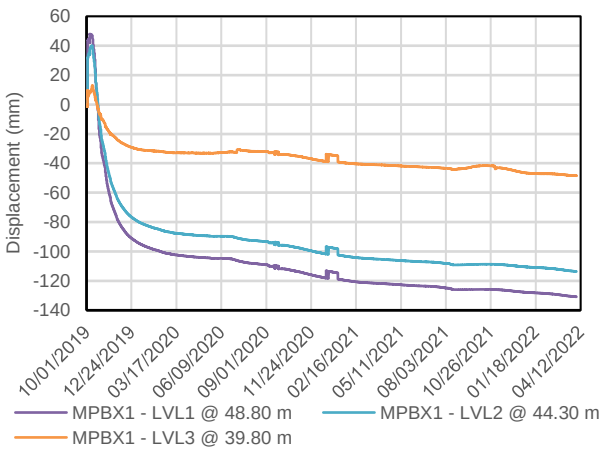

Figure 5. North approach MPBX monitoring data

A parallel installation of Shape Accelerometer Arrays (SAAs) was installed at both approaches, with one array installed above the CMCs and the second array installed parallel and adjacent to the CMCs. The SAAs were installed after construction of the granular construction access pad and after all the CMCs and wick drains were installed. It was therefore difficult to visually identify where CMCs had been installed during placement of the SAAs. The results from the SAAs installed across the CMCs indicate that ~25mm of settlement has occurred within the 2 years of monitoring following fill placement and the SAA installed adjacent to CMCs have settled approximately 150 mm. This indicates that the CMCs are carrying a greater portion of the embankment loading and reducing the vertically applied stress on the native clay deposit.

Figure 6 shows the vibrating wire piezometer (VWP) monitoring data from the four vibrating wire piezometers installed below the north approach embankment.

The data shows the excess pore water pressures (PWPs) resulting from fill placement in the area of the north approach dissipated within approximately 6 months following fill placement.

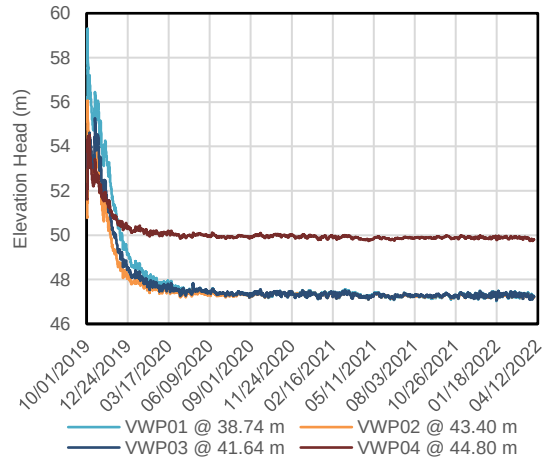

Figure 6. North approach VWP monitoring data (Static water level ~ 47 m)

Although not shown, the excess PWPs resulting from fill placement for the south approach embankment dissipated after approximately 3 months following fill placement. This was likely aided by the number of wick drains installed within the approach embankment area.

Figure 7 shows the delta pressure monitoring data from the four Vibrating Wire Pressure Cells (VWPC) installed in the north approach embankment. VWPC1 was installed on one of the CMCs while VWPC2 was installed on the ground between CMCs. The embankment height in the area of the instrumentation was approximately 6.8 m resulting in an expected embankment loading of 155 kPa. The data indicates that the steady state delta pressure acting on VWPC1 is 225 kPa while the steady state delta pressure acting on VWPC2 is 125 kPa indicating that the CMCs have taken a higher portion of the embankment load than the soils between the CMCs.

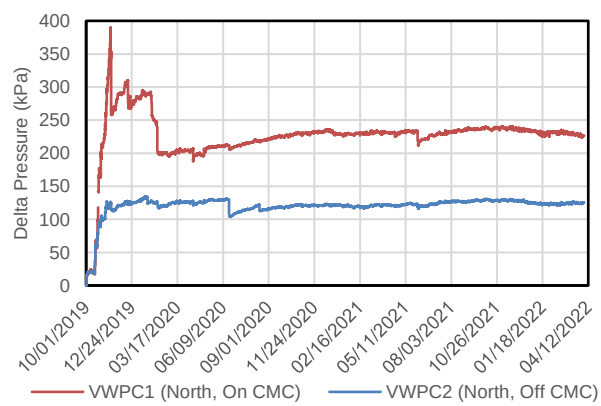

Figure 7. North approach VWPC monitoring data

The geotechnical design engineer had predicted that foundation settlements for embankment constructed without ground improvements would settle up to 1,000 mm over a period of up to 50 years. The historical settlement data for the existing embankments constructed of similar

height had recorded settlements in the order of 1,550 mm. Based on the data presented above, the use of CMCs in conjunction with wick drains has reduced the foundation soil settlement well below these magnitudes.

7 COST COMPARISON

Table 7 provides a cost comparison of four embankment design options considered for this project.

Table 7. Cost comparison

Option	Total Cost ($M)
Expanded Polystyrene	4.5 - 5
Wick Drains with Surcharge	4 – 4.5
Ground Improvement – CMC'S	3
Ground Improvement -Deep Soil Mixing	4.8 - 6

The table identifies that Ground Improvement using Controlled Modulus Columns was more cost effective than the other options considered.

8 CONCLUSIONS

Foundation Engineers are challenged to design and construct embankments over weaker, compressible soils. There are several methods in the Foundation Engineer's toolbox to ensure a safe, reliable embankment design that satisfies embankment performance criteria. The success of ground improvement technology employed at the Hwy 401/County Rd 2/34 project is demonstrative that ground improvement using CMCs and wick drains is a viable alternative that can accelerate construction and not compromise long term performance. On this project,

1. The actual settlements (with ground improvement) agree reasonably well with the predicted values.
2. CMCs are carrying a greater portion of the embankment loading and reducing the vertically applied stress on the native clay deposit.
3. The settlement measured across the CMCs indicate that ~25mm of settlement has occurred within the 2 years of monitoring following fill placement and the settlement installed adjacent to CMCs have settled approximately 150 mm.
4. The use of CMCs in conjunction with wick drains has reduced the foundation soil settlement well below the settlement estimates without ground improvement.
5. The combination of CMC ground improvement and the wick drains enabled the settlements to be realized within the 6-month period specified in the Contract.

Ground improvement reinforces MTO priorities and values that encourages sustainability of our infrastructure by investing in innovation and collaboration with engineering service providers and contractors. As an alternative to other conventional methods, ground improvement techniques demonstrate harmony with the environment, and is less intrusive minimizing disturbance associated with partial or full sub excavations, groundwater drawdowns, haulage of spoil and backfilling of materials. The benefits of ground improvement translate into projects being built faster and cheaper and is considered cutting edge technology for highway engineering projects at the MTO.

9 REFERNCES

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