

Ground Improvement for Embankment Design and Construction – A Case Study on Highway 15 and Crosby Creek in Eastern Ontario

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ABSTRACT

Operational improvements to an approximate 27 km stretch of Highway 15 between Seeley's Bay and Crosby Creek in Eastern Ontario included a new bridge. As part of those improvements, the existing 12 m long single span bridge over Crosby Creek, founded on timber piles, had reached its usable life span and was replaced with a new bridge.

The new realigned bridge, located approximately 16 m east of the existing bridge to address geometric design standards, is a 22.5 m span prestressed concrete box girder integral abutment bridge founded on steel H-piles socketed into the bedrock. Approach embankments on the north and south side of Crosby Creek have a maximum height of 4 m.

The approach embankments are founded on a 2.7 m to 3.9 m thick deposit of clayey silt to clay underlain by a silty sand to sandy silt till which in turn is underlain by gneiss and granite bedrock. Lying beneath a stiff to very stiff desiccated crust of thickness ranging from 1 to 1.8 metres in thickness, the soft clayey silt to clay stratum provided design challenges related to the settlement and stability of the embankment.

Following a comprehensive review of embankment foundation design alternatives, ground improvement utilizing aggregate piers was chosen as the preferred option. This paper presents the results of the foundation investigation program, the design methodology, the contract package development including performance specifications, contract delivery method, and embankment construction and monitoring of the embankment during and following construction.

INTRODUCTION

The Ministry of Transportation of Ontario (MTO) undertook a detailed design consultant assignment to provide engineering design services for improvements to Highway 15. GENIVAR was retained as the Prime Consultant for the assignment. The limits of the project extended for 27.1 km on Highway 15, from Seeley's Bay to Crosby. The scope of work included the replacement of the Crosby Creek Bridge (Site No. 16-023) and realignment of the approaches.

The existing bridge was a 12 m long single span steel girder bridge with a concrete deck and concrete abutments on timber piles. The bridge had a clear roadway width of 9.5 metres between barriers. The structure had undergone a number of repairs in the past and had approached its usable life span. A renewal option analysis concluded that the most appropriate action for renewal was a complete replacement.

The stretch of Highway 15 where the existing bridge was located was geometrically deficient and adjustments in the alignment were required to be in compliance with design speed parameters. On the re-aligned highway, the replacement structure was designed to be approximately 16 meters to the east of the existing bridge.

The deficiencies of the roadway width at the old structure were addressed by a new wider structure with a clear width of 13.5 metres between barriers matching the upgraded and realigned highway cross-section.

As part of the structural assessment, an integral bridge type with a span of 22.5 metres was selected to be the most appropriate structural solution that accommodated the geotechnical and topographic conditions while providing an economical and durable structure. The new structure consists of precast prestressed box girders covered with a concrete slab, waterproofed and paved. The new concrete abutments are founded on steel H Piles socketed in bedrock.

The subsurface conditions at the site presented some foundation engineering challenges. In view of a stratum of soft to firm clayey silt to clay deposit, the design of the approach embankments required due diligence in the settlement and stability analyses to ensure acceptable embankment performance during and following construction. The proposed embankment heights were up to 4 metres. Mitigation measures were needed for any portion of the approach embankment that had more than 2.5 m of new fill for both settlement and stability. Global stability was a concern when the embankment including any surcharging/preloading was greater than 3.4 m in height.

A suite of alternatives were compared during the detailed design of the approach embankment. Following a comprehensive review of the alternatives, 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.

SITE DESCRIPTION

The site of the bridge replacement over Crosby Creek is located on Highway 15, just south of the town of Crosby, and some 60 kilometres north of Highway 401 and east of Kingston in Ontario. The bridge is located within a clay plain through which Crosby Creek flows in a shallow bed. At the location of the crossing, Crosby Creek is oriented approximately east-west and flows in a westerly direction. The creek is approximately 15 to 25 m (from west to east) in width at the proposed crossing. The ground surface slopes up from the creek banks in both the north and south directions. On the north side of the creek, a grass covered marshy area extends northward 5 to 10 m from the edge of the creek.

SITE INVESTIGATION AND SUBSURFACE CONDITIONS

The site investigation at Crosby Creek was carried out in accordance with MTO guidelines; two (2) boreholes were advanced at each proposed abutment location and one (1) borehole was put down about 20 metres behind the abutment, along the west side of the proposed embankment. The four abutment boreholes were advanced to the surface of the bedrock and then the bedrock was cored to an additional 2 to 3 metres of depth. The approach embankment boreholes were augered to practical refusal.

The interpreted subsurface conditions, based on the borings, along the profile of the bridge replacement alignment are indicated on Figure 1.

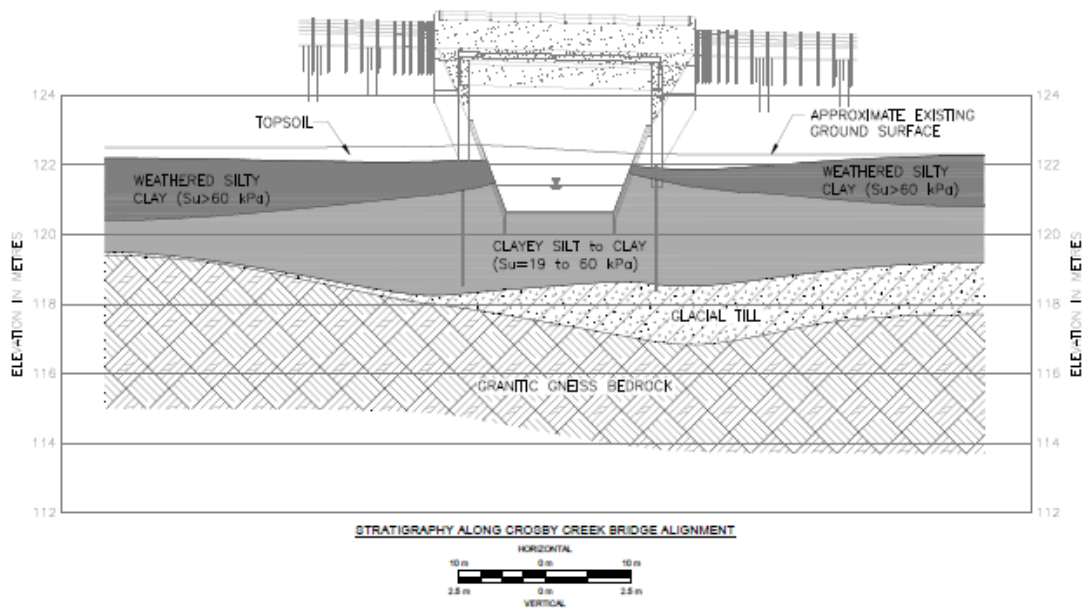


Figure 1 – Stratigraphy along Crosby Creek Bridge Alignment

As indicated on Figure 1, topsoil, up to about 0.6 metres in thickness, overlies a deposit of clayey silt to clay which is about 3 metres in thickness. The upper portion of the clayey deposit, up to about 1 metre in thickness, has been weathered to a very stiff to stiff brown crust. The remainder of the clayey deposit below the weathered crust is grey, unweathered, sensitive and generally firm to soft, with shear vane values typically ranging from about 19 to 25 kPa, with some higher values in the upper portion of the deposit. The results of two laboratory oedometer tests (stress controlled) on samples of the unweathered clay are indicated in Table 1.

Table 1 – Oedometer Test Results

Borehole No.	Sample Depth/Elev. (m)	Unit Wt. (kN/m ³)	$\sigma_{p'}$ (kPa)	$\sigma_{vo'}$ (kPa)	$\sigma_{p'} - \sigma_{vo'}$	C_c	C_r	e_o	OCR
08-3	3.30 / 119.3	18	62	24	38	0.59	0.01	1.06	2.6
08-5A	2.48 / 119.8	18	93	31	62	2.13	0.01	1.14	3.0

The clayey deposits are underlain by glacial till, ranging from about 0.1 to 1.6 metres in thickness, which is in turn underlain by strong granitic gneiss and granite bedrock.

EMBANKMENT DESIGN

The proposed embankments for the new bridge are up to about 4 m in height (above subgrade after stripping the topsoil). During design, it was estimated that the loading due to the weight of the new embankment fill would potentially result in consolidation settlement magnitudes of up to 100 mm and secondary settlement magnitudes of up to 60 mm over 20 years. These settlement magnitudes would exceed Ministry of Transportation Ontario (MTO) guidelines for tolerable post paving movement, within 50 m of the pile supported abutments.

In addition, the stability analysis carried out for design indicated that embankments up to 3.4 metres in height would have acceptable factors of safety against global instability but that embankments greater than that height (i.e., within a few metres of the abutments) would not have the required stability.

The foundations design report for this indicated a variety of potential options to address the settlement magnitudes and stability of the approach embankments and these included lightweight fill (e.g., expanded polystyrene or slag), pre-loading (with or without wick drains and/or surcharging) and in-situ soil improvement. Ultimately, MTO decided, with the moderate depth of soft clay to refusal (about 4 m), to further explore ground improvement technology.

A foundations instrumentation program was prepared for the design-build ground improvement for the bridge approaches. The instrumentation plan included standpipe piezometers (SSP), vibrating wire piezometers (VWP), settlement rods (SP), slope indicators (SI) and surface settlement monitors (SSM). Locations of the instrumentation are discussed within the

monitoring subsection of this paper. All instrumentation was intended to be installed after the completion of the ground improvement but prior to the placement of the embankment fill, with the exception of the SSM's which would be placed on the surface of the completed pavement. The intent of the foundations instrumentation was to measure the settlements, lateral deformation and rate of excess pore water dissipation (i.e., consolidation) observed with the ground improvement to the predicted non-improved behavior. In addition, there was some question in the designers' minds whether the disturbance of the sensitive clayey soils due to the ground improvement method would result in excessive settlement or reduced stability (or large lateral movements). It was hoped that the results of the monitoring would provide some indication of whether the re-moulding of the clayey soil during installation was a concern, for this or other installations in similar deposits.

CONTRACT DELIVERY MODEL AND PERFORMANCE CRITERIA

The Highway 15 operational improvements project for the highway and the structure between Seeley's Bay and Crosby Creek was tendered as a conventional Design Bid Build project under Contract 2013-4067 with the exception of the approach embankments to the new bridge. This component was tendered as a Design Build.

A Ground Improvement Special Provision that specified the requirement for the design and construction of the approach embankments prior to bridge construction was included in the Contract Documents. A minimum design life for the approach embankments of 75 years was specified. Design criteria included:

1. The magnitude of post construction settlement was limited to the limits specified in Tables 2 and 3.
2. The embankments founded on the ground improved native soils were to be designed for a factor of safety of 1.3 for static stability and a factor of safety of 1.1 for seismic stability.

Table 2 – Total Roadway Settlement

Distance From Abutment	Maximum Settlement During Warranty Period(mm)			
	0-20 m	20-50 m	50-75 m	>75 m
Non-Freeways	25	50	100	200

Table 3 – Roadway Differential Settlement Relative to Bridge Abutment Edge

	Maximum Settlement During Warranty Period(mm)			
Distance From Abutment	0-20 m	20-50 m	50-75 m	>75 m
Non-Freeways	15	25	40	40

The Contractor was to assess the sufficiency of the Foundation Investigation information included in the Contract Package and was responsible for undertaking additional foundation investigation if required to complete their design. Additional boreholes and Cone Penetration Tests were conducted. The Contract Documents specified a warranty period of seven years specified from the date of substantial completion. During this warranty period, the Contractor warrants that the embankments will meet the performance requirements. For any non-conformances, the Contractor is required to submit a proposal for remediation to the MTO for any noncompliance during the warranty period. The Contractor is not to proceed with any repairs until approval is given by the MTO.

DESIGN/BUILD

Geosolv Design/Build was the Ground Improvement subcontractor retained by Cruickshank, the Prime Contractor on the project. A Geopier® system (i.e., aggregate piers) was selected to support the two approach embankments, providing embankment settlement control, time rate of consolidation and global stability. A combination of the Armorpack® and Rampact® systems were used at varying spacing to meet the varying settlement and criteria and to limit post-construction settlement.

The Armorpack and Rampact Methods - General

The Geopier Armorpack system creates strong and stiff elements that exhibit strength and stiffness in very weak soils. The Geopier Armorpack elements are constructed by inserting a hollow mandrel within the patented 600 mm (24 in) diameter Geopier Armorpack Sleeve, and driving the sleeve to the design depth using a strong static force augmented by high frequency vertical impact energy. The mandrel consists of a hopper (at the top end) that serves as a conduit for delivering aggregate through the mandrel and a stone valve (at the bottom end).

Installation depth capacity normally ranges from about 3 to 11 m (10 to 35 ft.), depending on design requirements. The displacement process significantly reduces spoils. The Armorpack sleeves have small diameter perforations for relief of pore pressure in the matrix soils where required.

After driving to the design depth, the Armorpack sleeve remains in place at a desired depth. Aggregate is then placed inside the hopper propagating to the bottom of the mandrel and into

the sleeve. Compaction of the aggregate is then achieved through static down force and dynamic vertical ramming from the hammer. The process densifies aggregate vertically and forces the aggregate laterally into the confining sleeve, bulging the sleeve into the soft matrix soil. Once the aggregate is compacted within the sleeve, the mandrel and tamper foot are continued to be driven up and down allowing for a portion of aggregate to flow into the displaced cavity above the sleeve and forming rammed aggregate pier lifts. Figure 2 illustrates a schematic sketch of the Armorpack system. The Rampact system is constructed in a similar manner as the Armorpack system but does not use the confining shell.

The Armorpack process results in lateral stress increase in the matrix soil and combined with the high stiffness Armorpack element, provides settlement control with increased strength and stiffness in soft clay and organic soils. Applied loads are then supported by the densely compacted aggregate that is laterally confined by the sleeve, providing for a stiffer response at higher loads than aggregate piers, such as Rampact, that has no confining sleeve. This system also avoids the need for use of casings in collapsing or squeezing soils.

The decision to apply Armorpack vs Rampact is made based on the applied top of pier stress, the shear strength of the specific soil in a given area, and the required settlement control for the application.

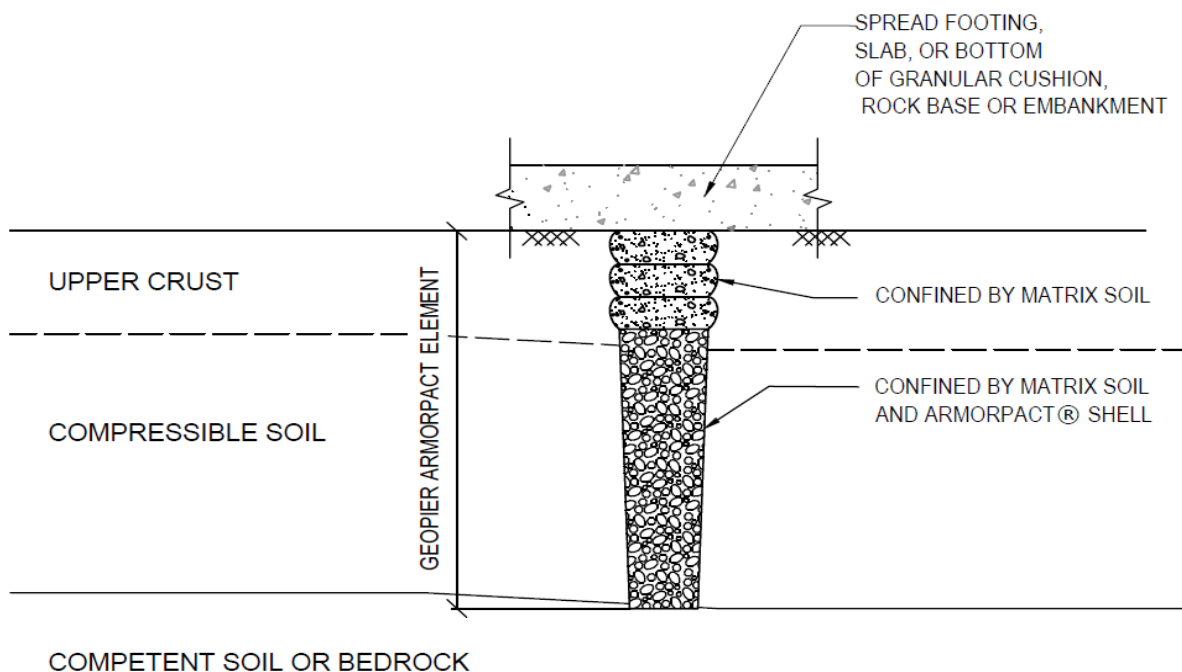


Figure 2: Geopier Armorpack System

Ground Improvement Design – Project Specific

Layout

The final ground improvement design for the Crosby Creek approach embankments consisted of Armorpact elements installed near the abutment at a spacing of 2.1 m, increasing to a spacing of 2.3 m to a distance of 25 m away from the abutment, and Rampact elements installed on a spacing of about 2.3 m between 25 m and 65 m from the abutments, as shown in Figure 3 below. The geopier elements were installed in depths ranging from 2.1 to 8.2 m.

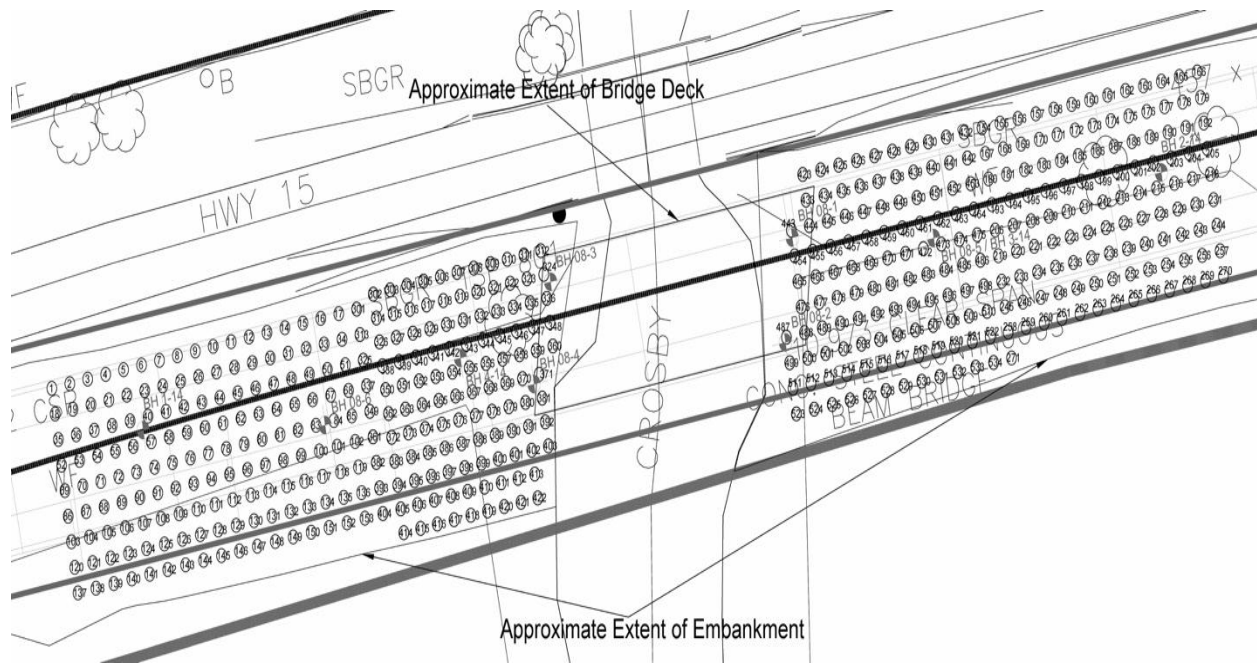


Figure 3: Geopier Layout

Settlement Analyses

Design for settlement control was carried out using the method proposed by Lawton and Fox, 1994, where, settlements are broken up into two different zones, the Geopier reinforced zone (Upper Zone) and the lower native matrix soil zone (Lower Zone).

Upper zone settlement calculations implement a composite elastic modulus analogy as shown in Figure 4. The Geopier elements act as stiff springs; the matrix soil between the piers acts as softer springs. The stiff Geopier elements attract a larger percentage of foundation-bottom stress than the soft springs. A composite elastic modulus (E_{comp}) can then be computed using Equation 1:

$$E_{comp} = (1 - R_a)E_m + R_aE_g \quad (\text{Eq. 1})$$

where E_g is the elastic modulus of the Geopier element, E_m is the elastic modulus of the matrix soil, and R_a is the ratio of the area coverage of the Geopier elements to the gross footprint area.

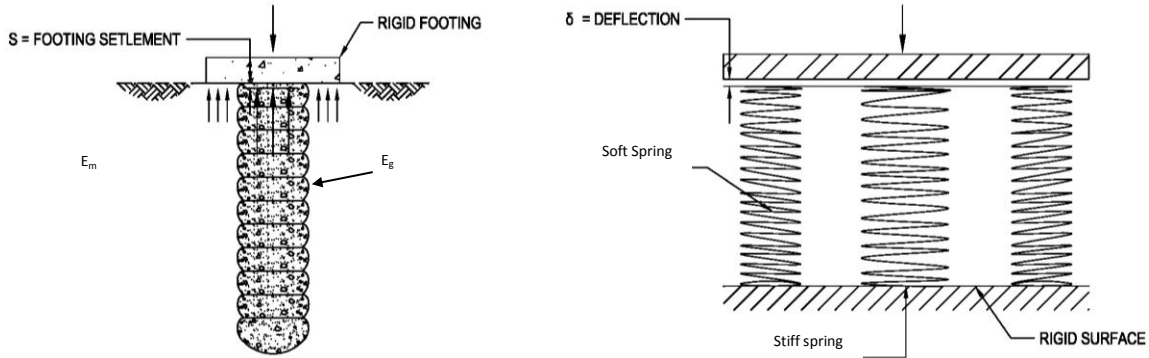


Figure 4: Geopier Design Analogy

Once composite elastic modulus (E_{comp}) is established from Equation 1, the settlement in the Upper Zone (s_{uz}) is simply computed as follows:

$$s_{uz} = \frac{qI_fH_{uz}}{E_{comp}} \quad (\text{Eq. 2})$$

where q is the average foundation-bottom stress, I_f is the stress influence factor in the upper zone, and H_{uz} is the thickness of the upper zone.

Settlements within the “lower zone” (zone of soils beneath the upper zone which receives lower intensity foundation stresses) are computed using conventional geotechnical settlement methods that involve: estimating the depth of stress influence below the foundation bottom (typically taken as twice the foundation width for square foundations and four times the foundation width for strip foundations); estimating the foundation-induced stress in the lower zone (established using conventional influence factor charts); and, estimating the compressibility of the lower zone soils.

Lower zone settlements (s_{lz}) are estimated using elastic settlement methodology with the equation:

$$s_{lz} = \frac{qI_fH_{lz}}{E_{lz}} \quad (\text{Eq. 3})$$

or

$$s_{LZ} = \left(\frac{C_c}{1+e_o} \right) H_{LZ} \log \left(\frac{p_o + \Delta q}{p_o} \right) \quad (\text{Eq. 4})$$

where q is the average foundation-bottom stress, I_f is the stress influence factor in the lower zone, H_{LZ} is the thickness of the lower zone, E_{LZ} is the secant modulus, C_c is the compression index.

The estimated settlement of Geopier-supported foundations (s) is determined by summing the upper zone and lower zone settlement values:

$$s = s_{UZ} + s_{LZ} . \quad (\text{Eq. 5})$$

Settlement parameter values used for the Crosby creek project are presented in Table 4 below.

Table 4: Design Parameter Values

Parameter		Geopier Element
Upper Zone (Bearing)	Geopier Armorpack Elastic Modulus, E_g , (MPa)	190
	Geopier Rampact Elastic Modulus, E_g , (MPa)	72
	Matrix Soil Elastic Modulus, E_m , (MPa)	2
Lower Zone (Compressibility)	LZ Elastic Modulus E_{LZ} , (MPa)	Incompressible

Settlement estimates and stress influence were calculated using the commercially available program Settle 3D by Rocscience. The Settle 3D program calculates settlements using either elastic modulus or consolidation relationships as described above and uses a Boussinesq stress influence. The results of this analysis are shown below in Figures 5-8, inclusive, indicating the settlement magnitudes assuming the use of either Armorpack or Rampact elements for the entire embankment construction; in practice both elements were used along the embankments as discussed.

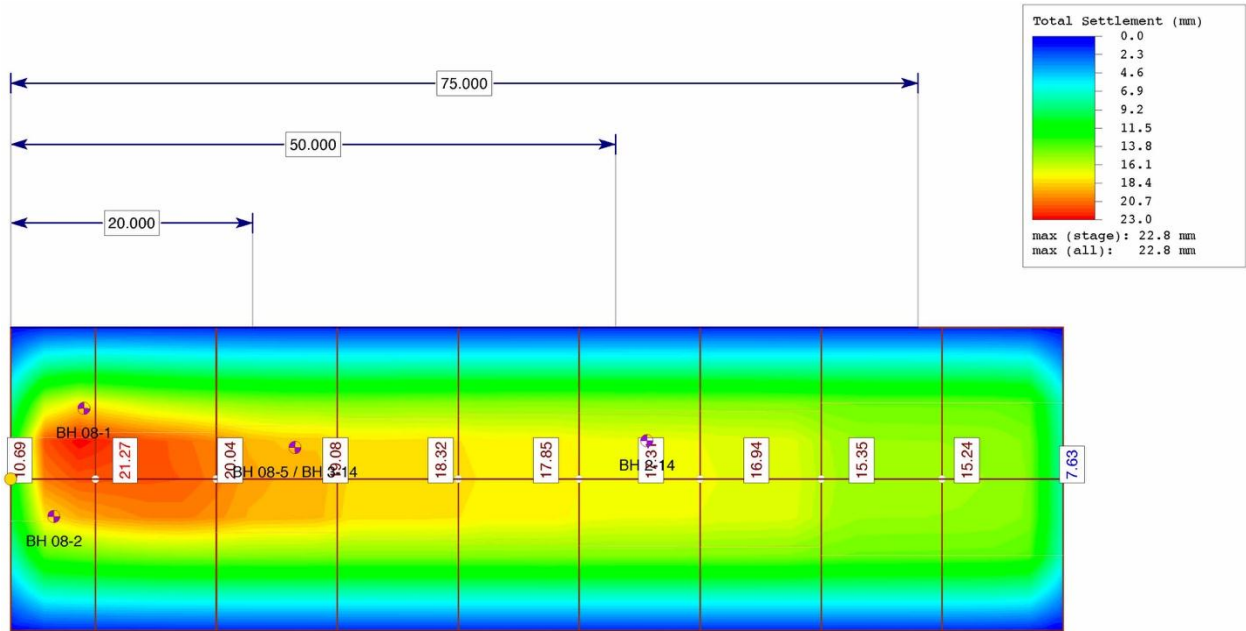


Figure 5: Settlement of North Embankment using Armorpack Elements

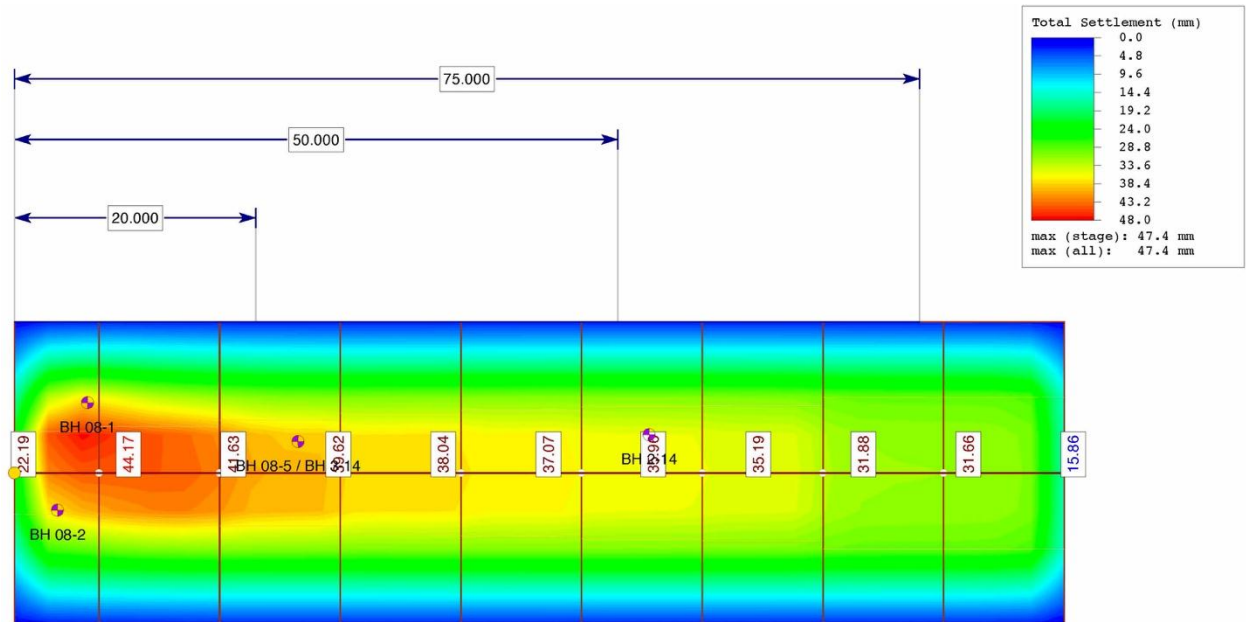


Figure 6: Settlement of North Embankment using Rampact Elements

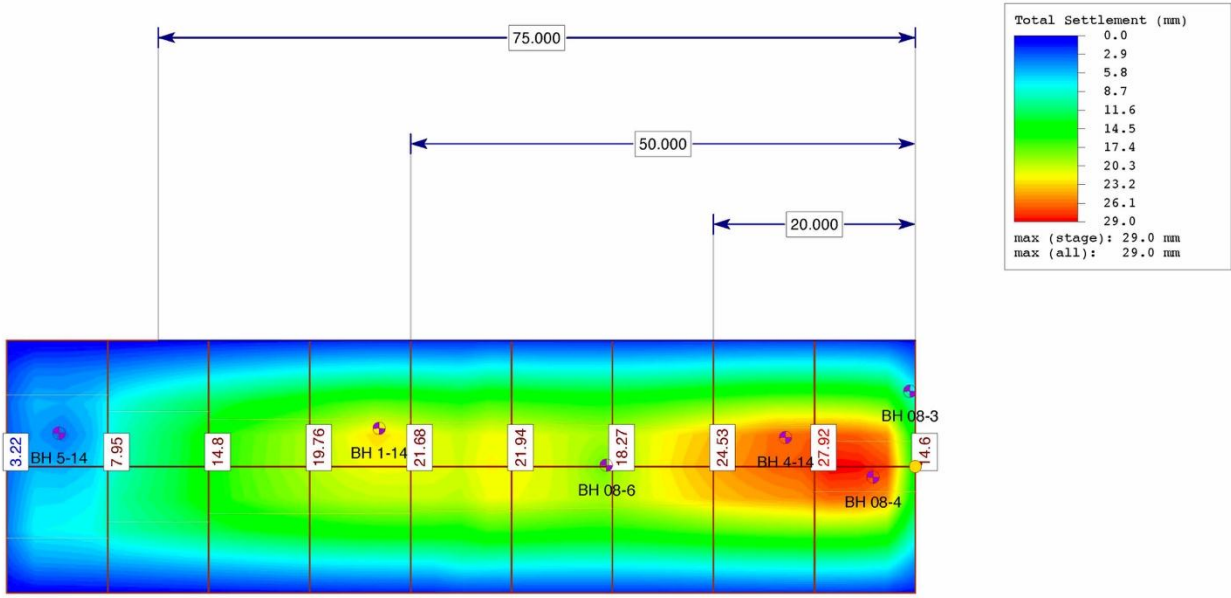


Figure 7: Settlement of South Embankment using Armorpact Elements

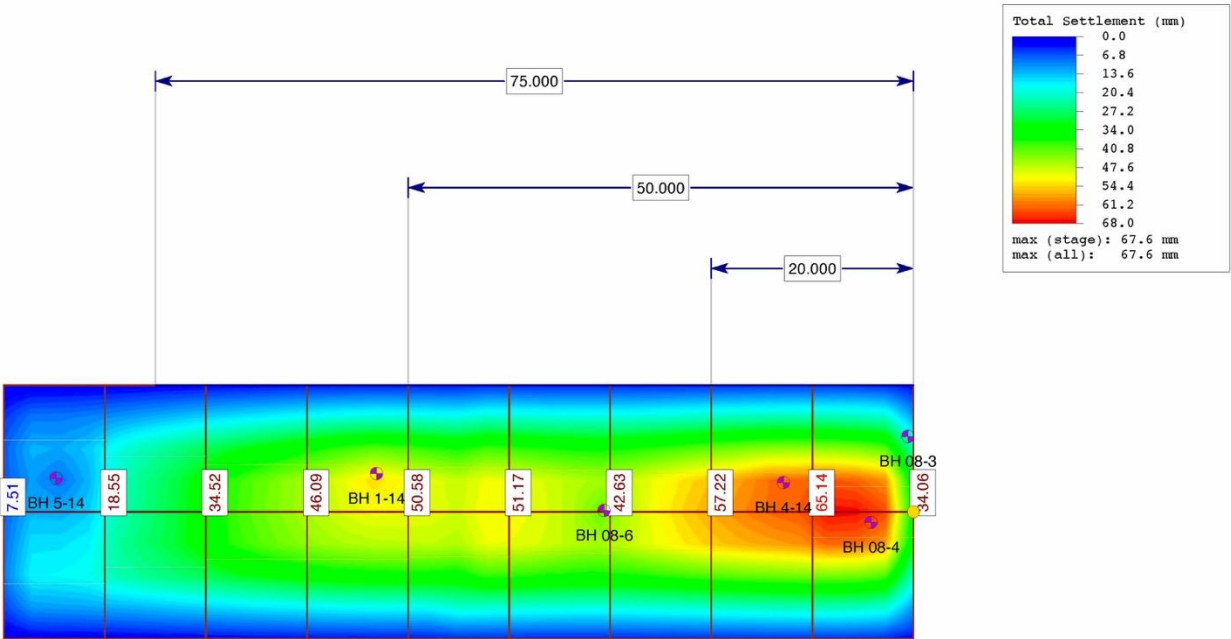


Figure 8: Settlement of South Embankment using Rampact Elements

Global Stability Analyses

The installation of Rammed Aggregate Pier system increases the composite shear strength parameter values within the aggregate pier-reinforced zones. The composite shear strength parameter values are estimated using the following equations (Barksdale and Bachus 1983, Mitchell et al. 1981, FitzPatrick and Wissmann 2002):

$$\phi_{comp} = \arctan \left[\left(\frac{R_s}{R_s R_a - R_a + 1} \right) R_a \tan \phi_g + \left(\frac{1}{R_s R_a - R_a + 1} \right) (1 - R_a) \tan \phi_m \right] \quad (6)$$

$$c_{comp} = \left[\left(\frac{1}{R_s R_a - R_a + 1} \right) (1 - R_a) c_m \right] \quad (7)$$

where, R_s is the stress concentration factor, R_a is the area replacement ratio, ϕ_g is the friction angle of the rammed aggregate pier, ϕ_m is the friction angle of the matrix soil, and c_m is the matrix soil cohesion. Factors of safety greater than 1.3 and 1.1 are desired for the static and seismic analysis, respectively.

Stability analyses, both static and seismic, were performed using the computer program SLOPE/W to evaluate the factors of safety against global instability for embankments. The parameter values used for the analysis of each wall section are included shown in Table 5.

Table 5: Soil Parameter Values for Global Stability

Type of Soil	C or Su (kPa)	ϕ' (°)
Geopier Element	0	45
Matrix Soil(silty clay crust)	128	0
Matrix Soil(silty clay)	45	0

The results of the analysis were a factor of safety of 1.6 for static global stability and 1.1 for seismic global stability analysis.

INSTRUMENTATION MONITORING

The instrumentation and monitoring points included the following:

- Two (2) standpipe piezometers (SPP) located west of the original Hwy 15 alignment, more than 10 m away from the footprint of the new embankment and ground improvement limits.
- Eight (8) settlement plates (SP) installed near the shoulder rounding split evenly between the east side and west side and the north and south embankments.
- Two (2) slope inclinometers (IC): one behind each abutment, outside of the wingwalls.
- Nine (9) vibrating wire piezometers (VWP) including 4 and 5 on the north and south sides of Crosby Creek respectively. Two VWPs on each side of the creek were installed near the toe of the embankment slope and five were installed approximately beneath the centerline.
- Eighteen (18) surface settlement markers (SSM) installed in the pavement surface: 3 cross-sections of 3 markers at approximately 20 m intervals on each embankment.

The locations of the instruments and monitoring points in relation to the ground improvement limits at the north and south embankments are shown on Figure 9.

All instrumentation and monitoring points were installed after completion of the ground improvement work and construction of a working pad and the bridge abutments. The standpipe piezometers, slope inclinometers and vibrating wire piezometers were installed between June 3 and 8, 2015. The settlement plates were installed between July 10 and 15, 2015, during which time the majority of the embankment fill was placed. The surface settlement markers were installed on September 14, 2015, immediately after paving of the surface course.

The planned monitoring frequency includes 25 monitoring events over a five year time period, following the schedule provided in Table 6.

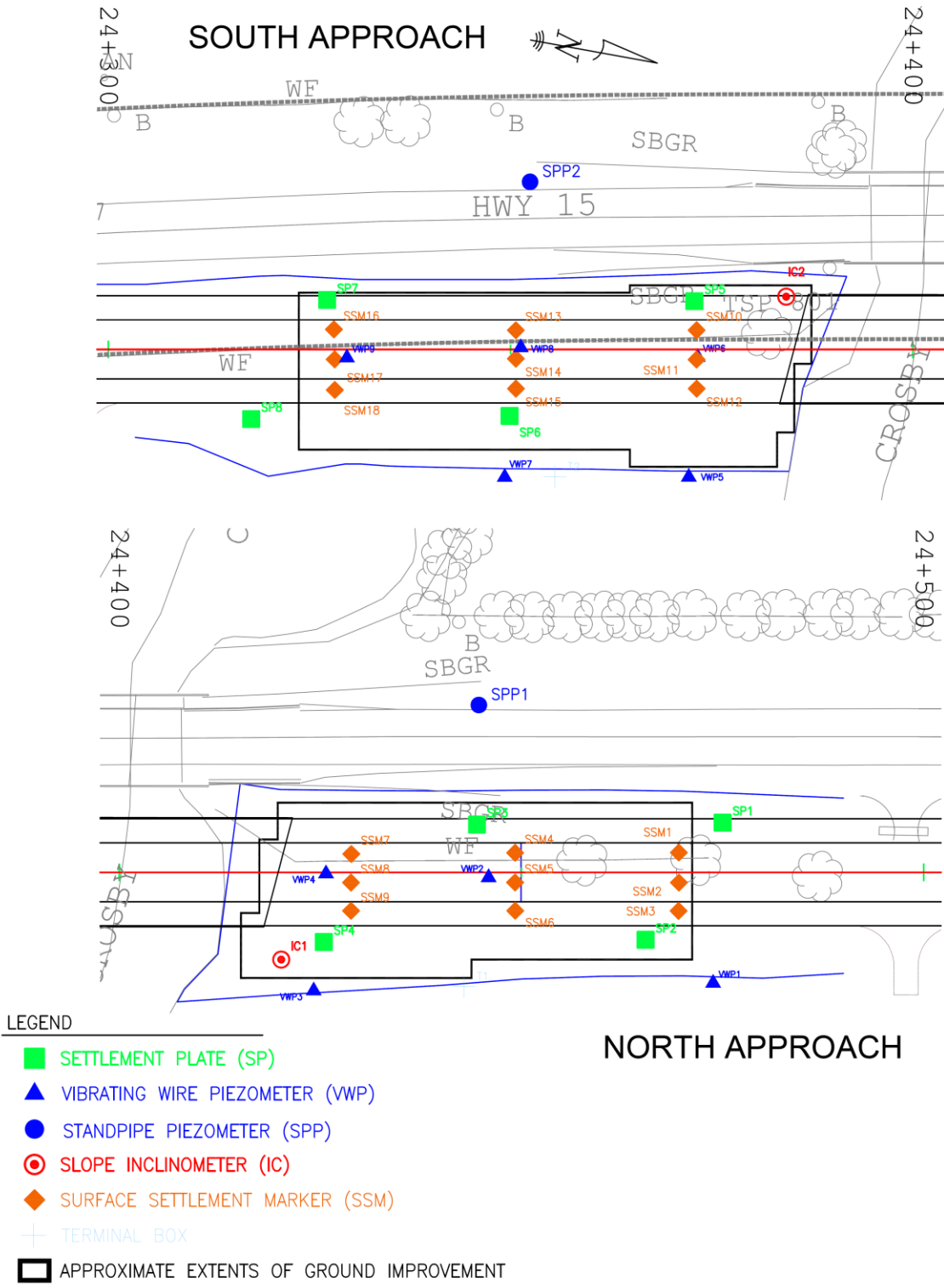


Figure 9 - Locations of instruments and monitoring points in relation to ground improvement limits at the north and south approach embankments.

Table 6 - Summary of Monitoring Frequency and Schedule

Contract Stage	Frequency	No. of Monitoring Events
Baseline Readings	3 readings on 3 consecutive days <ul style="list-style-type: none"> • VWP, IC & SPP • SSM 	3 3
During Embankment Construction	Once every 25% of embankment height	4
After Embankment Construction	<ul style="list-style-type: none"> • Twice per week for two weeks • Bi-weekly for 6 weeks • At 3 months • At 6 months • At 9 months • Annually for five years 	4 3 1 1 1 5
Total	-	25

The monitoring event for the milestone corresponding to 6 months after completion of the embankment construction was completed in March 2016. A summary of the settlement measured at the settlement plates is provided in Figure 10. Total settlement to date ranges from 2 to 45 mm, with settlement ranging from 4 to 13 mm measured during the week of primary embankment fill placement, followed by 5 to 31 mm of settlement in the approximately 2 month period between primary fill placement and paving. The measured settlement in the 6 months following paving of the highway ranges from -5 to 7 mm.

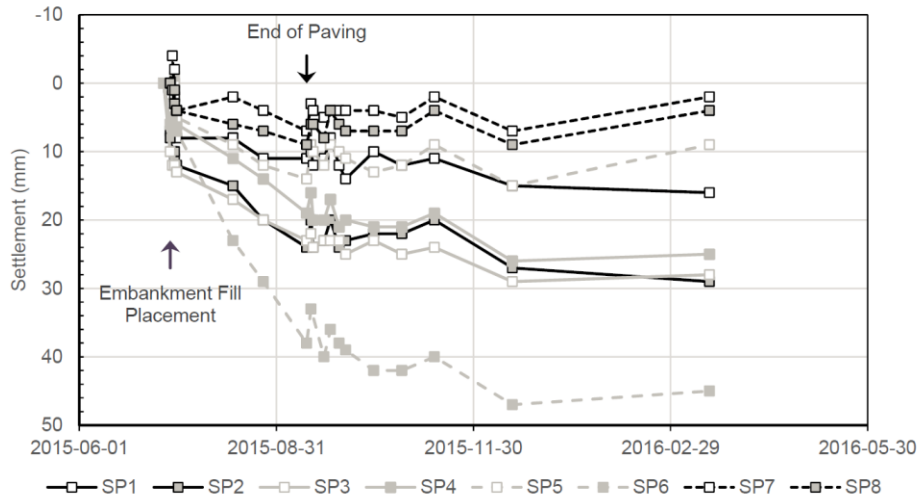


Figure 10 - Settlement Plates: total settlement versus time.

A summary of the settlement measured at the surface settlement markers is provided in Figure 11. As noted earlier, the SSM's were installed as six cross-sections. Each section consisted of a SSM at centreline and each edge of pavement. For presentation clarity, the mean settlement of the SSM's within each cross-section has been presented in Figure 11. The measured settlement in the 6 months following paving of the highway ranges from approximately 0 to 7 mm. It was noted that the mean elevation at each cross-section was higher during the March 2016 readings, possibly indicating some frost movement.

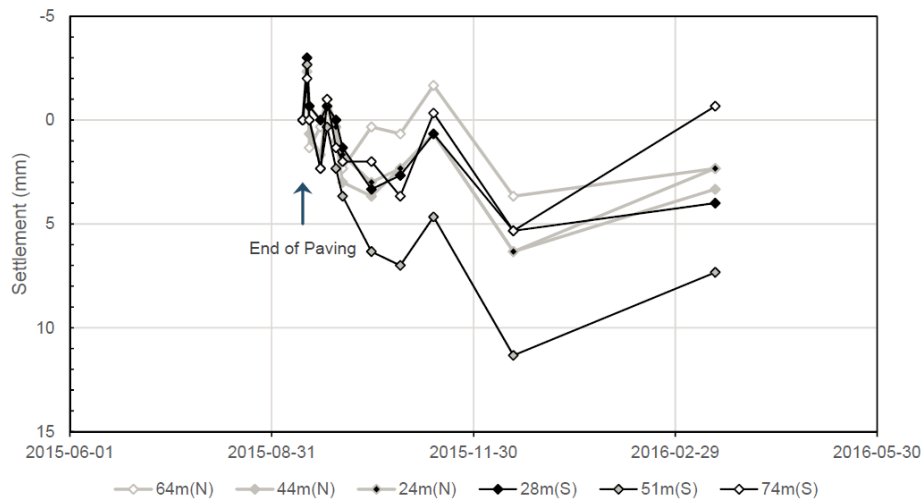


Figure 11 - Surface Settlement Markers: settlement versus time.

The VWPs were installed within the clay layer between the GeoPier elements, the tops of which were exposed prior to VWP installation. An increase in the pore pressure was measured during the placement of the embankment fill, followed by a gradual decrease. The pore pressure measurements from VWP 5 (located at the toe of slope beyond the limits of ground improvement) and VWP 6 (located beneath the middle of the highway within the ground improvement zone) are presented in Figure 12. For comparison, the change in the measured water level in SPP 2 compared to the baseline readings has been plotted as a pressure. A comparison of the pressures in VWP 5 and VWP 6 indicated that the excess pore pressure generated at the location of VWP 6 due to fill placement took approximately 3 months to dissipate.

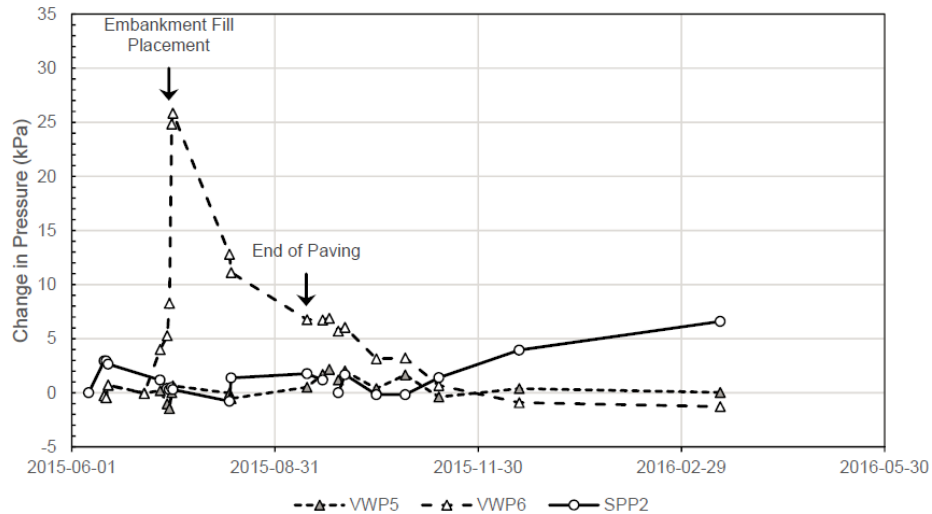


Figure 12 - Piezometric pressure versus time.

Regarding the inclinometers monitoring, the inclinometers have indicated a maximum lateral movement within the silty clay deposit of approximately 2 mm

INTERPRETATION OF MONITORING RESULTS

The settlement analyses completed by Geosolv for the design-build indicated that the settlement magnitudes within 25 metres of the abutments (i.e., where Armorpack elements were installed) were potentially up to about 30 mm, which is significantly less than the 160 millimetres (100 mm primary and 60 millimetres secondary) of settlement estimated by the foundations designers for the unimproved ground. The results of the monitoring to date indicate that settlement magnitudes of up to about 30 mm (and as little as 10 mm) have occurred to date adjacent to the abutments.

Further (beyond 25 m) from the abutments (i.e., where Rampact elements were used), the Geosolv design analyses indicated that the settlement magnitudes could potentially range up to about 50 mm. The measured settlement magnitudes at distances greater than 25 metres from the abutments are up to 45 millimetres and are more generally less than 30 millimetres.

The results of the VWP monitoring indicate that the pore water pressures returned to at or near the pre-construction values within 3 months after the placement of the embankment fill.

The inclinometer monitoring indicates negligible movements confirming embankment stability.

The results therefore seem to indicate that the aggregate piers have performed as designed and reduced the magnitudes of post-paving settlements to within acceptable values and resulted in a reduced time for those settlements (i.e., consolidation of the clay soil) to occur. The results

may also indicate that the disturbance of the sensitive clay was not a significant factor, since if the disturbance extended laterally to a significant extent across the separation distance between elements, the settlement magnitudes would likely have been higher.

It is not possible to evaluate the long term performance at this point and there do not appear to be any clear trends based on this data.

CONCLUSIONS

Foundation Engineers are challenged to design and construct embankments over weaker, compressible soils. There are several methods in the Foundation Engineer's tool box to ensure a safe, reliable embankment design that satisfies embankment performance criteria. The success of ground improvement technology employed at the Hwy 15/Crosby Creek project is demonstrative that ground improvement is a viable alternative that can accelerate construction and not compromise long term performance. Ground improvement reinforces the MTO priority that encourages sustainability of our infrastructure by investing in innovation and making smart investment decisions. As an alternative to other conventional methods, ground improvement techniques demonstrates harmony with the environment, and is less intrusive minimizing disturbance associated with partial or full subexcavations, groundwater drawdowns, haulage of spoil and backfilling of materials.. Ground improvement techniques require some time at the beginning of the project but can save on the overall schedule because there is no requirement for preloading or preloading/surcharging.

The benefits of ground improvement translates into projects being built faster and cheaper and is considered cutting edge technology for highway engineering projects at the MTO.

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