Geotechnical challenges associated with the design and construction of the new Champlain Bridge in Montreal

Riad Diab, Taravat Kashi Ghandi, Louis D'Amours & Jean Tardif
SNC-Lavalin, Montreal, Quebec, Canada

ABSTRACT
In 2015, the Canadian government awarded a $4.3 Billion Design-Build contract to the Joint Venture (JV) team Signature on the Saint Lawrence (led by SNC-Lavalin) for the design and construction of one of the largest infrastructure projects in North America, the new Champlain Bridge in Montreal. The complexity of the project posed unique geotechnical challenges on many levels. Some of the challenges are discussed in this paper. The overall project involved the construction of many smaller bridges, retaining walls, construction of roadway embankment over an old landfill, widening of existing highways over an old 11x5 m collector with unknown structural conditions, and designing foundations in potentially liquefiable soils.

This paper addresses the issues encountered and concerns raised during the geotechnical design and how these were addressed and resolved. Design and construction procedures, challenges and solutions are discussed in detail.

RÉSUMÉ
En 2015, le gouvernement canadien a attribué un contrat de conception-construction de 4,3 milliards de dollars au consortium Groupe Signature sur le Saint-Laurent (dirigée par SNC-Lavalin) pour la conception et la construction de l'un des plus grands projets d'infrastructure en Amérique du Nord, le nouveau pont Champlain à Montréal. La complexité du projet a posé des défis géotechniques uniques à plusieurs niveaux qui sont discutés dans cet article. Le projet global comprend la construction de nombreux petits ponts, des murs de soutènement, la construction d'un remblai routier sur un ancien site d'enfouissement, l'élargissement des autoroutes existantes sur un ancien collecteur de 11 x 5 m avec des conditions structurelles inconnues et la conception de fondations dans des sols potentiellement liquéfiables.

1 INTRODUCTION

The existing Champlain Bridge, constructed in 1958, across the St-Lawrence River between Brossard and Montreal required replacement as a result of age and critical structural conditions. The new Champlain Bridge, named Samuel de Champlain, was constructed by a joint venture, Signature on St Lawrence, comprised of SNC Lavalin, Dragados Canada, and ASC Infrastructure, partnering with SNC Lavalin, WSP and T.Y. Lin International as the lead design firms, and was opened to traffic on July 1, 2019. It was one of the largest infrastructure projects in North America and is one of the busiest crossings on the continent.

In addition to the new 3.5 km "main" bridge, the overall project involved the construction of tens of smaller bridges, including the 8-span, 470 m long, Ile des Sœurs (Nun's Island) Bridge, and over 20 retaining walls along the bridge approaches. The project also included the widening of highway 15 between the Turcot Interchange and the new bridge, and the improvement of the ramps leading from Highways 132 and 10 on the South Shore (Brossard) to the new bridge. Figure 1 below show the location of the project limits.

As sown on Figure 2, the new bridge's north span (towards Montreal) carries the northbound and westbound traffic of highways A-10, A-15, and A-20. The south span (towards Brossard) carries the southbound and eastbound traffic of highways A-10, A-15 and A-20. The central span, still under construction, will carry the South Shore branch of the forthcoming Réseau express métropolitain (REM), a 67 km automated light rail system, also designed and constructed by a joint venture led by SNC Lavalin.
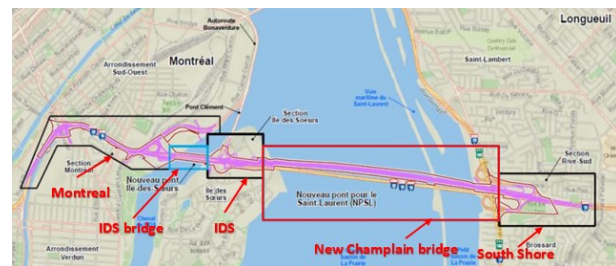

Figure 1. Champlain Bridge alignment

As per Transport Canada (client) requirements, the main standards to be used for the geotechnical design were the Canadian Highway Bridge Design Code (CSA-S6-14), the Canadian Foundation Engineering Manual (CFEM 2006), and AASHTO LRFD Bridge Design Specifications (2014), in decreasing order of precedence. The new bridge design life is 125 years and is classified as

lifeline bridge according to S6-14 importance category. The seismic design was to be performed for a return period of 2% in 50-year.

Figure 2. New Champlain Bridge

The bridge had to be designed and constructed in a 42-month timeframe. The complexity of the project posed unique geotechnical challenges on many levels. This paper presents some of the design and construction challenges.

2   SITE GEOLOGIC CONDITIONS

The general geology at the site consists of 1 to 4 m of glacial till deposit overlying the bedrock encountered at depths varying between 2 and 12 m.

The rock type is shale from the Utica formation. The surface of the bedrock is altered and fractured for depths varying from 1 to 5 m.

3   FIELD INVESTIGATIONS AND CHALLENGES

The design of the new bridge required extensive geotechnical investigations in order to use construction techniques adapted to the timeline.

The geotechnical investigation consisted of over 200 borings (including 69 offshore), over 40 Cone Penetration Tests, including 15 with seismic wave velocity measurements (SCPT). Over 80 pressuremeter (in soil) and dilatometer (in rock) profiles were performed at generally 1 to 1.5 m interval. Numerous undrained shear strength profiles were developed with Nilcon vane shear tests in the cohesive soil.

Depending on the size and importance of the structure, the design-phase investigation included one to three borings per substructure. Most of the "main" bridge substructures required 3 borings. All borings included a minimum of 10 m of rock coring, but in no case less than 5 m below the pre-determined shaft or bottom of footing elevations.

Geophysical investigation consisting of P&S sonic waves and downhole tests were performed in 9 and 6 borings respectively. The P&S method proved to be efficient to locate the fractured rock zones, as well as the numerous intrusive rocks intercepted by the borings.

In order to enhance recovery in the very dense till deposit and locate more precisely the interface between the till and the shale bedrock, a special drilling technique with the PQ3 core barrel having a specially designed diamond core bits were used to obtain, in one core run, almost intact sample of the till, fractured rock, and sound rock underneath. This technique proved to be very useful in establishing the limits between these geologic elements.

Laboratory tests were carried out on representative soil and rock samples to determine their physical characteristics and geotechnical properties. In addition, rock core samples were inspected and logged by an experienced geologist. A detailed record of the rock structural descriptions, including condition, orientation, spacing, weathering and alteration of discontinuities, was performed and used for the determination of the Geological Strength Index (GSI) and Rock Mass Rating (RMR) used for foundation design. The CERCHAR test was also performed on rock samples to determine the Abrasiveness Index (CAI) using steel styluses with a Rockwell Hardness Scale (RHS) of 55.

Carrying out the subsurface investigation presented a unique challenge. Over 80 SPT and CPT borings were drilled offshore with limited access to the river and very stringent environmental requirements (no sediment in water, biodegradable oil and fluids for drill rig, etc.), with the involvement of numerous agencies such as Transport Canada, St. Lawrence Seaway, Federal Bridge Agency, Hydro-Quebec, and others.

4   FOUNDATION TYPE

Deep foundation system, consisting mostly of drilled shafts socketed into rock, was the most suitable foundation type for supporting the bridges except for some of the "main" bridge substructures that were supported by spread footings on rock.

Figure 3. Main bridges segments

The drilled shaft diameter varied from 0.6 m to 2.2 m with the largest diameters used for the 8-span Ile des Sœurs (IDS) bridge.

The "main" bridge is 39 spans and comprises three consecutive segments: The West Approach, the Cable Stayed Bridge (CSB), and the East Approach (see Figure 3 above).

The Cable Stayed Bridge (CSB) is approximately 500 m long and crosses the navigation channel of the St. Lawrence Seaway between an existing dike and the Couvée Islands Bird Sanctuary.

The Main Span Tower (MST) is supported by 42 drilled shafts having 1.18 m diameter and socketed about 4 m into sound bedrock as shown on Figure 4.
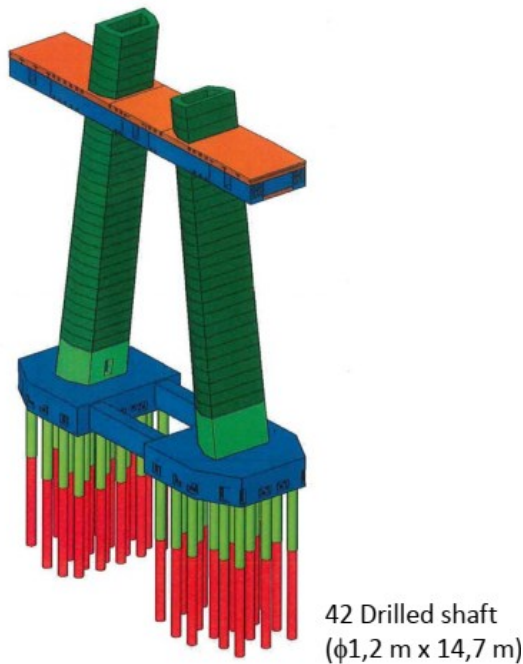

42 Drilled shaft
(φ1,2 m x 14,7 m)

Figure 4. Main Span Tower foundations

The West Approach (towards Montreal) is about 2 km long from IDS to the CSB with 28 substructures designated as WB (West Abutment), and W01 to W27. All supported by spread footing on rock (See Figure 5) apart from W01, W02, and WB where 1.18 m diameter drilled shafts were the selected foundation type.
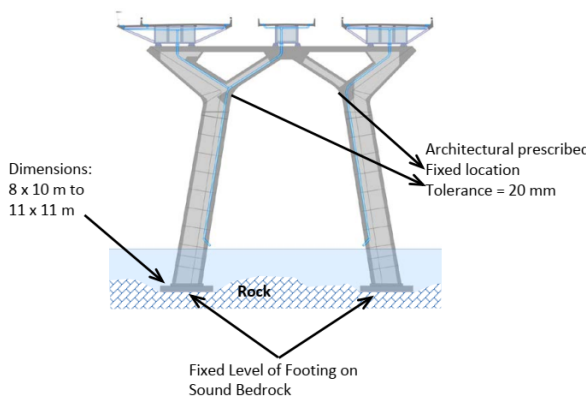

Dimensions:
8 x 10 m to
11 x 11 m

Architectural prescribed
Fixed location
Tolerance = 20 mm

Rock

Fixed Level of Footing on
Sound Bedrock

Figure 5. Typical "main" bridge shallow foundation

The East Approach is 770 m and extends from CSB to Brossard Shore with 11 substructures designated as EB (East Abutment) and E01 through E10. This approach includes three types of foundations: driven piles, drilled shafts and spread footings. Substructures E02 to E06 are on shallow foundations bearing on rock; EB and E07 are on groups of 1.18-meter diameter drilled shafts; and E08 to E10 on groups of 0.4-meter diameter driven piles.

5   DRILLED SHAFT LOAD TESTING

In order to optimize the drilled shaft geotechnical design and allow the use of higher resistance values for axial capacity, two Osterberg-cell (O-cell) tests were performed on July 1st and August 25, 2015 by Loadtest USA on sacrificial drilled shafts at the West Abutment and at the Main Span Tower (MST). The purpose of the tests was to determine the design values of side and base resistance at the ultimate limit state on drilled shafts constructed using means and methods which are identical to those to be used on production foundations. Both tests yielded similar results. Brief description of the test results and analysis at the MST is described herein.

The construction procedure is described below in Section 9.2. The tested shaft diameter and depth were 1.18 m and 19.6 m respectively, with socket length of about 5 m. The tests were performed using one (1) 13.8 MN bidirectional embedded jacks (O-cell) to load the base area of the shaft against the side resistance of the socket above the base.

At the maximum load, the displacements above and below the O-cell were 4.81 mm and 12.85 mm, respectively. Those displacements corresponded to maximum downward and upward loads of 20.53 and 20.18 MN. Seven levels of two sister bar vibrating wire strain gages were attached diametrically opposed to the reinforcing cage, including four in the rock socket area below the steel casing.

The data showed the maximum unit skin friction and unit end bearing mobilized during the test along the rock socket were 1.6 and 32.4 MPa, respectively.

Typically, the design approach for side resistance relates the unit side resistance, f_s and the square root of the unconfined compressive strength of the bedrock, $\sqrt{q_u}$. The method contained in Turner (2006) is presented as equation (1):

$$f_s = C.P_a.\sqrt{\frac{q_u}{P_a}}\qquad\qquad[1]$$

Where P_a is the atmospheric pressure (101 kPa), C is an empirical constant, and q_u is the unconfined compressive strength of the rock measured to be equal to 25 MPa at the load test location.

The CFEM (2006) recommends a range of values for the parameter C between 0.63 and 1.41 reflecting the variability of test results obtained by different authors, whereas AAHTO recommends the use of a value of 1, based on the most recent regression analysis of available load test data that is reported by Kulhawy et al. (2005).

One of the objectives of the load test was to calibrate the empirical parameter C against the results of the O-cell

test, at the project site for shale. Based on the load test results, a value of C equal to 1 (consistent with AASHTO recommendations) was back-calculated from the equation.

The O-cell test results also showed that approximately 14 MN in end bearing was mobilized at the vertical displacement at which the maximum skin friction was mobilized (see Figure 6). Thus, approximately 23 percent of the total resistance were provided by the end bearing at displacements corresponding to maximum mobilized skin friction.

The CFEM (2006) recommends the use of the Pells and Turner (1979) approach that is based on the theory of elasticity, to determine the load distribution between the end bearing and side shear. Using this approach, the tip resistance contribution would be approximately 12%. Therefore, the load tests showed the load transferred to the tip larger than that predicted by Pells and Turner approach, allowing the design to be optimized and resulting in a significant cost saving.
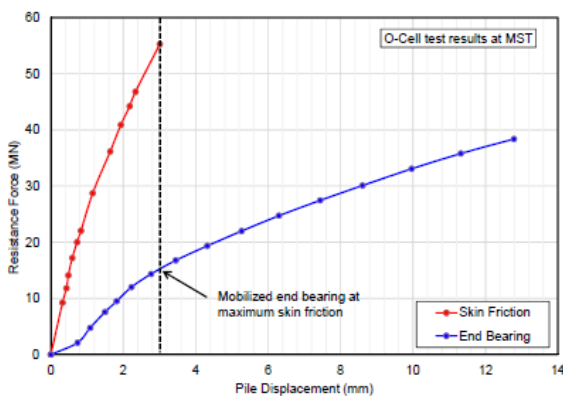

Figure 6. Skin friction and end bearing from O-cell test

6   SAINT-PIERRE COLLECTOR PROTECTION

Widening of Highway 15 in Montreal was planned as a part of the project. Most of the widened part was to be constructed over an existing old collector called Saint-Pierre Collector (CSP), extending over 2.5 km between the Turcot Interchange and Gaetan-Laberge Boulevard. This combined sewer overflow discharges the sanitary sewage and storm water runoff to the Atwater Water Pollution Control Plant Treatment. The CSP is a double reinforced concrete culvert with an average height of 4.6 m. Figure 7 shows the CSP during construction in 1933. Since the structural condition of the CSP was unknown, the collector owner's requirement was not to increase the stress on the collector due to the new construction.

As a result, the geotechnical design for Highway 15 above the CSP was prepared in such a way that no additional load would be applied on the Collector. This was achieved by the compensating fill approach which consists of removing a portion of the existing fill and installing lightweight engineered fill material to the required grade such that the Collector experiences no increase in applied stresses. The lightweight fill used for this purpose was composed of expanded polystyrene (EPS) blocks. Since the cost of the EPS blocks per cubic meter is much higher

than the regular earth fill material, and since the CSP extends over several kilometers along the project corridor, minimizing the use of EPS in the embankment would represent a large cost saving to the project.

Figure 7. Construction of Collector Saint-Pierre in 1933.

Due to the complex geometry, the design and optimization of the EPS configuration over the CSP was investigated using numerical simulations conducted using a 2D finite difference program FLAC8.0. The purpose of the study was to determine the optimum influence zone of the collector beyond which any earthwork would have no stress increase on the collector. Figure 8 below is a typical representation of the use of EPS for the highway widening that results in no stress increase on the collector.
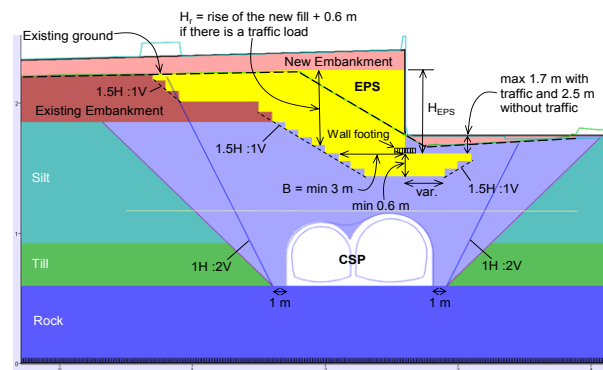

Figure 8. Typical EPS use for highway widening

To accurately estimate the stresses induced by the new construction on the collector, staged construction was applied in the numerical analyses and a stress hardening soil model was used to account for soil under unloading and reloading conditions. Initial conditions were modeled as horizontal layers of rock, till and fill material before building the CSP. Initial stresses were produced and followed by Plastic-Hardening analysis of the till layer. At the next stage, the collector and the backfill were built and then followed by another Plastic-Hardening analysis. The Plastic-Hardening model is a shear and volumetric hardening constitutive model used for the simulation of unloading-reloading situations. The existing embankment and retaining structures were then applied to the model and consequently stress and deformation analyses were

performed by applying a 17.6 kPa traffic load. At the final construction stage, the planned new fill was analyzed, and the final predicted traffic load was applied.

The soil stiffness parameters were back-calculated based on in-situ settlement measurement performed at one of the retaining walls, designated as MS-1, located in the vicinity of the studied segment area. Settlement platforms were installed, and settlement were recorded during the staged construction of the wall. The nonlinear modulus parameters of the soil were then calibrated by trial and error using FLAC. All stages of the MS-1 wall construction, including the initial condition, the excavation, and the wall layers were modeled in FLAC using the stage construction technique.

Different configurations of EPS blocks were introduced into the model to determine the minimum EPS volume needed for each representative cross section in order to avoid applying any additional load on the CSP as a result of the new embankment.

The analyses showed that generally, any load applied on the surface outside of the 1H:2V line did not have a significant effect on the horizontal and vertical stress on the collector.

Figure 9 represents a typical use of EPS over the CSP. As shown, in this particular section, the existing highway was to be widened by over 15 m. Within the 1H:2V zone of CSP influence part of the existing fill was excavated and EPS was used as backfill material up to the proposed grade in order to not increase the stress on the CSP. A standard Mechanically Stabilized Earth (MSE) wall was then constructed just behind the EPS and outside the zone of influence, to complete the highway widening.
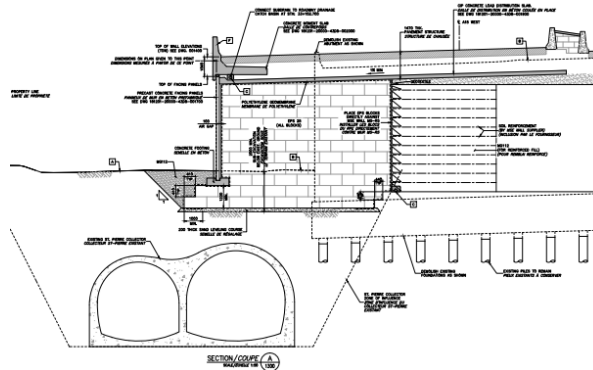

Figure 9. Use of EPS within the CSP zone of influence

The numerical simulation provided detailed information on the stress and deformation analysis of the cross section, as compared to analytical analysis, and helped reduce the project cost by over $5M.

7   CONSTRUCTION OVER OLD LANDFILL

The proposed Highway A15 between Gaetan-Laberge Blvd in Montreal and Ile des Sœurs (IDS) Bridge (about 550 m long as shown on Figure 10) was to be constructed along a new alignment where an old waste solid municipal landfill existed. The proposed highway profile was to be 2 to 7 m above the existing ground. Three single-span new bridges were to be constructed in this area. As a result of the poor soil conditions, these structures were to be founded on drilled shafts socketed into rock. Due to right-of-way limits, most of the new highway were to be supported by MSE Walls.

Over 25 borings were drilled in this area. The overburden consisted of heterogeneous fill material, up to 9 m thick, consisting of silt, sand and gravel with considerable amount of debris containing wood, steel, plastic, concrete, glass, metal, brick, garbage etc. The proportion of the debris was variable but could reach up to 90 percent in some zones within the fill. The underlying layer was a thin native glacial till soils underlain by the shale bedrock.
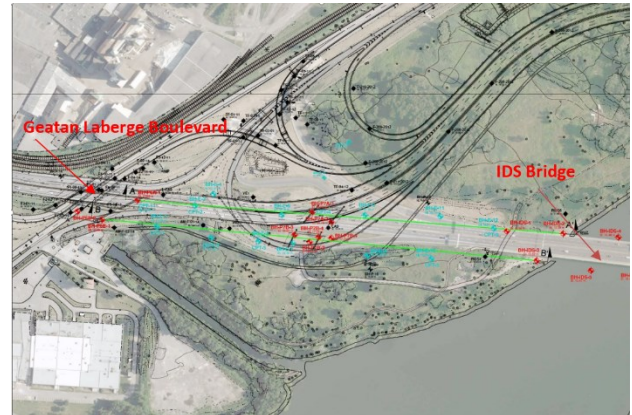

Figure 10: Old landfill location

Several ground improvement techniques were discussed with the construction team (CJV) including:

- Over-excavation and replacement
- Dynamic compaction
- Vibro-compaction (or vibro-replacement)
- Column supported embankment
- Lightweight fill
- Preloading

Due to the thickness of the heterogeneous fill along proposed highway, replacement would not have been practical and could have been too costly and problematic due the contaminated nature of the material and the presence of groundwater.

The effectiveness of dynamic compaction or vibro-compaction to achieve a suitable bearing material was questionable, particularly due to wood layers that could act as "springs" and prevent any increase in density of the underlying soils.

Although column supported embankment and/or the use of lightweight fill such as EPS would have been the best options, they are very costly. Therefore, CJV requested the design team to analyze the feasibility of the preloading option with phase construction.

A thorough analysis of the existing material properties was then conducted by the design team. The main geotechnical challenge was to estimate the strength and elastic parameters of the overburden needed to estimate the downdrag forces along the shafts of the three structures in this area, as well as the bearing capacity and settlement due to the placement of the new roadway fill on the heterogeneous fill.

Due to the nature of the debris material, the material properties could not be determined based only on the SPT N values. Therefore, SPTs were supplemented by pressuremeter tests to better predict the geotechnical properties and obtain more reliable estimate of the friction angle and the elastic properties. Over 25 pressuremeter tests were carried out in the area. The friction angle, φ, was estimated based on the following equation, proposed by Centre d'Études Ménard in 1970:

$$P_l = 2.5x2^{\frac{\varphi-24}{4}} \qquad [2]$$

Where P_l is pressuremeter limit pressure.

Based on the above equation, a friction angle of 29 degrees was estimated for the debris layer.

In order to better assess the settlement behavior of this material a 7 m temporary embankment fill (causeway) was constructed across the landfill during the design phase. The causeway served also to allow traffic during the demolition and reconstruction of the existing IDS Bridge. To monitor the settlement of the causeway, a total of 32 settlement platforms were installed along the causeway. The locations of the platforms were selected as close as possible from boring locations to facilitate correlation with boring data.

All settlement platforms showed mostly a clay-like behavior, as shown on Figure 11 below, which could be summarized as follows:

- "Primary" settlement, completed within 65 days
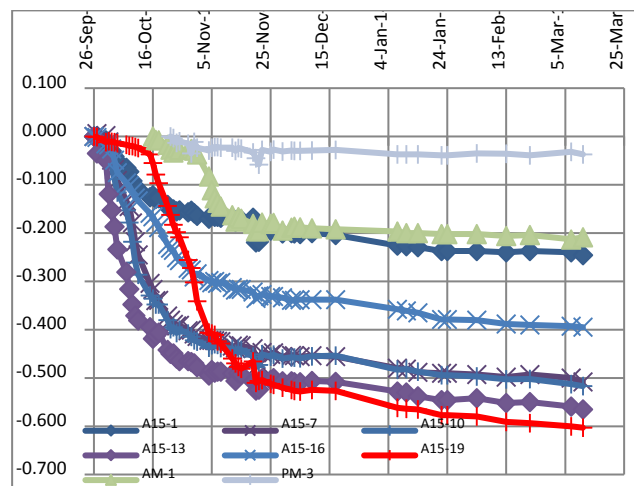- "Secondary" settlement rates of about ±0.55 mm/day.

Figure 11. Typical settlement over time curves

A coefficient of "consolidation" c_v of 0.6 m²/day was then back-calculated from those results. A correlation between c_v and landfill thickness H was developed using exponential curve fitting technique and could be expressed by:

$$c_v = 0.08.e^{0.18H} \qquad [3]$$

A secondary "consolidation" settlement rate, C_α, of 0.15 was also back-calculated. The value of C_α was found to be independent from the landfill or roadway fill thicknesses.

Although the secondary settlement was of concern for the project lifetime that needed to be dealt with, preloading and phase construction was the selected ground improvement alternative for the embankment construction.

As a result, all wall construction was proceeded in two phases to allow settlement to occur prior to final construction. In the first stage, a wire faced MSE wall was constructed. In the second stage, the panel facing was installed. The first phase involved placing a surcharge corresponding to the total height of the wall plus the equivalent of 1.5 m (30 kPa) on the existing ground. At this stage settlement platforms were installed during embankment construction to determine the magnitude and rate of settlement as describe in ASTM D6598. The settlement was monitored on weekly basis (twice a week) from the date of installation of the first device until the rate of settlement at each settlement platform was less than acceptable limits, generally less than 5 mm/week over two-week period. Then, in the second construction phase, the surcharge was removed, and construction was finalized by placing the facing 500 mm away from the front of the wall as well as the concrete barrier and the pavement on top of the wall. The 500 mm gap was then backfilled with BC 5-20 aggregate as specified in BNQ 2560-114. Figure 12 below illustrates the phase construction of one of the retaining walls designated as MS-R7.
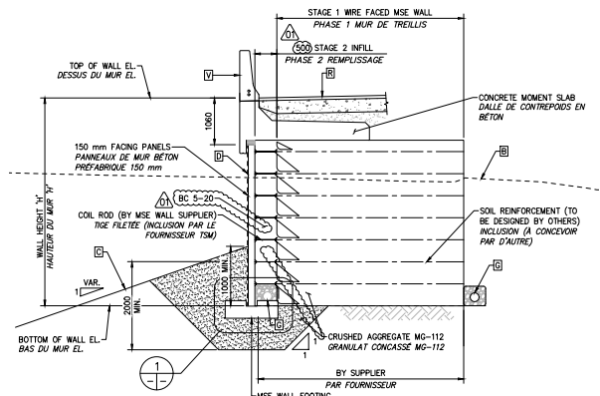

Figure 12. Staged construction of MSE Wall

The settlement magnitude and duration for the above mentioned 7 m height wall was about 150 mm and 6 months, respectively, consistent with the results obtained during the design phase.

A numerical modeling analysis, using the finite element software SIGMA 2D, calibrated with the field data was performed to estimate settlement following phase 2 construction.

8   DESIGN IN POTENTIALLY LIQUEFIABLE SOIL

Liquefaction triggering evaluations were performed for 2475-year, 975-year, and 475-year return periods based on simplified method proposed by Youd et al (2001). A design decision by the geotechnical team was made to consider the soil as liquefiable when the factor of safety was less than 0.9 and not susceptible to liquefaction for a factor of safety larger than 1.2. Numerous areas along the project alignment were found to be liquefiable including the

soils at the "main" bridge West Abutment and at the East Abutment of IDS Bridge. For factors of safety between 0.9 and 1.2, the soil was considered as potentially liquefiable and required more detailed analysis. One such a case was the soil encountered between Wellington Street and Atwater Avenue in Montreal.

The subsurface conditions in this segment of the project consisted generally of fill material (silty sand) underlain by glacial till. In some areas, however, a silt layer was encountered between the fill and the till. The silt layer varied in thickness from about 2 to 8 m and had a density ranging from very loose to medium dense. The silt was estimated to be potentially liquefiable. Consequently, additional subsurface investigation was conducted mainly to determine the shear wave velocity, V_s profiles of the deposit using the seismic downhole method. Moreover, large soil samples were collected and sent to the University of Sherbrooke for detailed liquefaction analysis. The measured V_s in the field were utilized as references to reconstitute the laboratory soil samples to the same density as in the field. In other words, V_s tests performed at the site were used with laboratory measurements of V_s on the soil samples at different void ratios to reconstitute soil specimens at densities comparable to those experienced in the field. The samples were then subject to cyclic direct simple shear (DSS) test.

The tested silty soils had a plastic and liquid limit of 18 and 22 percent respectively, and an average clay content of 10 percent. Two specialized laboratory apparatuses were employed for this purpose: the piezoelectric ring-actuator technique (P-RAT) to measure V_s values of soil samples extracted from the site, and the TxSS seismic simulator to define the cycling shear resistance of soil samples and perform direct measurement of pore pressure.

Once the reconstituted samples were obtained, a series of strain-controlled undrained TxSS tests were performed to define the relationship between the cyclic shear strain (γ_{cyc}) and the applied number of cycles at a frequency of 4.0 Hz (close to the natural frequency of the deposit) until the soil liquefies. Initial liquefaction was defined as pore water pressure ratio, $R_u = \Delta u/\sigma'_c$ of 0.9, where σ'_c is the initial confining pressure and Δu is the excess pore pressure. The results for the reconstituted sample showed that liquefaction was likely to occur after 20 cycles.

In order to evaluate the cyclic resistance (CSR) according to the number of cycles, numerical modeling of the potentially liquefiable soil layers, using the computer code FLAC, was performed by applying directly earthquakes compatible with the Eastern Canada seismicity. First, a relationship between the energy dissipated during cyclic loading with the built up of the pore pressure was developed based on the TxSS tests for the tested soil. The dissipated energy per unit volume for a soil sample in cyclic loading was determined by integrating the area bound by stress–strain hysteresis loops. Second, this relationship was integrated into a dynamic response analysis through a constitutive law enabling it to properly describe the hysteresis stress-strain behavior of the soil under consideration and making it possible to simulate the induced pore water pressure ratio R_u for any seismic loading and assess their potential liquefaction hazard.

Once the numerical model was calibrated, the shear distortions obtained numerically using FLAC and the SIMQKE ground motion at the depths where the soil samples were extracted, were applied to these samples in the seismic TxSS.

The analysis showed that seismic loading of the silty soil at the site would generate groundwater pressure in the order of 17% of the initial effective pressure. Hence the soils at the site were not susceptible to liquefaction under seismic loading for a return period of 2 percent in 50 years, although some reduction of strength would occur.

8.1 Drilled Shaft Design in Liquefiable Soil

By mutual agreement between the construction team (CJV) and the designers, increasing the shaft length and/or diameter was selected as the most viable and cost-effective method to address the negative impact of liquefaction on drilled shafts. Therefore, seismic design of foundations in liquefiable and potentially liquefiable soils took into consideration the additional forces induced by liquefaction on the foundations. The major effects of liquefaction on deep foundations are the loss of lateral support in the liquefied zone, ground settlement, and lateral spreading, which would result in downdrag load and additional lateral load on the deep foundations.

Foundations located in liquefiable soil, exhibiting a factor of safety of less than 0.9, were analysed for lateral spreading and downdrag. On the other hand, these two phenomena were assumed to be negligible for foundations located in the silty soil discussed above.

Occurrence of lateral spreading depends on the earthquake properties (magnitude and acceleration), site conditions (ground surface slope and thickness of the soil layer that is anticipated to liquify) and liquefied soil conditions (grain size and percent fine.) The foundation would be loaded by a laterally displacing soil mass in conjunction with inertial loading from the superstructure. The liquefaction-induced lateral spreading was particularly investigated for the shafts near the shoreline at the IDS bridge East Abutment and the "main" bridge West Abutment. The lateral spreading demands were computed as the sum of the full passive soil pressure on the foundation and 50 percent of the inertial loading from the superstructure as suggested in Caltrans 2012.

The liquefaction-induced downdrag forces on shafts were also considered. The post liquefaction settlement was computed using the methodologies proposed by Wu and Seed (2004) using the $(N1)_{60}$ adjusted to a reference clean sand. The downdrag forces in the liquefying layer were then calculated using residual strength values estimated, as recommended by Caltrans (2012), using the Kramer and Wang (2007) equations.

Lateral resistance along drilled shafts were calculated using nonlinear *p-y* curves for soils and rock. Although several soil models are available in the literature to simulate the behavior of liquefied soil, a design decision was made to assume, conservatively, a *total* loss of strength of the liquefiable soil. As for the silty soil discussed above, the increase of pore water pressure in the soil resulting from earthquake loading, although does not cause liquefaction, but will reduce the effective pressure

and hence its resistance. As a result, lateral resistance of the silty soil was adjusted (lowered) such that the *p-y* curve parameters were modified as follows Miyamoto (1987):

$$K_r = K_{max}\sqrt{(1 - R_u)} \qquad [4]$$
$$P_{ur} = P_{u-max}(1 - R_u) \qquad [5]$$

Where K_r and K_{max} are the reduced and static lateral subgrade reaction respectively; and P_{ur} and P_{u-max} are the reduced and static ultimate lateral resistance, respectively.

9 CONSTRUCTION CONSIDERATIONS

9.1 Driven Piles

Closed end pipe piles driven to refusal on the shale bedrock were used for some substructures. As a result of the pile terminating on bedrock the design of the pile was based on the structural resistance of the pile.

It is well documented in the literature that some shales, particularly weathered shale, exhibit relaxation (decrease in resistance) phenomenon occurring during and after pile driving, generally attributed to a release of locked in horizontal stresses (Thompson and Thompson, 1985). Hence, some practitioner engineers request the piles to be driven to a capacity (or pile penetration resistance) in excess of the required nominal resistance to account for the future loss of nominal resistance.

Thus, the nominal resistance was carefully assessed thorough dynamic testing program. The driving criteria was determined initially by running the wave equation program GRWEAP and then confirmed by dynamic testing. Restrike was performed on all piles, a minimum of 3 days after initial driving. The piles were re-driven until the established termination criteria was reached. The vertical displacement occurring between the end of initial driving (EOID) and the end of restrike (EOR) was measured for all piles. Piles that exhibited displacement larger than about 3 mm were re-struck again until the measured displacement fell below the acceptable limit.

Dynamic tests were performed on selected piles at both the EOID and EOR, and in some cases at the beginning of restrike (BOR) in order to quantify time dependent changes in nominal resistance.

One example is the structure carrying Highway 15 over René-Lévesque Boulevard in Île-des-Soeurs (Nun's Island), designated as P11. A design load of 1750 kN was required. The results of GRLWEA program indicated that 31.2 kN drop hammer with 4.6 m stroke would be able to mobilize the nominal resistance of 3500 kN with stress levels below the minimum yield strength of 350 MPa at refusal conditions of 5 blows/1 mm.

The mobilized static resistance at the EOID for one of the piles on axis 1, based on the Case Method solution, was 3600 kN, exceeding the 3500 kN required. This was substantiated by signal matching analysis (CAPWAP) which indicated a nominal resistance of 3670 kN, with most of the resistance (3380 kN) carried by the toe. The nominal resistance at the BOR, based on the Case Method and CAPWAP solutions, was roughly 3100 kN, or 500 kN less than at the EOID, indicating large relaxation occurring between the EOID and BOR.

The pile was then re-driven until the required refusal criteria of 5 blows/1mm was reached again. The measured vertical displacement to achieve the refusal criteria was 204 mm. A second re-drive of the pile was performed 3 days later where the displacement dropped to 58 mm. The third re-drive showed a displacement of less than 3 mm. At the end of the re-strike the mobilized resistance, as obtained by CAPWAP, had increased from 3670 kN to 3855 kN.

10 CONCLUSIONS

Replacement of the existing bridge across the St-Laurence River in Montreal posed significant geotechnical problems that were largely unsuspected prior to the geotechnical investigation and proposed bridge construction. The bridge had to be designed and constructed in a 42-month timeframe. The following concluding remarks could be made:

- The use of expanded polystyrene (EPS) blocks as lightweight fill along with numerical simulations for quantity estimate deemed very useful for the design optimization of the old collector protection.
- staged construction was a viable alternative to expansive ground improvements regarding the placement of the roadway embankment over an old waste solid municipal landfill.
- Laboratory cyclic shear tests conducted on potentially liquefiable soils helped determine the percentage loss of soil resistance and optimize the foundation seismic design.
- The O-Cell tests were a major contribution in optimizing the design of the drilled shafts socketed into in shale.

11 REFERENCES

Caltrans (California Department of Transportation) (2012). *Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading*.

Karray, M. Hussien, M., and Chekired, M. 2016, *Characterization of the Dynamic Properties of Soils at the New Champlain Bridge Corridor Project Using the TxSS Seismic Simulator* University of Sherbrook Report No. Geo-03-16, presented to SNC Lavalin.

MIYAMOTO, Y., 1987. *Pile Response During Earthquake and Performance Evaluation of Pile Foundation,* Kobori Research Complex, Kajima Corporation, Tokyo, Japan.

Thompson, C.D. and Thompson, D.E. (1985). *Real and Apparent Relaxation of Driven Piles*. American Society of Civil Engineers, Journal of Geotechnical Engineering, Vol. 111, No. 2, 225-237

Wu, J. and Seed, R.B. (2004). *Estimation of liquefaction-induced ground settlement (case studies)*, p*roceedings*, Fifth International Conference on Case Histories in Geotechnical Engineering, New York.