Geotechnical issues encountered during the design and construction of the REM project in Montreal

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ABSTRACT
In 2018, the Caisse de Dépôt et Placement du Québec awarded a $6.3 Billion Design-Build contract to the Joint Venture team NouvLR for the design and construction of a 67-km light rail system called the REM in Montreal, which will be one of the largest automated transportation systems in the world. The complexity of the project posed unique geotechnical challenges on many levels.

Part of the alignment was constructed over peatland where low bearing capacity and excessive settlement obstacles needed to be overcome. Many utilities, running in the vicinity of proposed embankments, required the design of a protection system by mean of column supported embankment. Ground improvement by mean of semi-rigid inclusions was to be utilized for embankment constructed over soft cohesive soil. Liquefiable soils were encountered in some areas where the dynamic response was to be assessed. This paper addresses the issues encountered during the geotechnical design and how these were addressed.

RÉSUMÉ
En 2018, la Caisse de Dépôt et Placement du Québec a attribué un contrat de de $6,3 milliards au consortium NouvLR pour la conception et la construction du réseau de métro léger de 67 km (REM) à Montréal, l'un des plus grands systèmes de transport automatisés au monde. La complexité du projet a posé des défis géotechniques à plusieurs niveaux.

Une partie du tracé a été construite sur une tourbière dont des obstacles de tassement excessifs et de faible capacité portante devaient être surmontés. De nombreux services d'utilité publics au voisinage des remblais proposés ont nécessité la conception d'un système de pieux. L'amélioration du sol au moyen d'un système de renforcement de sol par inclusions semi-rigides devait être utilisée pour les remblais construits sur des sols cohérents. L'analyse dynamique devait être effectuée pour des sols liquéfiables rencontrés aux quelques secteurs. Cet article traite des problèmes rencontrés lors de la conception et des solutions envisagées.

1 INTRODUCTION

Réseau Électrique Métropolitain (REM) is being constructed by a Joint Venture comprised of SNC Lavalin, Dragados Canada, Group Aecon Quebec, Pomerleau, and EBC, partnering with AECOM and SNC Lavalin as the lead design firms. The project, referred to as REM, is a fully automated light rail transit (LRT) proposed by the Caisse de dépôt et placement du Québec (CDPQ) Infra, to serve the major metropolitan areas in Montreal, Canada. Once completed, the 67 km REM will be one of the largest automated transportation systems in the world.

As shown on Figure 1, the new facility will link downtown Montreal, South Shore, West Island (Sainte-Anne-De-Bellevue), North Shore (Deux-Montagnes), and the Montreal-Pierre Elliott Trudeau International Airport.

The project comprises four segments: South Shore (SS) segment, with a total length of 15 km, will extend from downtown (Central Station) to the DIX30 commercial district passing across to Nuns' Island and then will use a rail deck constructed on the new Champlain Bridge (still under construction) to cross the St. Lawrence River. The Deux Montagnes (DM) segment will be mostly at grade and will consist of a direct conversion of the existing Deux-Montagne's line. The Sainte-Anne-De-Bellevue (SADB) segment will begin near highway A-13 and end at SADB Township with 17 km of elevated structure supported by two abutments and 340 piers. The airport segment will divert from SADB line to make a stop in Technoparc St-Laurent before terminating at Montréal–Pierre Elliott Trudeau International Airport.

Figure 1. REM project alignment

As per the client (CDPQ) requirements, the design standards to be used for the geotechnical design are the Canadian Highway Bridge Design Code (CSA-S6-14), the

Canadian Foundation Engineering Manual (CFEM 2006), and AASHTO LRFD Bridge Design Specifications (8th Edition, 2017), in decreasing order of precedence.

The project had to be designed and constructed in a 5-year timeframe. The complexity of the project posed unique geotechnical challenges on many levels. This paper presents some of the design challenges.

2 SITE GEOLOGIC CONDITIONS

The general geology of the layout consists of a till deposit overlying the bedrock at varying depths (2 to 17 m). The overburden soil overlying the till is mostly granular fill although clay deposits from the Champlain Sea up to 9 m in thickness is encountered in some areas. The surface of the bedrock is altered and fractured for depths varying from 1 to 3 m. Three rock types are encountered along the REM alignment: Limestone and dolomite along DM and SADB, and shale along the entire SS segment.

1 EMBANKMENT CONSTRUCTION OVER SOFT SOIL

A 3 to 11 m thick Champlain Sea clay deposit was encountered on the approach roadways leading to the bridge over Industrial Boulevard on the DM segment. The fill height to be placed on the existing ground to reach the proposed grade varied from 2 to 11 m with its maximum height at the abutments. The clay properties were determined from series of field and laboratory testing including vane shear and consolidation tests. The clay was of medium to high plasticity with average plasticity index of 40 percent, and an average water content of 55 percent. The strength and consolidation properties are shown on Table 1. The profiles of the effective stress and the pre-consolidation pressure are shown on Figure 2 for 8 m fill height.

Table 1. Champlain Sea clay properties

Soil parameters	Crust	Bottom clay
S_u (undrained shear strength), kPa	--	45 - 70
C_r (average recompression index)	0.11	0.08
C_c (average compression index)	1. 2	0.5
OCR	3 to 7	
C_α (secondary compression index)	0.06	0.025
C_v (coefficients of consolidation), m^2/s	1.7E-07	1.3E-08

The settlement analysis showed that unacceptable amount of settlement would occur during the construction of the embankment. Further, without consolidation acceleration measures, consolidation settlement duration would be expected to be in the order of several years.

To reduce the timeframe to a level which would be feasible for construction, the installation of PVDs along with the placement of adequate preload would be required to accelerate the consolidation to the degree necessary to satisfy the project construction schedule constraints. Moreover, staged construction would be required to prevent a shear failure in this material. The duration of each loading stage would be expected to be several months (for 4x100 mm wick drains placed in square pattern with spacings varying between 2.2x2.2 m to 1.0x1.0 m).

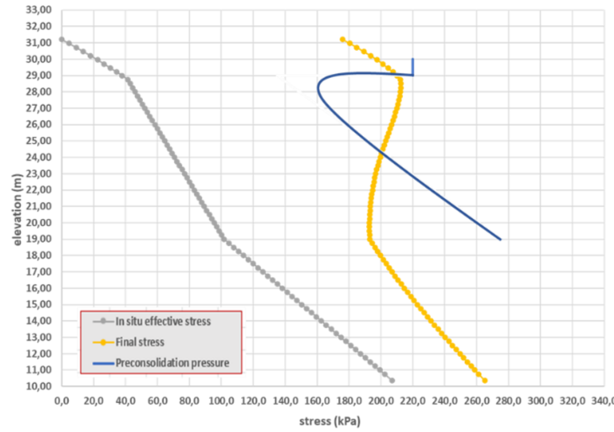

Figure 2. Effective stress and pre-consolidation pressure

Due mainly to differential settlement concerns caused by variable clay layer thickness and embankment height, as well as the tight construction schedule, an alternate solution, using Controlled Modulus Columns (CMC) inclusion, was proposed to the construction team. This alternative was found to be the most practical and cost-effective option.

CMC are semi-rigid inclusions installed using a specially designed proprietary hollow stem displacement auger through which grout or concrete is injected by pressure-grouting when tooling is being withdrawn. The requisite design and installation technology is patented by Menard and its parent firm in France. A load transfer platform (LTP) made of well compacted granular material is usually required above the CMCs to better distribute the loads through arching between the CMCs and the surrounding soils.

Unlike pile supported embankment, there is a sharing of load between the CMC and the surrounding soil, i.e. the columns are not intended to directly support the loads imposed by the embankment but rather to reduce the load sustained by the soil. Because of their high stiffness, the CMC columns attract high stresses compared to the surrounding soil. As a result, the improved soil acts as a composite material with an equivalent vertical modulus that depends on the soil and CMC properties, column spacing and diameters, thickness, and properties of the LTP.

At the request of construction team, the REM geotechnical team performed a preliminary design for the purpose of feasibility and cost estimate. The requirements for the ground improvement were to achieve a factored bearing resistance of 375 kPa and total and differential settlement of 50 mm and 1:750, respectively.

The preliminary design was based on the principle that the stress applied to the top of the column, q_c (as well as on the soil matrix) depends on the total stress, q, the stiffness ratio (expressed in units of pressure/deflection) between the column element and the surrounding matrix soil, R_s, and the percentage area covered by the columns, R_a, as per the following equation:

$$q_c = q\left\{\frac{R_s}{R_sR_a - R_a + 1}\right\} \qquad [1]$$

The settlement of the columns was then estimated from the quotient of the stress applied at the top of the column and the stiffness modulus of the columns. The settlement of the matrix soil, between the reinforcing element was computed using conventional geotechnical approaches derived from one-dimensional consolidation tests.

The stiffness ratio was estimated to be about 400 for CMC concrete compressive strength of 20 MPa. The elastic modulus of the granular LTP was assumed to be 30 MPa.

The final design was performed by Menard using the computer software PLAXIS 2D and an axi-symmetric model. The design was reviewed and approved by the REM geotechnical team. A combination of CMC and vertical drain were used as shown on Figure 3. The CMC were installed in a rectangular pattern with variable grid depending on the embankment height. The column spacing was 1.8x2.0 m at the maximum embankment height and increased gradually to 2.3x2.8 m at the end of the wall. The CMC column diameter varied from 0.42 to 0.32 m. The concrete used in the CMC had an unconfined compressive strength of 20 MPa and an elastic modulus of 20 GPa.

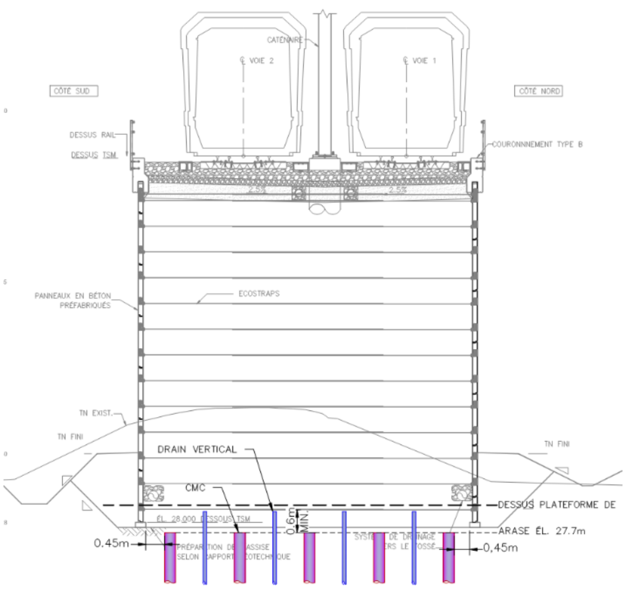

Figure 3. Typical CMC layout (near the abutment)

Considering the impact that settlements can have on the construction schedule and the service life of the roadway, construction of the approach roadway was instrumented using 22 settlement plates installed at the bottom of the embankment. Monitoring of settlement is still ongoing.

2 CONSTRUCTION OVER PEATLAND

A large area, called "Bois-de-Liesse" sector along DM segment involved doubling the railway line. The sector extends from the Bois-de-Liesse recreational trail, north of Highway 13 to the intersection of the railway line with the Alexander Street. This sector is characterized by the crossing of a wetland and the so-called Bertrand stream. The crossing of highly compressible organic or clayey soils under the existing CN (Canadian National) railway line (Track 1) and the adjacent proposed new line (Track 2) was to be taken into consideration. CN indicated that maintenance is being carried out on a regular basis for grade adjustment at Track 1.

Historical geotechnical data, confirmed by the new investigation, indicated the peat was identified as moderately to very strong decomposed (H6 to H8 according to Von Post classification system - ASTM D5715). The thickness of this layer varied from 0.3 to 0.6 m in the southern part to reach 2.0 to 3.3 m in the northern half of the area. Figure 4 shows the peat distribution along Tracks 1 and 2 with the dark red colour indicating the thickest area and the dark blue the thinnest area. The future level of the existing rail will be raised by 300 to 500 mm compared to its current level. The new track (Track 2) will be built on the west side of Track 1 with the fill height to be placed in the order of 3 to 5 m.

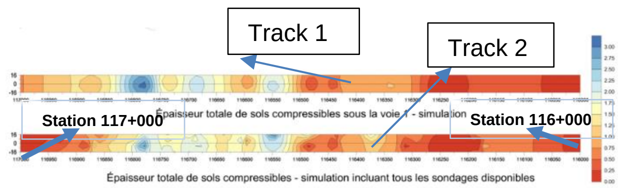

Figure 4. Peat thickness distribution along Tracks 1 and 2

The laboratory results indicated the peat was in normally consolidated state with water content varying between 104 and 171%. The organic content was generally over 60% so the material would be characterized as an organic material mixed with fine soils. The peat properties are summarized in Table 2 below. It is worth mentioning that the difference in strength properties between Tracks 1 and 2 is due to the fact that the peat under Track 1 was compressed and consolidated by the railway embankment and traffic loading during many years.

Table 2. Peat Properties

Soil parameters	Track 1	Track 2
C_r (average recompression index)	0.1 to 0.2	
C_c (average compression index)	0.5 to 1.5	
S_u (in situ shear strength), kPa	58 to 87	25
e (void ratio)	2.0	4.0
E_m (elastic deformation modulus), MPa	1.1	
γ (unit weight), kN/m^3	12.4	
C_α (secondary compression index)	0.025 to 0.01	
C_v (coefficient of consolidation), m^2/s	3.1E-07	4.4E-07

Different options of soil improvement were presented to the construction team at the beginning of the project. The options included preloading embankment, pile system supported by slab, CMC columns, light weight fill (EPS) and stabilized embankment with one or two layers of geogrid.

Analysis has confirmed that the use of preloading surcharge represents the most effective method of treating the soils in this sector so as to meet the project objectives in terms of budget and schedule.

It was necessary to evaluate the maximum height of embankment that it is possible to build on very compressible soils without risking a shear failure of the embankment. The following Anderson and Hemstock equation was used for this purpose:

$$H_{ult} = 5.5 * \frac{S_u}{\gamma} \quad [2]$$

Thus, using a factor of safety of 2, the backfill should not exceed 5.3 m in height. For this height, the limit equilibrium analysis using software SLOPE/W developed by GEO-SLOPE international LTD, with slope sides of 2H:1V, was performed to ensure that a minimum factor of safety of 1.5 against rotational slip is obtained.

The assessment of settlements was made by using the software Settle3. Figure 5 below illustrates the settlement distribution underneath Tracks 1 and 2.

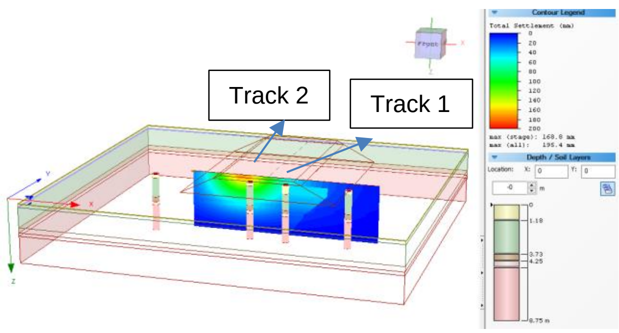

Figure 5. Typical settlement distribution under the tracks

Figure 6 illustrates the time rate settlement underneath Tracks 1 and 2 at the critical section where the peat thickness was about 3 m, and the embankment height was about 4 m.

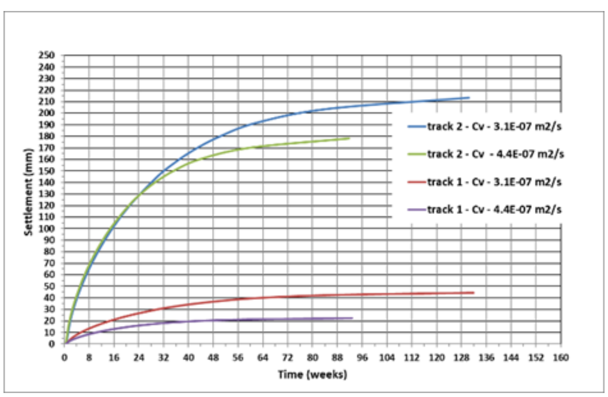

Figure 6. Time rate settlement underneath the tracks

The total consolidation settlement would be about 30 and 200-mm underneath Tracks 1 and 2, respectively. The majority of settlements would occur in the first six months. To these values must be added secondary settlement, which could reach 175 to 200 mm over a period of 30 years, at a rate of 5 to 10 mm per year. The magnitude and duration of settlement were deemed acceptable with regard to construction schedule.

Prior to the construction, number of groundwater wells were installed in peatland to assess the consolidation rate. Further, several settlement plates and displacement markers were installed for the purpose of monitoring the magnitude of settlement and lateral deformation over time. Results of settlement monitoring showed that the measured settlement was more than the estimated theorical settlement presented on Figure 6. As an example, the theoretical settlement, 24 weeks after fill placement at the critical section, was 140 mm whereas the measured settlement reached 169 mm. On the other hand, the measured consolidation time was somehow faster than anticipated. This can be explained by the difficulties in estimating the preconsolidation parameters (C_r and C_c) from the consolidation curves that showed signs of soil disturbance. This could also be explained by insufficient laboratory testing due to difficulties in extracting intact soil sample and assessing accurately the consolidation behaviour of the peat layer.

For instance, some longitudinal cracks were observed along the proposed Track 2 (See Figure 7 below) at a location where the embankment height was about 3 m. At the request of the engineering team, the construction works and the train operation over the existing railway got shut down in order to prevent any hazardous incident. A site visit by the REM geotechnical team was conducted and following an intensive investigation, it was concluded that there was no evidence of rotational slip nor shear failure, but rather the cracks were the result of differential settlement. As a matter of fact, the peat underlaying the existing Track 1 has been consolidated and compacted during years of the train operation while the peat underlaying the projected Track 2 has not been given enough time to consolidate and gain strength.

Figure 7. Crack appearance during construction

As a result of the incident, some remediation work was done by the construction team while monitoring the settlements. The failure area was leveled to the planned elevation and compacted. Furthermore, a berm was constructed to avoid any further movement as a precaution measure.

3 UTILITY PROTECTION

Numerous utilities run along the project alignment. In agreement with the respective owners, most utilities were relocated. However, a few numbers of small (less than 300 mm in diameter) and larger pipes were to remain, particularly the watermain and stormwater sewer pipes crossing Ile Bigras, O'Brien Street and Alexander Street. The pipe diameters at those locations varied between 900 and 1200 mm and the fill height to be placed on the existing ground were between 6 and 10 m.

The general geology at the three locations was similar and consisted of loose silty/sandy soil with variable thickness overlaying the till deposit. The rock was encountered at depths varying from 7 to 15 m.

The estimated settlement due to the fill placement exceeded 20 mm. The structural condition of the pipes was unknown, and the owner's requirement was not to increase the stress on the pipes due to the new construction. Figure 8 shows the pipe crossing at Alexandre Street where a back-to-back 7-m height wall was to be constructed.

Although the use of lightweight fill (EPS Blocks) could have been a viable alternative, Column Supported Embankment (CSE) approach was the more cost effective and therefore the selected utility protection approach. The design of the CSE was based on Collin et al. guidelines which are based mainly on the arching theory for fill soils and the tensioned membrane theory for the geosynthetic. Accordingly, the embankment load was considered to be supported entirely by the end-bearing columns and therefore, did not directly take into account potential support from the foundation soil between columns. A Load Transfer Platform (LTP) reinforced with multiple geosynthetic reinforcement layers was used to minimize the number of columns required to support the embankment. The LTP was designed to act as a beam to transfer the load from the embankment above the platform to the columns below such that soil arch is fully developed within the depth of the platform, and the strain in the geosynthetic reinforcement is small enough to preclude any differential settlement at the top of the embankment. Thus, the total load that each column was required to carry was based on the tributary area which is a function of the column spacing and layout configuration (rectangular or triangular pattern).

The selection of the type of column and diameter depended largely on the applied load (embankment height) and constructability. Due to the proximity of the pipe, driven piles were excluded to avoid any potential damage due to vibration. The selected pile type was 350 to 600 mm bored concrete piles sitting on the top of rock and placed in a rectangular pattern. A minimum clear distance of 450 mm was maintained between the pipe and the drilled piles. A 1.2 m wide individual precast pile cap was placed on top of each column (pile) basically to reduce the clear span between columns so that the thickness of the load transfer platform could be reduced to be equal to or greater than one-half the clear span as recommended by Collin et al.

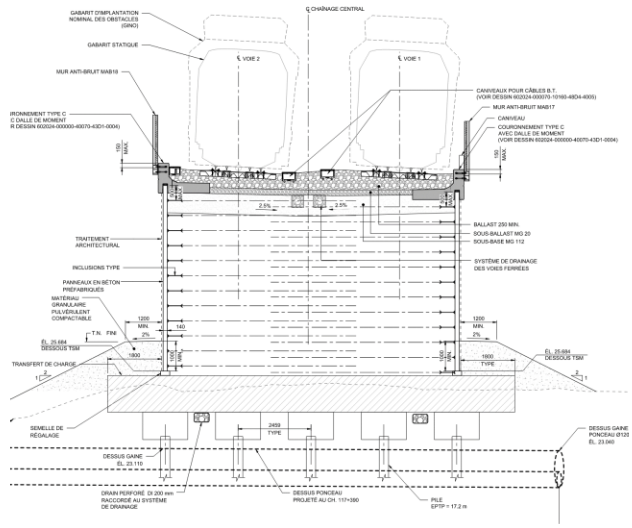

Figure 8. Utility crossing at Alexander Street

The LTP consisted of 1.2 m thick compacted select fill reinforced with four layers of bi-axial geogrid to stiffen the soil and develop beam-type action in transferring the embankment load to the columns. The vertical spacing between layers of reinforcement was 152 mm. A base reinforcement geotextile was installed to provide separation between the subgrade and the select fill and to support the soil below the zone of arching through catenary action. The factored tensile load applied at the base reinforcement geotextile varied between 245 and 400 kN per meter width, depending on the column spacing and the embankment height.

Depending on the geometry at each specific site, the LTP was supported by 2 or 4 rows of piles spaced 1.8 to 2.4 m apart in the longitudinal direction, and between 2.1 and 2.4 m in the transverse direction (to bridge over the pipes). Figure 9 below illustrates a diagram of column supported embankment at O'Brien Street.

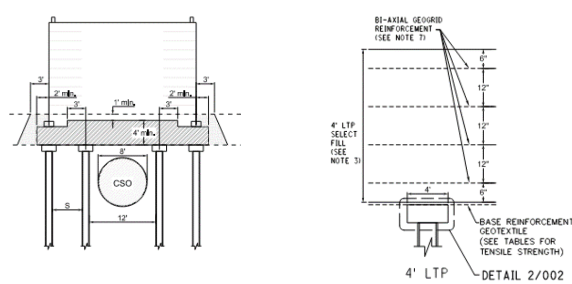

Figure 9. Typical LTP detail

4 DESIGN IN POTENTIALLY LIQUEFIABLE SOIL

Liquefaction triggering evaluations were performed for 2475-year return periods, as per the project agreement, based on the 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 isolated areas along the project alignment, particularly along the SADB segment, were found to be liquefiable.

The major effects of liquefaction on deep foundations are the loss of lateral support in the liquefied zone, ground settlement, and lateral spreading (for deep foundations near shorelines).

The liquefaction-induced downdrag forces on piles and shafts were 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 (2015) equations.

Lateral resistance along deep foundations 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 liquified soil, a design decision was made to assume, conservatively, a *total* loss of strength of the liquefiable soil.

When the factors of safety were 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 at, and in the vicinity of, Sunnybrook Station in Montreal.

The subsurface conditions in this area of the project consisted generally of fill material (silty sand) underlain by 2 to 8 m of silt layer which was underlain by glacial till. The silt layer had a density ranging from very loose to medium dense and presented significant interlayering. The silt content varied between 75% and 91%, and a clay content between 9% and 25%. The plasticity index PI and liquid limit LL were about 9 and 25%, respectively.

The silt was estimated to be potentially liquefiable in the classical sense according to Seed et al. (2003) criteria and moderately susceptible to liquefaction according to Bray and Sancio (2006). 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, high quality large diameter undisturbed samples, extracted using the Osterberg sampler tubes (an internal diameter of 127 mm), were collected and sent to the University of Laval for detailed liquefaction analysis.

All samples were reconsolidated to or near the effective field vertical pressure (σ'_{vo}). A static stress-strain behavior of each soil was next studied under constant volume direct simple shear test (DSS). The purpose of these static tests was to establish the undrained shear strength ratios (S_u/σ'_{vc}). The Over-Consolidation Ratio was in the range between 1.0 and 1.8 and the undrained shear strength ratio varied between 0.31 and 0.43 with an average of 0.35. The stress-strain behavior of the silt was either elasto-plastic or showed a slightly strain-hardening response.

The samples were then subject to cyclic direct simple shear (DSScy) test. Uniform bi-directional cyclic shear stresses were applied to the consolidated specimens under constant volume conditions. The horizontal cyclic shear force is applied by a double entry pneumatic piston at a frequency of 0.1 Hz.

The results of the cyclic constant volume DSS tests carried out showed that under cyclic loading of T_{cyc}/σ'_{vc} = 0.35 the excess pore pressure ratio $r_u = \Delta u/\sigma'_{vc}$ initially increased during the cyclic shearing, but it stabilized at an average of approximately 70% of the vertical consolidation stress σ'_{vc}. The tests ended reaching a cyclic shear strain of ±5%. It was concluded that the behavior could be categorized as cyclic softening and not cyclic liquefaction.

The results were found to be in line with the behavior of fine-grained soils (clay and silt) in eastern Canada as described by LeBoeuf et al. (2016) and shown in Figure 10 below. The figure shows the range of maximum cyclic stress level at which the soil will not suffer cyclic failure (strength degradation or deterioration), regardless of the number of cycles, i.e. any cycling below this stress ratio will not induce failure and will not impact the initial shear strength.

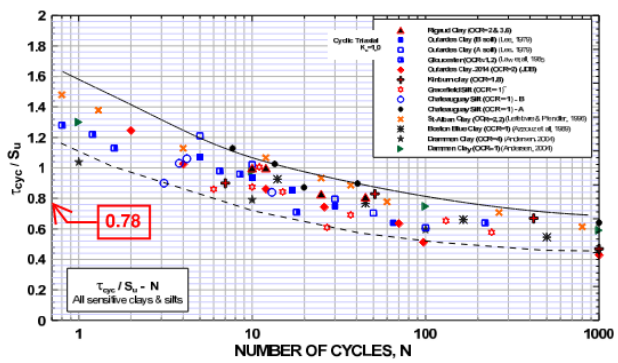

Figure 10. Cyclic strength data for fine-grained soil in eastern Canada (after LeBoeuf et al. - 2016)

Figure 10 shows that at a cyclic stress level (T_{cyc}/S_u) of about 0.78 for the soil at the site, and the number of cycles typical of seismic vibrations, classical cyclic liquefaction cannot be triggered even though cyclic softening and strength loss, induced by cyclic pore pressure combined to fatigue loading, would occur.

Lateral resistance of drilled shafts and piles constructed in the silty soil was adjusted (lowered) such that the *p-y* curve parameters were modified as per Miyamoto (1987):

$$K_r = K_{max}\sqrt{(1 - R_u)} \quad [3]$$
$$P_{ur} = P_{u-max}(1 - R_u) \quad [4]$$

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

5 CONCLUSIONS

Design of the REM project in Montreal posed significant geotechnical problems that were largely unexpected prior to the geotechnical investigation. The following concluding remarks could be made:

- The use of column supported embankment deemed very useful for the protection of utilities.
- Preloading at the peatland with settlement monitoring was necessary to capture the real

consolidation behavior of the peat as compared to the theoretical analyses.

- Use of semi rigid CMC was a cost-effective alternative for the construction of embankment over thick, soft clay deposit.
- Laboratory cyclic shear tests conducted on potentially liquefiable soils helped determine the percentage loss of soil resistance and optimize the foundation seismic design.

6 ACKNOWLEDGMENT

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