The elastic response of a Champlain Sea clay deposit under test embankment loading conditions in Beauharnois, Quebec

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ABSTRACT
Two test embankments were constructed and instrumented over a Champlain Sea clay deposit in Beauharnois, Quebec, in 2017. The general soil stratigraphy of this area consists of a layer of organic soil followed by a 25 m deep, highly compressible Champlain Sea clay deposit. Below a depth of approximately 4 m, the clay is classified to be extra sensitive to quick and over-to-normally consolidated. The test embankments were 2 m and 3 m in height and both had plan areas of 50 m by 50 m. The instrumentation included vibrating-wire settlement gauges and multi-stage vibrating-wire piezometers, both equipped with data-loggers to collect pore-water pressures and ground settlements. The embankments were constructed and maintained for about four months. Primary consolidation was not initiated during the monitoring period; however, ground surface rebound measurements were obtained as the embankments were removed. In addition, an interpretation of the rebound measurements was completed to estimate the aggregate elastic modulus of the soil deposit.

RÉSUMÉ
Deux remblais de surcharges ont été construits et instrumentés sur un dépôt d'argile de la mer Champlain à Beauharnois (Québec) en 2017. Le site est situé entre le canal de Beauharnois et la rivière Saint-Louis, à environ 500 m du lac Saint-Louis. La stratigraphie générale du sol de ce secteur consiste en une couche de sol organique suivie d'un dépôt d'argile de la mer Champlain de 25 m d'épaisseur et hautement compressible. En dessous d'une profondeur d'environ 4 m, l'argile est classée comme étant très sensible à liquide et normalement à surconsolidée. Les remblais de surcharge mesuraient 2 m et 3 m de hauteur et avaient tous les deux une superficie de 50 m par 50 m. L'instrumentation comprenait des sondes de tassement à corde vibrante et des piézomètres à corde vibrante sur plusieurs niveaux, tous deux équipés d'enregistreurs de données pour collecter les pressions interstitielles et les tassements du sol. Les remblais ont été construits et maintenus en place pendant quatre mois. La consolidation primaire n'a pas été initiée pendant la période de surveillance; cependant, des mesures de rebond de la surface du sol ont été obtenues lorsque les remblais ont été retirés. Cet article présente une interprétation des mesures de rebond pour estimer le module d'élasticité global du dépôt de sol.

1 INTRODUCTION

A 800 m-long warehouse development is to be built in an area located between the Beauharnois Canal and Rivière Saint-Louis, approximately 500 m from Lac Saint-Louis, just west of Montreal, Québec. The development was to include up to 2.1 m grade raise for a loading dock. This area was subject to previous investigations in the 1980s by Dion et al. (1986). It was hypothesized that the Champlain Sea lake bottom, at end of the final deposition period, was approximately 7 to 9 metres above the current ground elevation as result of erosional events. The general soil stratigraphy for this area consists of a layer of organic soils followed by a thick Champlain Sea clay deposit.

The clay is classified to be extra sensitive to quick below a depth of approximately 4 m, over-to-normally consolidated in accordance with the Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 2006). Using the classical consolidation design methodologies results in a conservative yield stress profile in this type of environment. The consequence of this is that costly foundation support methods for large slab on grade foundations are required to avoid the anticipated large-scale settlements. It was, therefore, crucial to overcome the uncertainties that are involved in the estimation of the yield stress profile to produce a much more cost-efficient design.

Full-scale test embankments were constructed to predict stress levels beyond the predicted yield stress and the load at which significant settlements occur. An air photo of the site including the test embankments is shown on Figure 1. The test embankment details were documented in El-Dana et al., 2017. The objectives of the test embankments included researching the consolidation behavior of Champlain clay deposits as well as the rebound measurements after removing the earth loads.

Each test embankment included two Vibrating Wire Piezometers (VWPs) 3-stage installations and two settlement gage installations. The test embankments facilitated the monitoring of the porewater pressure responses (i.e. changes in piezometric elevations due to new fill loading/unloading) to help define the in-situ soil yield stress and embankment settlements.

Figure 1. Air photo of the site showing the test embankments (retrieved from Google Earth, dated May 19, 2017)

2 GENERAL SOIL STRATIGRAPHY

The general soil stratigraphy within the site consists of a layer of remolded organic silty clay layer resulting from past agricultural activities, underlain by a thick native silty clay deposit (Champlain Sea clay), underlain by granular till over bedrock. The bedrock encountered consists of Dolomitic Sandstone to Sandstone. The depth to bedrock varies between 22 and 30 m.

Figure 2 presents the soil moisture content and estimated void ratio versus geodetic elevation. The upper 4 m of the clay has a moisture content in the range of 30 to 40 % that is less than the liquid limit of the soil. This zone represents the drier and stiffer zone of the clay. This upper layer of the clay deposit was found to be quite competent and is, therefore, it poses little challenges to construction related activities.

Figure 3 presents the sensitivity and liquidity index versus elevation. The in-situ and laboratory shear vane test results indicate that the clay deposit, below an elevation of 38 m becomes wetter and is classified to be extra sensitive to quick clay in accordance with the 2006 Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 2006). The moisture content is higher than the liquid limit yielding a liquidity index of more than one.

Figure 4 presents the preconsolidation stress profile predicted using the shear strength measurements collected through field vane and cone penetration tests. The figure also presents previous preconsolidation stress measurements reported by Dion et al. (1986). Using this data, various design lines for the preconsolidation pressure may be predicted as presented on Figure 4. These lines would typically produce significantly different design recommendations. Figure 4 also shows the elevations of the vibrating wite piezometers (VWPs) installed as part of the monitoring program to predict the highest preconsolidation pressure profile of the clay deposit.

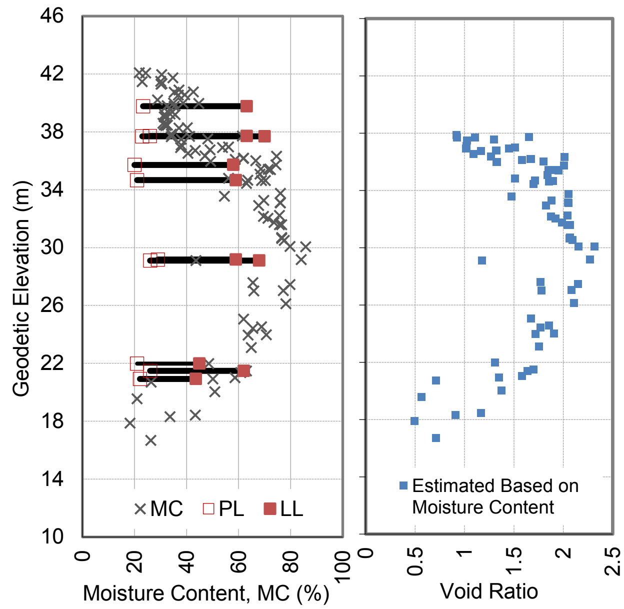


Figure 2. Moistue content and void ratio versus elevation

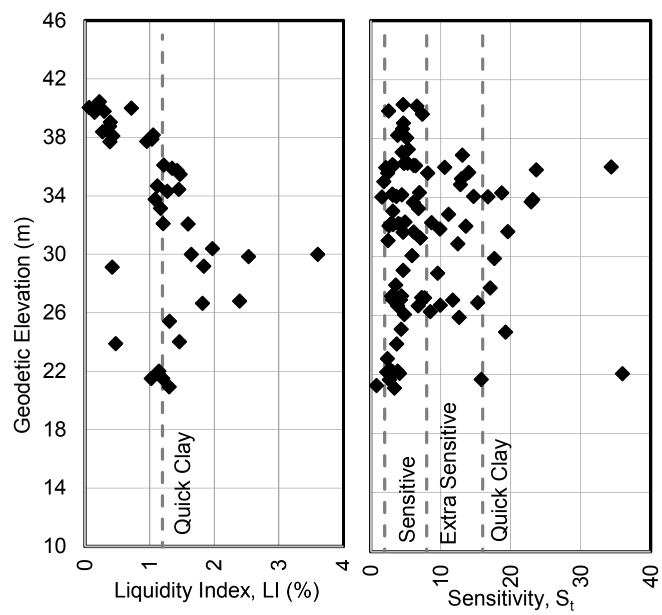
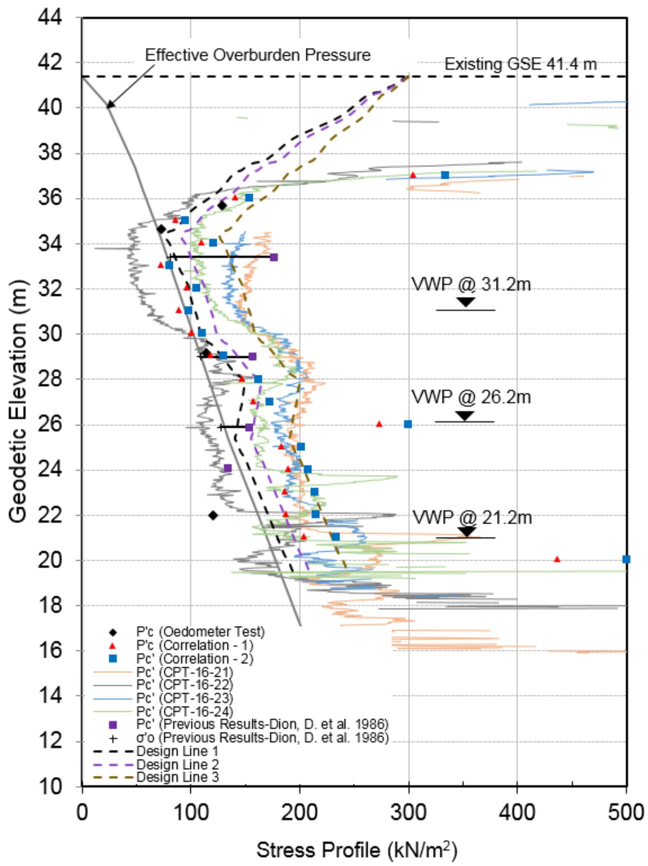


Figure 3. Sensitivity and liquidity index versus elevation



Correlation-1: $Pc' = S_u/(0.2+0.0024*PI)$
Correlation-2: $Pc' = (222/LL)*S_u$
PI: Plasticity Index, LL: Liquid Limit, Su: Shear Strength

Figure 4. Overburden pressure and preconsolidation pressure profiles

3 FIELD MONITORING

The field program included constructing two test embankments that were 2 m and 3 m in height and both had plan areas of 50 m by 50 m as documented in El-Dana et al., 2017. For the 2 m high test embankment (Test Embankment 2M), it was estimated that the total fill weight corresponded to approximately 46 kPa. For the 3 m high test embankment (Test Embankment 3M), it was estimated that the total fill weight corresponded to approximately 68 kPa. Figures 5 and 6 present the piezometric elevations versus time for both test embankments. An increase in porewater pressures was recorded as a result of the weight of the embankments. The recorded magnitude of the increase in porewater pressure varied with the change in depth below the ground surface.

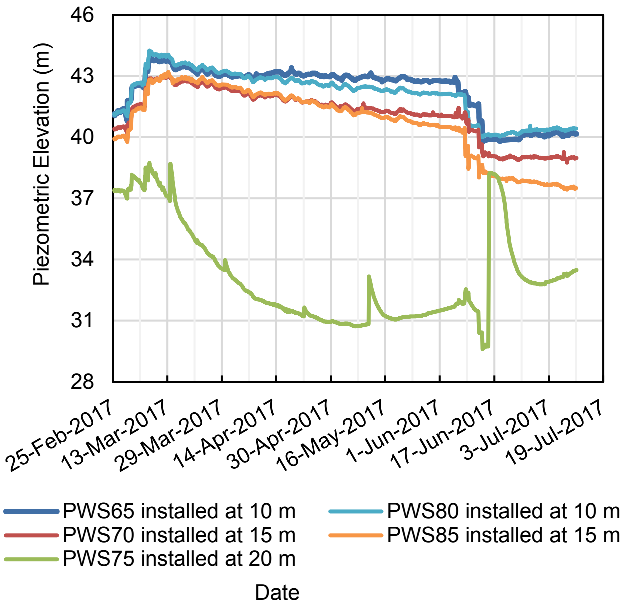


Figure 5. Piezometric elevation versus time for Test Embankment 2M

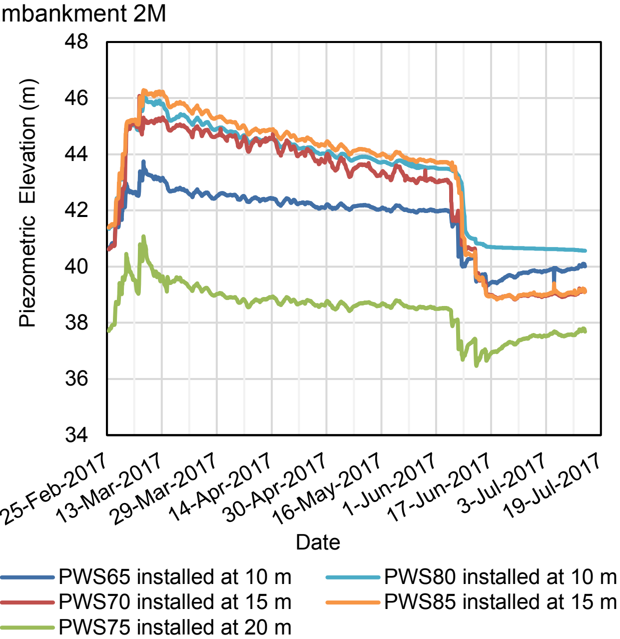


Figure 6. Piezometric elevation versus time for Test Embankment 3M

4 THE POREWATER PRESSURE CHANGE

The increase in the porewater water pressure was calculated as a percentage of the fill weight (i.e., B-bar). The B-value reflects the pore water pressure change (Δu) with regard to the change in vertical stress ($\Delta\sigma_v$) as shown in Equation 1.

$$\Delta u = \text{B-bar} * \Delta\sigma_v \quad [1]$$

The change in vertical stress is mainly due to the weight of added layers of materials. Figures 7 and 8 present the B-bar value based on the measured piezometer elevations for the two test embankments. B-bar values of up to 67% to 80% were measured within the quick clay portion. Some decrease in porewater pressures was recorded within three to four months after the installation of the test embankments.

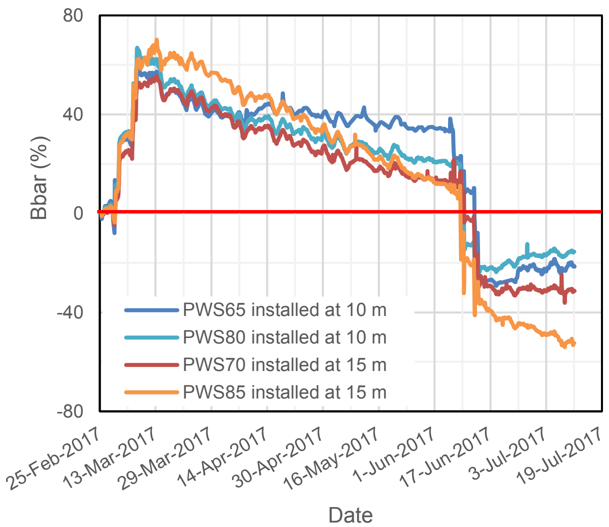


Figure 7. B-bar value versus time for Test Embankment 2M

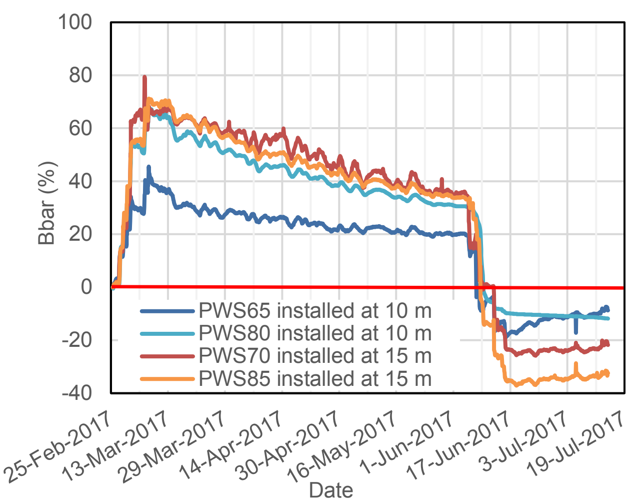


Figure 8. B-bar value versus time for Test Embankment 3M

Figure 9 presents the B-bar values obtained at 10 m, 15 m and 20 m below the ground surface. It is noted that the rate of porewater pressure dissipation within the clay measured at Test Embankment 2M was higher than the one at Test Embankment 3M. The collected measurements from the two test embankments were used to estimate the maximum preconsolidation stress profile for the clay deposit. Figure 10 shows the predicted maximum yield stresses at piezometer levels (Elevations 31.2 m and 26.2 m) installed beneath the test embankments within the clay deposit layer compared to the yield levels that were estimated based on the previous laboratory tests. The piezometer installed at Elevation 21.2 m showed considerable decrease in pressure which is attributed to its proximity to the bottom of the clay deposit. Therefore, it could not be used to confirm the yield stress level of the clay deposit. It is noted that if the test embankments have had been left in place for more time, the predicted yield stresses would typically increase with time as more porewater dissipation occurs, assuming that no settlements would occur. However, if sudden significant settlements occurred this would be indicative that the yield stress level had been reached within some portions of the compressible deposit.

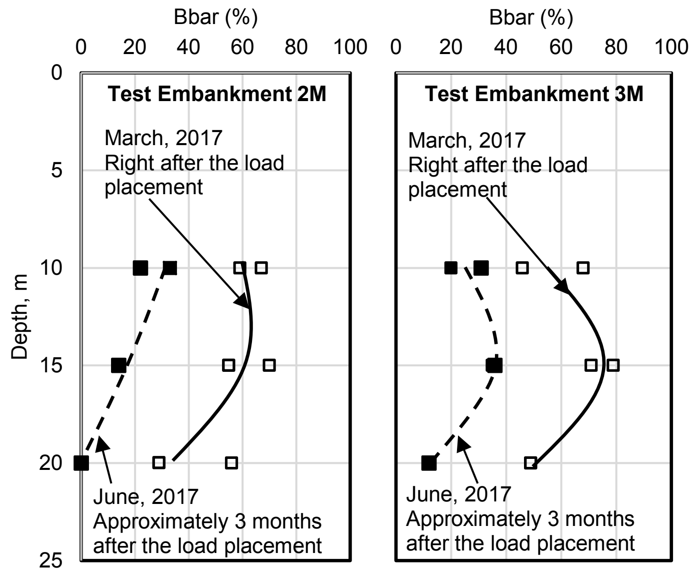


Figure 9. Estimated B-bar values versus depth

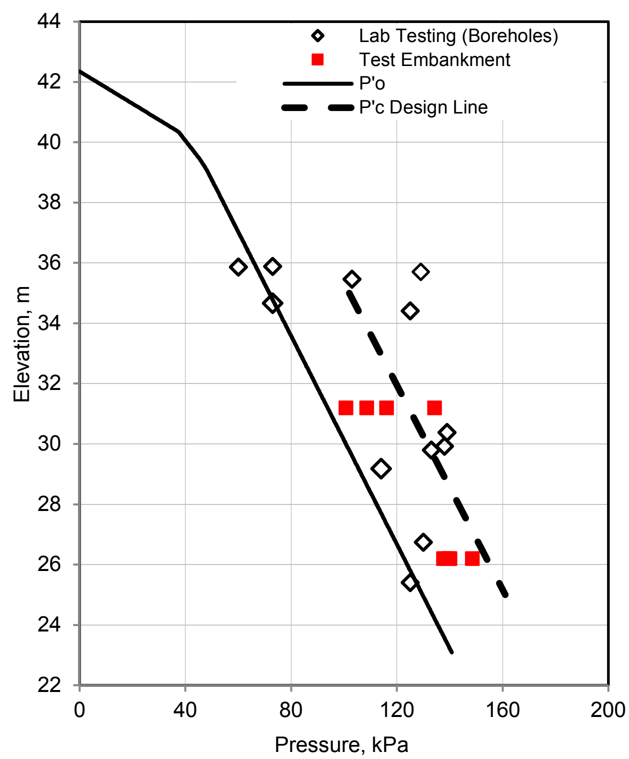


Figure 10. Effective Overburden Pressure, P'o, versus Preconsolidation Pressure, P'c, test values

Figure 11 presents excess porewater pressure measurements with the increase in the total vertical stress resulting from the test embankments loads. The excess porewater pressure values measured equaled the applied load magnitude or was of a close magnitude (meaning a B-bar value of 100%) with the increase of the load to close to 20 kPa. Less load was, however, translated to excess porewater pressures after 20 kPa. This trend was found to be useful to predict the excess porewater pressure expected for Champlain Sea clays under different loading conditions.

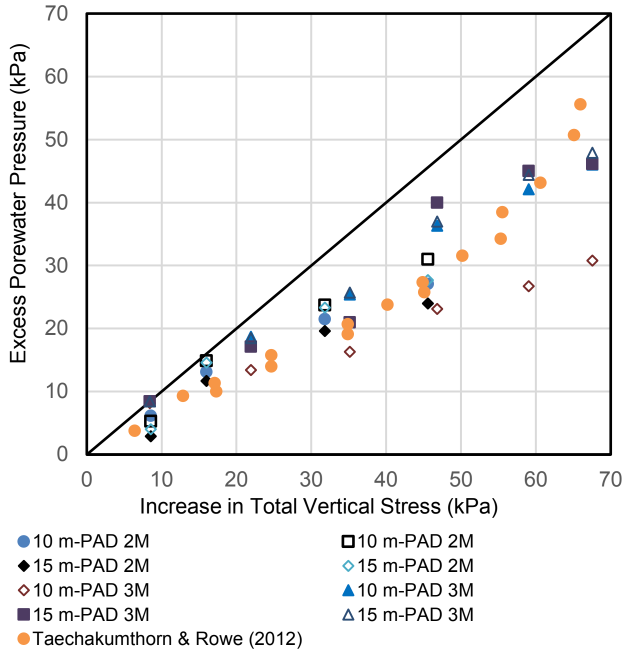


Figure 11. Excess porewater pressure versus the increase in total vertical stress

5 SOIL ELASTIC RESPONSE

According to the settlement measurements for the two test embankments as reported in El-Dana et al., 2017, no significant settlements were measured after the application of the fill loads up to four months. It was concluded that the primary consolidation was not initiated during the planned monitoring period. Ground surface rebound measurements were, therefore, obtained as the embankment earth fills were removed. Figure 12 presents the recorded ground movement at the center of Test Embankment 2M as the load being removed. As can be seen in Figure 12, the maximum settlements recorded was close to 20 mm before removing the earth fills.

As part of the evaluation of the settlement data, a geotechnical model was established for the site. The rebound settlements were calculated using the Settle3D software by RocScience. The Boussinesq stress distribution was selected for the model. According to the modeling results, an aggregate elastic modulus of 150 MPa was back calculated for the Champlain Sea clay at this site under unloading condition.

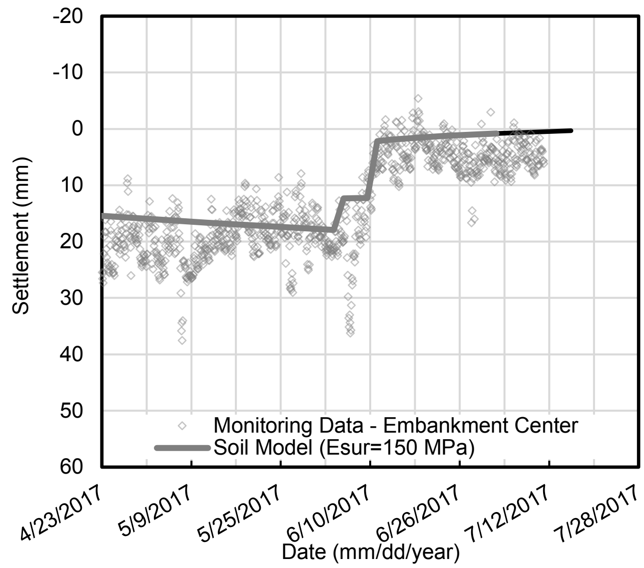


Figure 12. Predicted and measured rebound settlements

6 CONCLUSIONS

This research work was developed to reduce the uncertainty in the prediction of the preconsolidation profile of the Champlain clay deposit in Beauharnois, Quebec and to rationalize the anticipated foundation settlements. Two test embankments were constructed and instrumented. It was concluded that the primary clay consolidation was not initiated within approximately four months from the application of loads.

A porewater pressure coefficient (B-bar value) of close 100%, under the application of the fill lifts (up to a load of approximately 20 kPa), was measured within the clay deposit encountered at this site. B-bar values of close to 70% and 80% were measured within the quick clay portion under 46 kPa and 68 kPa, respectively. An aggregate elastic modulus of 150 MPa was back calculated for the Champlain Sea clay.

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