Geotechnical, Geophysical and Hydrogeological Characterization of Soft Sensitive Leda Clay in Ottawa and Associated Challenges in Foundation Seismic Design

Farbod Saadat, Ph.D., P.Eng., and Shane Dunstan, P. Eng.
DST Consulting Engineers Inc. A Division of Englobe Corporation, Ottawa, Ontario, Canada

ABSTRACT
A geotechnical, geophysical and hydrogeological investigation was conducted in Champlain Sea basin to 30.5 m depth for the design of a new control building with basement and a deep metering chamber extending to about 12.4 m depths at the existing Delorme wastewater pumping station in Ottawa. The Site is about 4 km south of Ottawa River within the major physiographic region of the Ottawa - St. Lawrence Lowland and the minor physiographic region known as the Ottawa Valley Clay Plain. The investigation found thick deposits of sensitive Leda Clay and mostly non-swelling marine clay, silty clay and silt deposits extending beyond 30 m depth. Considering the high sensitivity and weakness of this Leda Clay with moisture contents wet of Liquid Limit and observed groundwater below the upper crust and the resultant seismic site classification and anticipated loading, this case study discusses both the design and potential constructability issues and challenges.

RÉSUMÉ
Une étude géotechnique, géophysique et hydrogéologique a été menée dans le bassin de la mer Champlain à une profondeur de 30.5 m pour la conception d'un nouveau bâtiment de contrôle avec sous-sol et une chambre de dosage profonde s'étendant jusqu'à environ 12.4 m de profondeur à la station de pompage d'eaux usées Delorme existante à Ottawa. Le site est situé à environ 4 km au sud de la rivière des Outaouais, dans la principale région physiographique des basses terres d'Ottawa - St. Laurent et dans la région physiographique mineure connue sous le nom de plaine d'argile de la vallée d'Ottawa. L'enquête a révélé des dépôts épais d'argile Leda sensible et principalement des dépôts d'argile marine, d'argile limoneuse et de limon non gonflants, s'étendant au-delà de 30 m de profondeur. Compte tenu de la sensibilité et de la faiblesse élevée de cette argile Leda avec des teneurs en eau humides de Limite Liquide et des eaux souterraines observées sous la croûte supérieure et de la classification du site sismique résultante et de la charge prévue, cette étude examine des solutions pratiques pour surmonter les défis de conception et de construction.

1 INTRODUCTION

A comprehensive geotechnical, geophysical, and hydrogeological investigation was conducted for the proposed rehabilitation of the Delorme Wastewater Pumping Station at 1847 Des Épinettes Avenue in Ottawa (Figure 1). The project Site lies within the minor physiographic region known as the Ottawa Valley Clay Plain, as part of the major physiographic region of the Ottawa-St. Lawrence / Champlain Sea Lowland (Chapman and Putnam, 1984).

The Ottawa Valley Clay Plain region is generally characterized by relatively thick deposits of sensitive and mostly non-swelling marine clay, silty clay, and silt deposited within the Champlain Sea basin (Hache R., 2018, Roy E. Hunt, 2007; Leroueil, 1997; Penner 1965 & 1978). This thick clay deposit (Leda Clay) overlie a relatively thin commonly reworked glacial till and glaciofluvial deposits and bedrock (Hache R., 2018).

The proposed pumping station upgrades (Figures 2 and 3) include design and construction of a new combined control building/swab launch chamber, with one level basement below the upper clay crust, on a rigid raft foundation to be founded on the underlying weaker unweathered clays at 4.8 m depth (El. 81.7 m) and supporting a maximum Serviceability Limit States (SLS) design bearing pressure of 77 kPa.

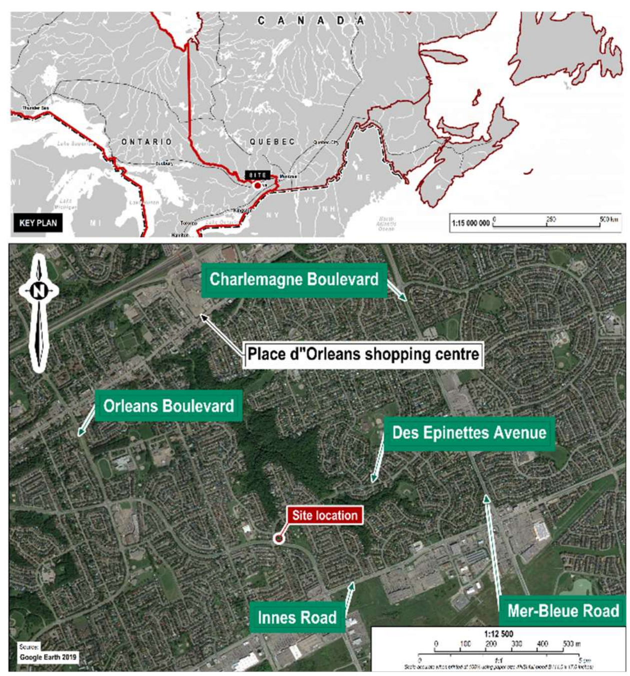

Figure 1. (a) Site key map, (b) Site location

Figure 2. Plan View of Existing and Proposed Facilities and Borehole Locations

The proposed construction also includes a new deep wet well (Bypass Chamber) down to 12.4 m (El. 74.5 m) to be advanced by proposed sunken-shaft method with incremental addition of precast concrete rings (Figure 3).

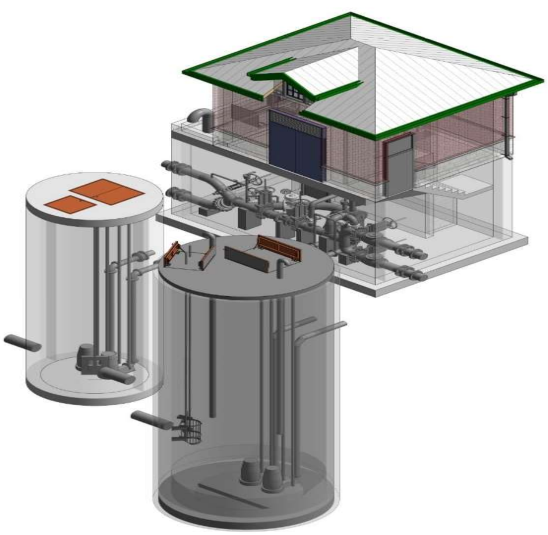

Figure 3. Proposed Facility Upgrades

2 METHODOLOGY AND SYNOPSIS OF RESULTS

Four (4) carefully sampled boreholes (3 monitoring wells) were advanced in two stages (2017-2018) down to 30.5 m depth. In-situ field vane tests (FVTs) were conducted at regular intervals on undisturbed and remolded clay. The recovered undisturbed Shelby tube and spilt spoon soil samples were characterized by complete index tests and 1D consolidation tests. Considering the soft and sensitive nature of deep Leda clay deposit (Hache R., 2018, City of Ottawa, 2004), an additional downhole shear wave seismic survey was also conducted to measure the shear wave velocity profile down to 30 m depth to confirm the Seismic Site Classification, as per Table 4.1.8.4 of the Ontario Building Code (OBC-2012).

The Site generalized stratigraphy consists of a brown desiccated very stiff clay crust underlain by a firm to soft grey unweathered silty clay. The hydrometer test results confirmed that this deposit has a very high fines content, between 90 to 100 percent. Based on the FVTs, sensitivities were found to range from 2 to 10, meaning that the shear strength after disturbance is 2 to 10 times less than the intact shear strength. For example, during the drilling program the rods, augers, and casings at times were heavier than could be supported by the shear strength of the clay soils at depth. As a result, the drilling tools were sinking and had to be restrained at the surface.

Summaries of in-situ and laboratory test results are presented in Figures 4 to 11.

Table 1. Summary of Subsurface Conditions

BH	Topsoil/ Asphalt/Fill Depth (m) (El. m)	Native Brown Desiccated Clay Crust (m)	Native Gray Unweathered Clay [1] (m)
MW17-01	0.8 (El. 85.7)	0.8-6.8 (El. 85.7-79.7)	6.8-15.0 (El. 79.7-EOB)
BH17-02	0.2 (El. 86.1)	0.2-4.3 (El. 86.1-82.0)	4.3-15.0 (El. 82.0-EOB)
MW18-03	1.5 (El. 84.9)	1.5-4.3 (El. 84.9-82.1)	4.3-6.1 (El. 82.1-EOB)
MW18-04	3.8 (El. 82.7)	-	3.8-30.5 (El. 82.7-EOB)

[1]EOB = End of borehole (no refusal)

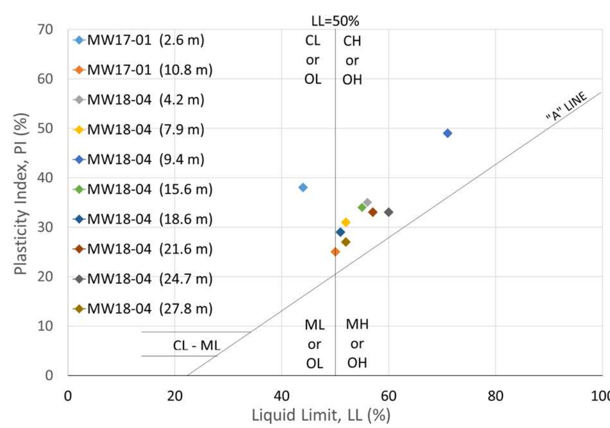

Figure 4. Atterberg Limits for High Plasticity Clay (CH)

The groundwater levels in monitoring wells MW17-01, MW18-03, and MW18-04 were recorded and are summarized in Tables 1 and 2. The results generally indicated rising groundwater levels over time and perched conditions towards ground surface in the upper disturbed (reworked) soils following prolonged precipitation and snowmelt events in early spring. Presence of discrete silt and sand lens/seams were also suspected within this clayey deposit with higher permeability and potential seepage during precipitation events and/or wet seasons.

3 COMPARISON OF SITE CHARACTERISTICS WITH CHAMPLAIN SEA CLAYS LITERATURE

Three 1D laboratory consolidation tests were conducted on select recovered undisturbed soil samples. The results for upper, middle and lower clay deposits are summarized on Figure 5 and compared with published literature in Table 3. Reliability of key compressibility parameters, such as estimated pre-consolidation pressure (σ'_p) and compressibility indices (C_c) is well-known to be of paramount importance in estimation of anticipated settlement and foundation design. Considering the variability and ranges of "pre-consolidation pressure interpretations using the oedometer test" from the classical Casagrande (1936) method to Storer J. Boone (2010), and reliable (slightly conservative) values of pre-consolidation pressure were selected and used for foundation design, as per Table 3.

Table 2. Groundwater Monitoring Results

BH	Well Screening Intervals (m)	Monitoring Date (MM/DD/YY)	Water Level (m)
MW17-01 (El. 86.5)	7.5 -11.0 (El. 79.0-75.5) Unweathered grey silty clay (Intermediate / Middle Clay)	11/15/17	8.4 (El. 78.1)
		12/20/17	6.2 (El. 80.3)
		11/2/18	5.4 (El. 81.1)
MW18-03 (El. 86.4)	2.6 -6.1 (El. 83.8-80.3) Weathered brown clay crust (Shallow / Upper Clay)	10/23/18	2.2 (El. 84.2)
		11/02/18	2.0 (El. 84.3)
		11/13/18	1.8 (El. 84.6)
MW18-04 (El. 86.5)	26.8 -30.5 (EL. 59.7-56.0) Unweathered grey silty clay (Deep / Lower Clay)	10/23/18	6.0 (El. 80.5)
		11/02/18	6.5 (El. 80.2)
		11/13/18	6.4 (El. 80.3)

Considering the variability of "pre-consolidation pressure interpretations using the oedometer test" (Storer J. Boone, 2010), utilization of reliable values for key compressibility parameters, such as estimated pre-consolidation pressure (σ'_p) and compressibility indices (C_c) proved to be of critical importance in both design of proposed mat foundation and serviceability of proposed piping, as demonstrated in the following paragraphs.

4 SOIL PARAMETERS FOR DESIGN UNDER STATIC & SEISMIC LOADING CONDITIONS

Based on the results of the field investigations and laboratory testing, and comparisons with literature (i.e., Terzaghi, Peck and Mesri publications 3rd Edition; Kenney 1959; CFEM 2006), representative soil parameters are recommended in Table 3 for design of foundations and retaining walls under both static and seismic conditions (using the pseudo-static analysis).

For design of yielding retaining walls, the active earth pressure coefficient, K_a, is recommended, whereas for non-yielding permanent walls, such as basement walls, the at-rest earth pressure coefficient, K_o, is recommended. The resultant of the applicable static or at-rest force is assumed to act at 1/3H above the base of the wall where H is the Height of the wall. The resultant forces from the anticipated hydrostatic groundwater pressures (rising up to ground surface) must be taken into account.

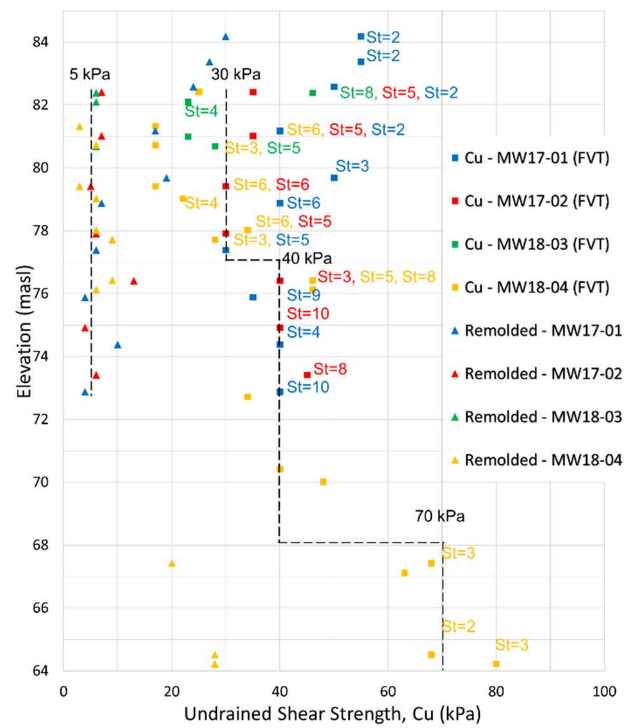

Figure 5. Sensitivity and In-Situ Undrained Shear Strength C_u Profiles of Undisturbed & Remolded Clay

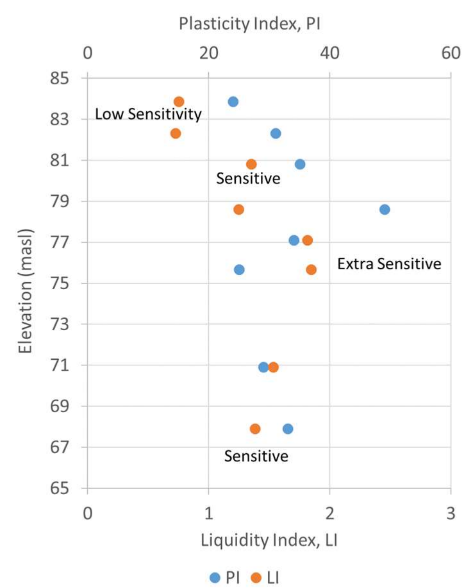

Figure 6. Profiles of Liquidity Index & Plasticity Index

Table 3. Comparison of Consolidation Results with Champlain Sea Clays Literature

Laboratory Results			Published Literature		
σ'$_p$ (kPa)	C_c	C_r	σ'$_p$ (kPa)	C_c	C_r
MW18-03 ST1 (El. 83.0 m) Clay Crust (El. 86.5 to 82.0 m)					
240	0.65	0.04	240(2)	0.31(2) 0.55(3) 0.18(4)	0.061- 0.065(4) 0.034(5)
MW18-04 ST1 (El. 80.0 m) Upper Gray Clay (El. 82.0 to 79.0 m)					
80	1.38	0.04	80(1)	0.41(2) 1.11(3) 0.22(4)	0.082- 0.138(4) 0.044(5)
MW18-04 ST1 (El. 80.0 m) Middle Clay (El. 79.0 to 66.0 m)					
120	3.08	0.04	80-171(1)	0.45(2) 1.20(3) 0.24(4)	0.090- 0.308(4) 0.047(5)
Lower Unweathered Grey Clay (El. 66.0 to 49.0 masl)					
NA	NA	NA	220(1)	0.41(2) 1.20(3) 0.22(4)	0.081- 0.308(4) 0.043(5)

(1) Leroueil (1983)
(2) Skempton (1944)
(3) Rendon Herrero (1983)
(4) Nagaraj and Murty (1985)
(5) Principles of Geotechnical Engineering (Das-2006)

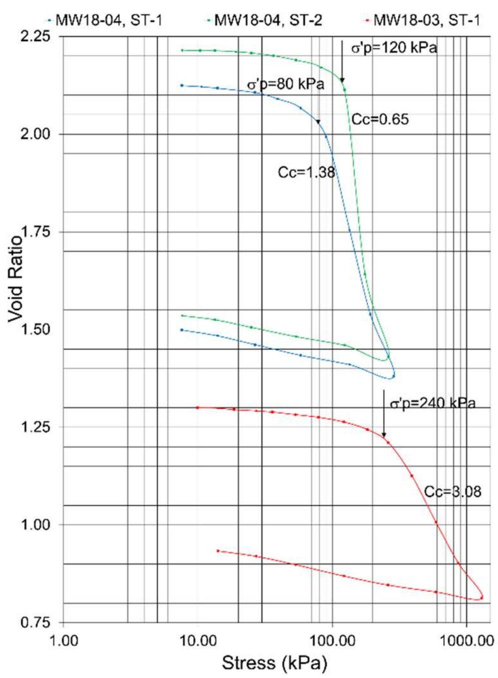

Figure 7. Results of 1D Consolidation Tests

For design of subsurface walls subjected to seismic lateral forces Pseudo-Static Mononobe-Okabe design approach was recommended, which includes both takes into account both geotechnical and geometric components (as per Section 24.9 of CFEM-2006). The total active thrust under seismic loading (P_{ae}) is defined as follows:

$$P_{ae} = ½\ K_{ae}γ\ H^2\ x\ (1- k_v) \qquad [1]$$

Where: H = Height of the wall, K_{ae} = horizontal component of active earth pressure coefficient including effects of earthquake loading, k_v = Vertical component of the earthquake acceleration (typically a range of $(2/3)k_h$ to $(1/3)k_h$ is considered, but a value closer to $(2/3)k_h$ is recommended), k_h = Horizontal component of the earthquake acceleration (typically Peak Ground Acceleration (PGA) or a factor thereof is used).The Site Class-adjusted PGA for the Site is 0.448g at Site Class E, where g is the acceleration due to gravity.

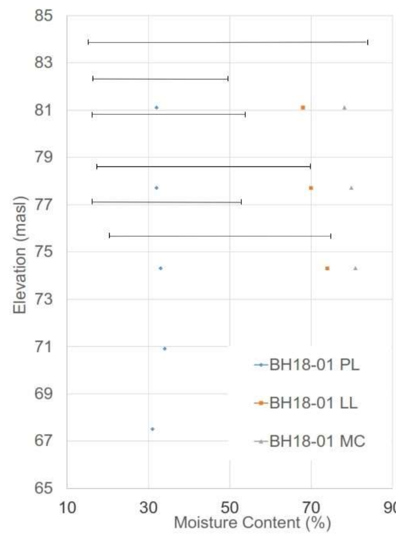

Figure 8. Natural moisture content & plasticity profiles

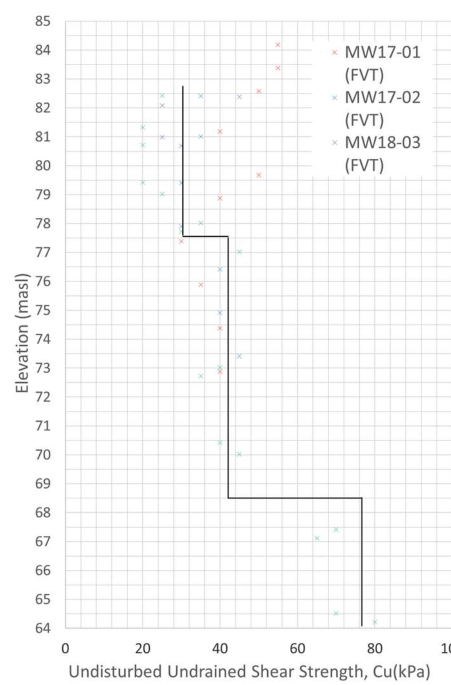

Figure 9. Undisturbed Undrained Shear Strength C_u

Table 4. Representative Soil Parameter under Static and Seismic Conditions

Subsoil depths (m)	ɣ (kN/m³)	φ' (°)	C' (kPa)	C_u (Avg.) (kPa)	Mononobe-Okabe Earth Pressure Coefficients[1] K_{ae}	K_{pe}
Brown Upper Crust (0.3–4)	18-20	26	5-10 (5)	55-145 (100)	1.88	1.88
Gray Middle Firm Clay (4-7)	17	20	2	20-50 (30)	1.75	1.75
Lower Firm Clay (7-30)	15-16.5	19		30-45 (35)		
Gran. B Type I OPSS 1010	22	32			1.97	2.19

[1] ɣ = Bulk Density (kN/m³)
φ' = Effective Angle of Internal Friction (degrees)
C'= Effective Cohesion (degrees)
C_u = Undrained Shear Strength (kPa)
MSR = Modulus of Subgrade Reaction

For passive earthquake pressure (P_{pe}) the following equation can be used:

$$P_{pe} = \frac{1}{2} K_{pe} \gamma H^2 \times (1 - k_v) \quad [2]$$

Where: K_{pe} = horizontal component of passive earth pressure coefficient including effects of earthquake loading.

Equation 2 includes both the active pressures under static (P_a) as well as the increased force due to seismic forces. The active force under static conditions is assumed to act at a point of 1/3H above the base, and the seismic force is assumed to act near 2/3H above the base, where H is the height of the wall. Therefore, the point of application for P_{ae} may be calculated from the following:

$$h = [(0.33H \times P_a) + (0.6H \times P_e)] / P_{ae} \quad [3]$$

5 DOWNHOLE SEISMIC SURVEY

Considering the limited size of the Site, DST drilled an additional deep borehole (MW18-04) and retained Geophysics GPR International Inc. to perform a down-hole seismic refraction survey within the same geotechnical borehole. Summary of the results are presented on Figure 10 indicating Seismic Site Class '*E*' (better than initial '*F*' Site Class).

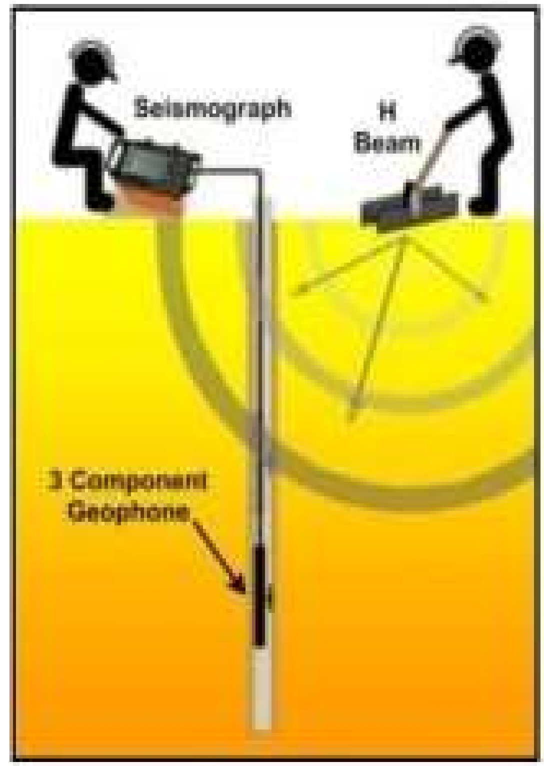

Figure 10. Downhole Seismic Survey in MW18-04

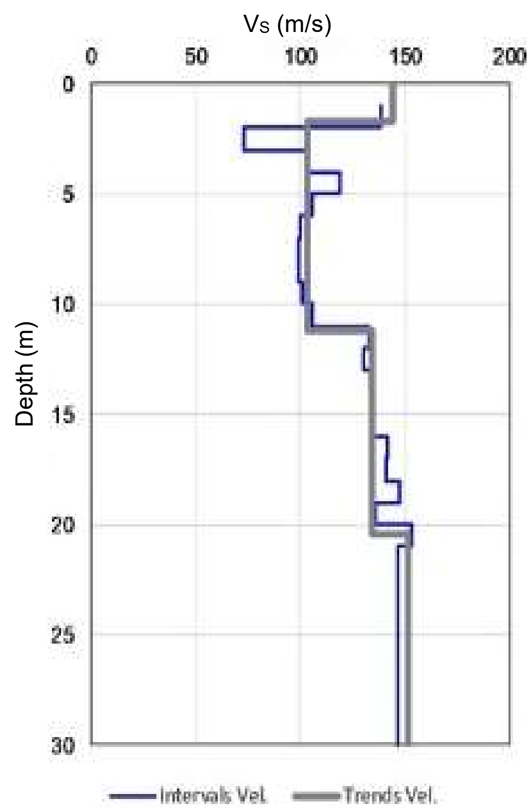

Figure 11. Variation of shear wave velocity (Vs) with depth

6 OVERVIEW OF MAJOR DESIGN AND CONSTRUCTABILITY ISSUES

Considering highly frost susceptible nature of clayey overburden at the Site, the frost penetration depth was estimated at approximately 1.8 m, based on the Ministry of the Environment's published data (established on an 85% probability, the design freezing index for the Ottawa area estimated to be 1,050 degree-days Celsius, or 1,922 degree-days Fahrenheit).

Given the results of the investigation, the ideal foundation solution for the proposed building is normally to raise the foundation elevation, so it is founded within the upper more competent desiccated hard to very stiff crust. However, the operational needs of the proposed building required a full basement and much lower foundation level and with this new building design obtaining reliable compressibility parameters for the underlying weaker clay deposit became of paramount importance.

6.1 Reliance on Key Consolidation Parameters for Estimation of settlement of Mat Foundation

The generalized stratigraphy and carefully selected compressibility parameters were used to estimate the potential settlements for the Combined Control Building/Swab Launch Chamber. The load bearing clays (below the upper crust) on this Site are normally consolidated, they are very sensitive to even minimal grade raises and potential changes to clay cations chemistry that could potentially compromise the structure of undisturbed marine clay deposit. Grade raises must be limited to a maximum of 0.2 m. Any grade raises on this Site would result in excessive consolidation settlement, which would affect the existing structure, proposed structures, and adjacent properties. If grade raises are required, then light weight aggregates or manufactured lightweight geofoam fills will need to be considered and will require further settlement estimates performed.

For a raft foundation founded on the native undisturbed unweathered grey clay, a factored Ultimate Limit States (ULS) bearing resistance of 80 kPa is recommended under undisturbed conditions. This includes for a geotechnical resistance factor of $\Phi = 0.5$. Table 5 presents estimated settlements at the proposed founding elevation, given the size of mat foundation and anticipated SLS design contact pressure. It should be noted, however, that the typical Kv1 value provided above is for a standard 0.3 m x 0.3 m area and does not include possible surrounding grade raises. The inclusive MSR provided above is based on the settlement estimate and is inclusive of the building geometry and possible additional grade raises.

6.2 Limitation on Potential Grade Raises

The grey unweathered clays are weak, saturated, sensitive, and normally consolidated. Any grade raises on this Site would result in excessive consolidation settlement, which would affect the existing structure, proposed structures, and adjacent properties. Therefore, grade raises should not be allowed, unless light weight aggregates or manufactured lightweight geofoam fills are used and are properly engineered.

Table 5. Estimated Structure-Specific Settlements

Foundation Design Parameters & Estimated Settlements	
Mat Foundation	5.9 m x 8.5 m(1)
Foundation elevation	El. 81.7 masl(1)
Maximum SLS design contact pressure	77.0 kPa(1)
Estimated unloading due to basement excavation	-4.8 m x 17 kN/m^3 (-81.6 kPa)
Maximum allowance for global grade raise	0.2 m x 20 kN/m^3 (4.0 kPa)
Estimated Long-Term Settlements	
No grade raise	25 to 40 mm(2)
Additional settlement due to grade raise only	25 to 35 mm(2)
Estimated total long-term settlement	35 to 75 mm(2)
Estimated MSR for proposed mat foundation including grade raise	MSR = 1.5 MPa/m
Estimated MSR for a small square 0.3 m x 0.3 m	K_{v1} < 5 MPa/m

(1)The proposed mat foundation is designed as relatively flexible foundation and the maximum foundation soil contact pressure will be limited to maximum SLS design value of 77 kPa.
(2)Subject to confirmation and approval of Contractor's methodology prior to construction.

Using the proposed design loading, and potential variation of consolidation parameters for the load bearing sensitive clay deposit (as presented above), total settlement of mat foundation was estimated between 35 and 75 mm. Considering that settlement tolerances for similar structures supported on raft foundations is typically up to 50 mm total settlement and up to 40 mm differential settlement. Considering the variability and sensitive nature of the underlying clay deposit and the potential for the estimated total settlement to exceed the above-noted typical values were recognized and the proposed building and corresponding utility connections were designed in such a way to accommodate the above-noted maximum potential total and differential settlements. In general, no additional load increases beyond the actual magnitude of off-loading was permitted and the weight of the new structure was kept approximately the same weight as weight of soil that is being excavated. The mat foundation design for the proposed sub-structures was made feasible by appropriate detailing and modifications to ensure structural flexibility and integrity. Finally, appropriate MSR design values were developed for design of mat foundation for the new building, as per section 7.7 of CFEM (2006).

6.3 Protection of Sensitive Clays Against Mechanical Disturbance and Chemical Weathering

The sensitive clays on Site are subject to softening when exposed to excess moisture or disturbance. Contractors should employ construction methods which limit construction traffic over exposed clay subgrade surfaces and keep exposure to excess moisture to a minimum. Excavations through the clay should be completed with a smooth-edged bucket to minimize disturbance to subgrades. It is recommended that a mud-slab be employed as a protective layer and to provide a clean surface to the build rebar and formwork.

6.4 Evaluation of Potential for Foundation Uplift and Basal Heave

Based on the water levels encountered in the monitoring wells, it is expected that designers will need to resist uplift forces caused by buoyancy. Due to the low permeability of the clays, it is recommended that when designing against foundation uplift, buoyancy calculations be performed considering that the groundwater level is at the surface grade. Buoyancy forces can be estimated using the following formula:

$$U = \gamma \, Vs \qquad [4]$$

Where: U = uplift force by buoyancy (kN), ɣ = unit weight of water (9.8 kN/m^3), Vs = volume of submerged structure below groundwater (m^3).

Buoyancy forces are typically resisted by increasing the dead weight of the structure itself, or with the use of additional anchors. Buried chambers can have their buoyant forces resisted using a buried retaining ring. If used, designers are able to consider the weight of the soil above the retaining ring in uplift resistance calculations. In designing a retaining ring, the height of the ring will need to be such that only the soil above the ring, and a zone of influence drawn upward and outward by 2V:1H, should be considered. New backfill within this zone should be a free-draining non-frost susceptible soil, such as an Ontario Provincial Standard Specification (OPSS) 1010 '*Granular B, Type I*'. A bulk unit weight of 20.0 kN/m^3 above the water level can be used for uplift calculations. A submerged unit weight of 10.2 kN/m^3 must be used below the water table.

6.5 Impact of Clay Sensitivity on Frictional Resistance

To minimize potential disturbance of load bearing sensitive clay deposit below the new deep chamber driven steel sheet piles may be utilized extending to significant depth. Based on the reported subsurface conditions to date, the skin friction (adhesion) values for the unweathered grey silty clay deposit are estimated to vary depending on the degree of potential disturbance during construction as presented in Table 6.

Considering the measured sensitivities of 2 to 10 from the FVTs, the available skin friction (adhesion) is highly dependant on the installation method and how much the chosen method disturbs the clay. The design skin friction (adhesion) parameters specific to their installation method should be selected and reviewed by experienced Geotechnical Engineers in shoring design in weak sensitive clays.

Table 6. Estimated ultimate limit state (ULS) skin friction

Material Interface with Gray Clay	Undrained Shear Strength[1], C_u (kPa)	Recorded Sensitivities, S_t (kPa)	ULS Adhesion[1] (kPa)
Concrete Sheet Piles Secant Piles	15-25[1]	2-10	10-18[1]

[1]For undisturbed clay deposit only (if the construction/ installation process disturbs subsoils, then the suggested values must be divided by S_t for design.

6.6 Planning and Execution of Deep Excavations

Deep excavations within these sensitive clays will not be straight-forward, especially considering the rising groundwater table and the low shear strengths and sensitivity of the clay deposit at the proposed excavation elevations. Therefore, an experienced contractor with significant working experience in similar conditions is required, and pre-qualification of excavation contractors should include submission of a formal Excavation Plan including their proposed Shoring methods versus open cut and sunken-shaft methods for all proposed structures in advance of construction.

6.7 Dewatering and Construction Monitoring

The results of groundwater monitoring from three monitoring wells installed with screens at varying depths throughout the clay deposit (Tables 2 and 3), indicates that the recorded water levels have been rising over time with the highest ground water level at approximately 1.8 mbgs (El. 84.6 masl), but it could potentially rise up to the existing grade during a rainy/wet period/year. Therefore, the design for all proposed underground infrastructure in submerged conditions include the full potential uplift/buoyancy forces, where applicable. For example, the excavations for the Combined Control Building/Swab Launch Chamber are to extend to an approximate depth of 4.8 mbgs (El. 81.7 masl). The excavation for the Bypass Chamber, if completed using deep excavations and Engineered Shoring will extend to an approximate depth of 12.4 mbgs (El. 74.5 masl). Therefore, excavations for this Project will extend well below the anticipated water table.

Both surface water and groundwater seepage are expected in all excavations and will need to be controlled. Water quantities will depend on seasonal conditions, depths of excavations, presence and lateral extents of water bearing silt and sand seams, and the duration of the open excavations. Groundwater will travel easily through the fill material, and especially near the fill-native interface. Furthermore, any existing utility trenches which join or intersect the excavations may act as a drain and supply off-Site water into the excavations. These may need to be plugged or grouted at the outset of construction to mitigate the possibility. This may include consideration of prevailing groundwater chemistry and pumping from sumps, ditches, and/or well points in such a way to minimize potential disturbance of the clay structure.

Comprehensive construction dewatering techniques by a specialized dewatering contractor will be required during construction in such a way to avoid potential disturbance or collapse of the structure of the load bearing marine clay deposit. This may include pumping from sumps, ditches, and/or well points. Recommendations for appropriate dewatering measures beyond conventional sump pump techniques such as a positive dewatering system to temporarily lower the static groundwater level must be provided by a specialized dewatering contractor based on the findings and recommendations of the hydrogeological assessment. The dewatering efforts will depend on a number of factors, including excavation depths, season, weather conditions, and the length of time the excavations are kept open. However, in our opinion, it should be left to the contractor to determine the means & methods of dewatering necessary to meet the project performance requirements, and align with their construction methodology and schedule.

Following review of final foundation and earthworks design drawings and specifications issued for construction, a rigorous monitoring program has been proposed which included pre-construction surveys and vibration monitoring of nearby sensitive infrastructure and houses; development of a risk management plan and subsequent monitoring of foundations and earthworks construction and observations of settlement of adjacent structures and infrastructure throughout the project.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

Braja M. Das, 2006, Principles of Geotechnical Engineering, Sixth Edition.

Canadian Foundation Engineering Manual (CFEM), 4th Edition, 2006. Canadian Geotechnical Society.

Canadian Foundation Engineering Manual Errata (2013)

Canadian Geotechnical Society (2006). Canadian Foundation Engineering Manual.

Canadian Standards Association, July 2009, A23.1-09/A23.2-09, Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete.

Chapman, L. J. and Putnam D. F. 1984, *The Physiography of Southern Ontario*, 3rd ed., Ontario Ministry of Natural Resources.

City of Ottawa (2004), Extent of Champlain Sea and Sensitive Clay Deposits, Available from http://documents.ottawa.ca/sites/documents.ottawa.ca/files/documents/cap118405.pdf

E. Penner (1965), A Study of Sensitivity in Leda Clay, National Research Council, May 1965.

E. Penner and K.N. Burn (1977), Review of Engineering Behaviour of Marine Clays in Eastern Canada, Published by National Research Council, May 1978.

Excerpts from Norwegian Geotechnical Institute Publication Number TRIS Accession No. 00235335 dated Sept.16, 1968 by T. Loken) entitled Formation of Quick Clay and Chemical Weathering in Norwegian Clays (International Transport Forum/Transportation Research Board (TRB) & The National Academics of Sciences Engineering Medicine via TRID (TRIS & ITRD database online).

Hache, R, Saadeldin, R., El-Dona A., Girouard, E. (2018), A Proposed Reporting Approach for Champlain Sea Clays and Other Similar Deposits, Proceedings of Can. Geotech. Society Geo-Edmonton Conference

Kenneth L. Lee (1978), Cyclic Strength of a Sensitive Clay of Eastern Canada, Published by National Research Council, Study at School of Engineering and Applied Science, University of California.

Rendon Herrero (1983)

Roy E. Hunt (2007) Book entitled Characteristics of Geologic Materials and Formations, A Field Guide for Geotechnical Engineers

Leroueil, S. (1997), Geotechnical Characteristics of eastern Canada Clays, Proceedings of the International Symposium on the characterization of soft marine clays.

Leroueil, S. (1983), Propriétaires Characteristiques des Argiles de l'Est du Canada', Can. Geotech. Journal 20 (P. 681-685).

Nagaraj and Murty (1985), Prediction of the Pre-consolidation Pressure & Recompression Index of Soils, Geotech. Testing Journal, Vol.8, No.4, 199-202.

Occupational Health and Safety Act and Regulations for Construction Projects, Ministry of Labour Publications

Skempton (1944), Notes on the Compressibility of Clays, Quarterly Journal of the Geological Society of London, Vol. 100, 119-135.

Updated Slope Stability Guidelines for Development Applications in the City of Ottawa (2012).

Storer J. Boone (2010), A Critical Reappraisal of "Preconsolidation Pressure" interpretations using the oedometer test, Can. Geotech. Journal 47, pages 281-296.