Preliminary Geotechnical Investigation

Proposed 6-Storey Marriot Hotel
40 Frank Nighbor Place
Kanata, ON

401 Real Estate Trust
C/O
API Development Consultants Inc.
1464 Cornwall Road, Unit 7
Oakville, ON
L6J 7W5

May 10, 2023
Englobe Ref No: 02211293.000

For 401 Real Estate Trust

Prepared by:

LICENSED PROFESSIONAL ENGINEER
May 11, 2023
S.A.K. RATMONO
100212722
PROVINCE OF ONTARIO

Shanti Ratmono, M.Eng., P.Eng.
Project Manager
Geotechnical Engineer

Reviewed by:

LICENSED PROFESSIONAL ENGINEER
May 11, 2023
S. W. DUNSTAN
100152399
PROVINCE OF ONTARIO

Shane Dunstan, P.Eng.
Team Lead
Geotechnical and Materials - East

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1 Introduction

Englobe Corp. (Englobe) is pleased to present the findings of our preliminary pre-design geotechnical investigation and foundation recommendations report for the proposed 6 storey Marriot Hotel (Project) located at 40 Frank Nighbor Place in Kanata, Ontario (Site).

Englobe was retained by API Development and Consultants Inc. (Designer) on behalf of 401 Real Estate Trust (Client) to carry out a preliminary geotechnical investigation consisting of 3 boreholes and 1 monitoring well advanced within the footprint of the proposed building. A signed authorization to proceed with this investigation was provided by Mr. Drew Barlow of the Client on January 03, 2023.

This report is prepared for the sole use of the Client and their Designers. The use of the report, or any reliance on it by any third party, is the responsibility of such third party. This report is subject to the limitations shown in Appendix A. It is understood that the Project will be performed in accordance with all applicable codes and standards present within its jurisdiction.

2 Site and Project Description

The Site of the proposed development is located at 40 Frank Nighbor Place in Kanata, Ontario. The existing Site is an undeveloped lot bounded by vacant farmland to the north, a gymnasium to the east, Carp River to the west, and a parking lot to the south. The location of the Site is shown on Figure 1: “Site Location Map” provided in Appendix B. The Site is relatively flat and is at a similar elevation to the roadway. To the west, the Site gradually slopes downward toward the Carp River. Based on a review of the publicly available aerial photos on the City of Ottawa’s geoOttawa GIS viewer, it appears there may be historical filling and grading on Site prior to 2002.

Englobe’s understanding of the Site and the Project is based on the following drawings provided by the Client:

- ‘Site Plan’, Drawing No. A-100, dated February 22, 2023, prepared by Saplys Architects Inc.
- Plan Drawings, Drawing No. A200, A201, A201A, A201B, A201C, A201D, A202, A202A, A203, A205, dated February 22, 2023, prepared by Saplys Architects Inc.; and
- Elevation Drawings, Drawing No. A301 to A305, dated February 22, 2023, prepared by Saplys Architects Inc.

The proposed building will cover an approximately area of 1,691.5 m^2 and will contain 1 partial basement level. Based on the architectural and Site plans received, we understand the ground floor of the building will be at an approximate elevation near 95.6 m above sea level (masl). Therefore, the partial basement will be at an approximately 3.0 meters deeper at an approximate elevation near 92.6 masl.

The existing Site is at an approximate elevation near 94.9 to 95.3 masl, therefore there will be an approximate grade raise of 0.3 to 0.7 m across the Site.

At the time of preparation of this report, Englobe has not been provided with any structural drawings of the proposed foundations for the new development. It is understood that the Project is currently in the pre-design stage. Therefore, it important to emphasize that the general recommendations in this report should be considered as preliminary in nature at this stage. Englobe should be retained to review the proposed foundation drawings and grading plans once they become available to ensure conformance with the general recommendations provided within this report.

3 Scope of Work

Englobe’s geotechnical scope of the work was outlined in our proposal (Ref No: P2211293.000, dated December 5, 2023) and was agreed to by the Client on January 3, 2023 by means of a signed offer of services. In general, our mandate was limited to the following activities:

- Retain a utility subcontractor to provide both public and private underground utility clearances;
- Retain a geotechnical drilling subcontractor to drill the following boreholes with a track mounted drill rig:
    - One borehole advanced to auger refusal plus 1.5 m of rock coring, or to a maximum depth of 20.5 m.
    - One borehole advanced to auger refusal, or to a maximum depth of 10 m.
        - A monitoring well installed in the overburden
    - Two boreholes advanced to a maximum depth of 6 m.
- Supervise the fieldwork and log the subsoils conditions at the borehole locations based on the samples recovered.
- Submit representative soil samples to the geotechnical laboratory for further testing; and
- Prepare this geotechnical investigation report.

The work was performed consistent with the agreed scope of work.

4 Field Investigation and Laboratory Testing

4.1 Geotechnical Drilling Fieldwork

The drilling component of this current geotechnical investigation was performed on February 21 to 23, 2023. The drilling consisted of the advancement of 3 boreholes and 1 monitoring well within the footprint of the proposed building. The boreholes and monitoring wells were designated as: BH23-01, MW23-02, BH23-03 and BH23-04, and were drilled at depths ranging from 6.1 to 20.3 meters below ground surface (mbgs). All boreholes terminated within the silty clay, except for BH23-04 that which had additional rock coring. The location of the boreholes is shown on the Figure 2: "Borehole Location Plan" provided in Appendix B.

A geotechnical drilling subcontractor, CCC Geotechnical & Environmental Drilling Ltd., was retained to perform the drilling. All boreholes were drilled using a track mounted drill rig. The boreholes were advanced through the overburden using continuous-flight hollow-stem augers and into the bedrock using double barrel wireline diamond coring methods. Monitoring well MW23-02 was installed with screen sealed into the silty clay overburden.

Samples of the clayey soils were collected in the boreholes using a standard 50 mm outside diameter split-spoon (SS) sampler driven at 0.75 m intervals by an automatic Standard Penetration Test (SPT) hammer. Selected undisturbed samples were collected using thin wall (TW) Shelby tubes. The compaction of the cohesionless soils was assessed using recorded SPT N-values and the shear strength of clayey soils was assessed using Field Vane Tests (FVT) and Pocket Penetrometer (PP) resistance values.

The subsurface soil, bedrock, and groundwater conditions at the borehole locations were logged by Englobe field staff based on the samples that were recovered. The recovered soil and rock core samples were submitted to Engobe's Ottawa geotechnical laboratory for further review and geotechnical laboratory testing on selected samples.

The ground surface elevation of the boreholes was surveyed by Englobe field staff using a total station and related to the geodetic elevation of existing sanitary manhole lids of 94.62 and 94.64 masl located on Frank Nighbor Place. The ground surface elevations of the boreholes are shown on the Borehole Logs provided in Appendix C.

4.2 Geotechnical Laboratory Testing

The laboratory testing component of this geotechnical investigation consisted of consolidation testing on 3 representative silty clay samples, 7 Atterberg limit tests, 3 unit weights, and moisture content testing on all recovered soil samples. The results of the laboratory testing are presented on the Borehole Logs provided in Appendix C and the Laboratory Test Results provided in Appendix D.

One soil sample was submitted to the environmental laboratory for testing of selected concrete attack and metal corrosion parameters (pH, sulphide, sulphate, redox potential, chloride, and resistivity). Soil sample BH23-04 SS2 was submitted to Paracel Laboratories under chain of custody number 138077. The results were received on February 28, 2023 under report number 2308193. The results of the laboratory testing are attached in Appendix D and further discussed in Subsection 6.9.

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5 Description of Subsurface Conditions

The subsoil conditions encountered at the borehole locations are briefly discussed in the following subsections with a graphical representation of Site-specific conditions at each location presented on the Borehole Logs provided in Appendix C. A summary of the general stratigraphy encountered in the boreholes advanced at this Site is presented in Table 5-1 below.

Table 5-1: Summary of Borehole Stratigraphy

Borehole ID	Approximate Topsoil Thickness (mm)	FILL Elevation (masl)	Silty Clay (Crust) Elevation (masl) Depth (m bgs)	Unweathered Silty Clay Elevation (masl)	TILL Elevation (masl)	Bedrock Elevation (masl)
BH23-01	-	95.0 - 91.4 (0-3.6)	-	91.4-88.9[1] (3.6-6.1)	-	-
MW23-02	40	-	95.1-92.8 (0-2.3)	92.8-85.3[1] (2.3-9.8)	-	-
BH23-03	-	-	95.0-92.7 (0-2.3)	92.7-88.8[1] (2.3-6.2)	-	-
BH23-04	-	-	95.0-92.7 (0-2.3)	92.7-79.8[1] (2.3-15.2)	79.8-78.2 (15.2-16.8)	78.2-74.7[1] (16.8-20.3)

[1] End of Borehole (EOB)/Termination Depth

It is important to note that the soil descriptions presented below and in the Borehole Logs represent the soils encountered at the test locations only. They may vary between and beyond borehole locations. This is especially true in previously excavated and/or filled areas such as near existing and former utility trenches.

5.1 Topsoil

An approximately 40mm thick topsoil layer was encountered within MW23-02. In the remaining borehole locations, there was no topsoil cover as the surface was tilled for the fall.

It is important to note that the topsoil thickness noted above are for planning purposes only. They should not be used for quality assessments or quantity take-offs.

5.2 FILL

Cohesive FILL consisting of silty clay trace gravel was encountered at ground surface within BH23-01, and extended to a depth of approximately 3.6 mbgs, corresponding to elevation of 91.4 masl.

5.3 Silty Clay

A native very stiff to firm silty clay was encountered at ground surface in BH23-03, and BH23-04, below the topsoil in MW23-02, and below the FILL in BH23-01. The native silty clay extended to the termination depths of BH23-01, MW23-02, BH23-03 and to a depth of approximately 15.2 m in BH23-04, corresponding to elevation of approximately 79.8 masl. The native silty clay was first encountered in a weathered crustal state at ground surface and extending to depths of approximately 2.3 mbgs, corresponding to elevation ranging from approximately 92.8 to 92.7 masl. Below the upper desiccated crust, the silty clay was generally found in an unweathered condition. The weathered silty clay was brown in colour, and the unweathered material was mainly grey in colour. The unweathered silty clay extended to the termination depths of BH23-01, MW23-02, BH23-03 and to a depth of approximately 15.2 m in BH23-04, corresponding to elevation of approximately 79.8 masl.

The moisture contents of the silty clay material ranged from 30 to 63%. The moisture contents of the recovered samples are presented on the Borehole Logs in Appendix C.

Field vane tests performed in the silty clay crust measured undrained shear values between from approximately 74 to 172 kPa, which generally indicates a stiff to very stiff consistency. Field vane tests performed in the unweathered silty clay measured undrained shear values between from approximately 35 to 117 kPa, which generally indicates a firm to very stiff consistency, but is mainly in a firm to stiff consistency. The corresponding sensitivity of the unweathered silty clay ranged from 2 to 36, suggesting the silty clay deposit is medium sensitive to quick in accordance with Section 3.1.3.4 of the Canadian Foundation Engineering Manual, 4th Ed., 2006 (CFEM 2006).

Englobe previously tested undisturbed samples of the silty clay obtained using thin-walled shelby tube samplers at an adjacent Site. The undisturbed soil samples were sent for one-dimensional consolidation by oedometer, Atterberg Limits, and unit weight testing. The results of the previous laboratory geotechnical testing are presented in Table 5-2 and shown graphically with elevation on Figure 3: “Geotechnical Model” provided in in Appendix E.

Table 5-2: Summary of One-Dimensional Consolidation and Unit Weight Test Results

Elevation	Consolidation Parameters*				Bulk Unit Weight, γ (kN/m^3)
	Initial Void Ratio, e_o	σ'_p (kPa)	C_c	C_r	
Current 2023 Investigation (this Site)					
Weathered Crust					
92.7 to 92.1	1.33	225	0.43	0.08	16.7
Unweathered Silty Clay					
88.9 to 88.3	1.97	120	1.30	0.1	15.6
85.2 to 94.6	1.55	120	1.03	0.03	16.7
Previous 2022 investigations on adjacent property					
Weathered Crust					
91.8 to 91.2 m	1.305	300	1.04	0.003	16.2
Unweathered Silty Clay					
88.8 to 88.2 m	1.174	156	1.16	0.05	16.9
85.7 to 85.1 m	1.252	161	1.36	0.03	16.4

Six soil samples of the silty clay were tested for Atterberg limits and the results are summarized in Table 5-3 below.

Table 5-3 Summary of Atterberg Limits in Silty Clay

Sample ID	Elevation (masl)	LL (%)	PL (%)	PI (%)	WC (%)	Description
BH23-04, TW1	92.7-92.1	57	22	35	42	Clay of High Plasticity (CH)
BH23-04, TW2	88.9-88.3	52	24	28	71	Clay of High Plasticity (CH)
BH23-04, TW3	85.1-84.5	40	21	19	56	Clay of Low Plasticity (CL)
BH23-04, SS7	85.9-85.3	52.5	20.0	32.5	62.8	Clay of High Plasticity (CH)
BH23-04, SS9	82.8-82.2	56.3	20.3	36.0	51.5	Clay of High Plasticity (CH)
BH23-04, SS10	81.3-80.7	54.6	24.2	30.4	51.3	Clay of High Plasticity (CH)

5.4 Glacial Till

A glacial till layer consisting of clayey silt trace gravel was encountered below the silty clay layer in BH23-04. The depth of the glacial ranged from approximately 15.2 to 16.8 mbgs, corresponding to approximate elevations ranging from 79.9 to 78.3 masl.

One soil sample of the till was tested for Atterberg limits and the results are summarized in Table 5-4 below.

Table 5-4 Summary of Atterberg Limits in Till

Sample ID	Elevation (masl)	LL (%)	PL (%)	PI (%)	WC (%)	Description
BH23-04, SS11	79.8-79.2	17.2	11.1	6.1	27.6	Clayey Silt (CL-ML)

5.5 Limestone Bedrock

Bedrock was encountered in BH23-04 at an approximate depth of approximately 16.8 mbgs. This corresponds to an approximate elevation near 78.2 masl. The bedrock was confirmed by coring. Bedrock consisted of grey, poor to good quality, slightly weathered limestone. The rock quality designation (RQD) of the core sample was calculated to range from 27 to 72%.

Rock core photos are provided in Appendix E.

5.6 Groundwater

A monitoring well was installed in MW23-02 and screened within the silty clay overburden from a depth of approximately 2.5 to 4.6 mbgs, corresponding to elevations of 90.5 and 92.6 masl, respectively. The water level was measured within the monitoring well on February 24, 2023 and was measured to be at a depth of approximately 2.0 mbgs, corresponding to elevation of approximately 93.1 masl. This corresponds to the water level being within the desiccated crust layer. Monitoring well details and water level measurements are shown on the Borehole Logs provided in Appendix C.

It should be noted that groundwater levels are subject to seasonal fluctuations and response to precipitation, flooding, and snowmelt events. Typically, they are at their highest during the spring thaw. It is important to emphasize that a hydrogeological investigation in support of PTTW, EASR application or dewatering volume estimate was not requested at the time of this geotechnical investigation.

6 Discussion and Recommendations

Based on the results of geotechnical field and laboratory investigation performed, the following discussion is provided to assist the Client and their Designers with the development of foundation general arrangements and geotechnical design for the proposed Project. The recommendations provided within this report are based on our understanding of the proposed Project which is summarized above in Section 2 and are general in nature. If any of these understandings change, then Englobe should be contacted to assess the implications of those changes on the recommendations provided herein.

Based on the soil conditions encountered at the borehole locations, and assuming that they are representative of the soil conditions across the Site, the most important geotechnical considerations for the design of the foundations for the proposed Project are expected to be the following:

- **Pre-Design Geotechnical Investigation:** It is understood that this Project is currently in the pre-design stage. Therefore, it important to emphasize that this report should be considered as preliminary in nature. Englobe requests to be retained to review the contemplated foundation and earthworks designs once they become available to provide the necessary comments to ensure conformance with the general recommendations provided within this report.
- **Grade Raise Limitations:** The native clayey soils on this Site are subject to consolidation settlement. The bearing pressures provided within this report are based on the assumption that there are no significant (i.e. greater than 0.7 m) grade raises planned for this Site. If grade raises are envisioned, then additional case-by case settlement analyses and revision of serviceability limits will likely be required to assess the impact.

- **Low Bearing Resistances:** The native unweathered clayey soils on this Site are subject to consolidation settlement, and the proposed foundation depth is below the crust. Therefore, it is recommended that this structure be founded on end-bearing piles driven to refusal. Additionally, it may be feasible to construct a fully compensated raft foundation depending on the building loads, however, additional investigation will be required. Within this report Englobe is providing preliminary raft design parameters for Designers to assess raft feasibility. These will need to be confirmed using additional sCPTu testing.
- **Assumed Slab on Grade Loadings:** A typical floor slab loading for a lightly loaded slab on grade would involve a maximum pressure of 24 kPa. Englobe has not been provided with any specific floor slab requirements such as racking, process equipment or other concentrated loadings. If higher distributed loads or specific point loads resting directly on the slab are envisioned, then Englobe should be retained to perform additional consulting in regard to design of the floor slab.

6.1 Site Preparation

All existing FILL soils, surficial topsoil, vegetation and/or other deleterious materials (e.g. any loose, wet, and/or otherwise disturbed native materials), debris, or disturbed soils must be completely removed within the footprint of the new building, down to competent undisturbed native silty clay soils capable of supporting the proposed development.

The Site surrounding the excavation should be graded in the early stages of construction to provide for positive control of surface water and directing it away from the excavation and subgrades. Appropriate provisions should be made for collection and disposal of groundwater, storm water, and runoff including an adequate pumping system.

6.1.1 Subgrade Preparation

Excavations for footings or rafts should extend below all FILL soils down to the native undisturbed stiff to firm silty clay, and below the design frost depth. Based on the boreholes, the native silty clay is expected to be encountered close to ground surface at approximately elevation ranging from 95.1 to 95.0 masl, however the frost depth is 1.5 mbgs for heated buildings.

All footing or raft subgrades must be evaluated and approved by a Geotechnical Engineer to ensure that the native subgrade is free of any organics, roots, FILL, loose or disturbed soils and can support the design bearing pressure. Any identified local anomalies or soft spots should be subsequently sub-excavated, replaced with new Engineered Fill in accordance with the comments in Section 6.12.

The existing silty clay on this Site is sensitive to strength loss upon disturbance. If it is disturbed by over-excavation, remoulding, equipment and foot traffic, or subjected to excess water, it will lose its initial strength and will need to be sub-excavated. Contractors should use excavation methods that minimize disturbance to the clay subgrades. Final excavations should be performed with a smooth-edged ditching bucket. It is recommended that designs incorporate the use of a lean mix concrete mud mat on the approved subgrade surfaces to protect the sensitive clay and to provide for a clean dry working surface to construct the footing.

6.2 Excavations

Based on Englobe's current understanding of the Project, we anticipate that the deepest excavations will be at an approximate depth of 3.0 m bgs for the partial basement level, corresponding to a minimum approximate elevation near 92.6 masl. Excavations will extend through the native clay crust. Excavations for the proposed building will also be below the observed water level. Based on the excavation depths required, it is anticipated that excavations will be performed using sloped open excavations where space permits.

All excavations must be undertaken in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA) Regulations for Construction. The comments within this subsection are intended to be an addition to, and not a replacement of the current OHSA requirements.

- The existing native silty clay would be considered as a "Type 3 Soil" according to the regulations. However, if it becomes wet, muddy, or below the water level it would become a "Type 4 Soil". According to the OHSA, excavations which penetrate through multiple soil types should be considered as having the highest soil type.

The stability of the excavation side slopes is highly dependent on the Contractor's methodology and layout. No surface surcharges should be placed closer to the edge of the excavation than a distance equal to twice the depth of the excavation unless an excavation support system has been designed to accommodate such a surcharge.

Designers and Contractors should review the geometry, depth, and sloping requirements of all planned excavations. Proposed excavation dimensions should be compared to adjacent load bearing structures, to ensure they are not undermined. Undermining is avoided by ensuring that no excavations penetrate below an imaginary line drawn outward and downwards 7V:10H below the toe or founding level of any load bearing structures. If the limit of not undermining adjacent structures cannot be satisfied, then an Engineered Shoring system and/or underpinning program will need to be considered.

The existing silty clay on this Site is sensitive to strength loss upon disturbance. If it is disturbed by over-excavation, remoulding, equipment movement, foot traffic, or subjected to excess water, it will lose its initial strength and will need to be sub-excavated. Contractors should use excavation methods that minimize disturbance to the clay subgrades.

6.3 Temporary Construction Dewatering

Contractors should be prepared to handle any surface or groundwater infiltration by ditching, pumping and/or other methods in order to maintain dry working conditions. If excavations intercept existing or former service trenches, then the backfill in these trenches could act as a drain supplying unexpected offsite water into excavations.

As discussed in Section 5.6, monitoring well MW23-02 was installed at the Site. The water levels recorded on February 24, 2023 was measured to be at a depth of approximately 2.0 mbgs, corresponding to elevation of approximately 93.1 masl. Given that excavations are expected to extend below to an approximate elevation near 92.6 masl, the excavation will extend below the groundwater table.

Hydrogeological consulting in support of a Permit to Take Water (PTTW) or an Environmental Activity Sector Registry (EASR) application were not within Englobe's scope of work. Assessments of the quantity of water to be expected in excavations, was not part of Englobe's scope of work.

It should be emphasized that dewatering can cause ground settlement that extends laterally beyond the immediate area of dewatering. It is recommended that the contractor assess the likely impact on nearby existing structures, underground services, roadways, groundwater wells and use methods which will control the dewatering impact. A pre-construction survey documenting the conditions of nearby settlement-sensitive facilities/infrastructure be completed prior to start of construction.

6.4 Foundations

Based on our understanding of the proposed Project and the soil conditions encountered within the boreholes, it is strongly recommended that the foundations for the proposed building be constructed on end-bearing piles driven to refusal. A single raft foundation may also be feasible, however additional investigation would be necessary to confirm the resistance. Englobe is providing only preliminary raft subgrade moduli so Designers can consider if the low raft resistance values are feasible.

6.4.1 Option 1: Deep Foundations

Axial Capacity

If the proposed building requires higher bearing capacities than those provided for raft foundations on the undisturbed stiff to firm silty clay, pile foundations may be contemplated. Steel H-Piles or concrete-filled steel end-bearing pipe piles driven to refusal on bedrock are considered viable deep foundation options for this Site. Good quality bedrock is encountered within BH23-03 at approximate elevation of 76.2 masl.

For such end-bearing piles driven to bedrock refusal, a SLS condition for a nominal 25 mm of settlement is not applicable. The following table presents typical values of the anticipated factored bearing resistance of some available pile sizes under ULS conditions. This table is intended to provide assistance to the designer in estimating approximate quantities and possible layout of piles within the structure in conceptual/preliminary design stage. It should be noted that the actual achievable pile resistance depends on several parameters as discussed below and could vary considerably across the Site, therefore supplementary geotechnical investigation and review of conceptual foundation design are recommended for detailed design.

Table 6-2: Typical Values of The Anticipated End-Bearing Resistance of Some Available Pile Sizes Under ULS Condition

Pile Type	Pile Designation Imperical (Metric)	Preliminary Factored Axial Capacity at ULS
H-Piles	HP8x36 (HP200x54)	1175 kN
	HP10x57 (HP250x85)	1275 kN
	HP12x74 (HP310x110)	1675 kN
Concrete Filled Pipe Piles	7” x 0.375” (178 x 9.5)	725 kN
	9 5/8” x 0.500” (245 x 12.7)	1375 kN

On private construction projects such as this, it is typical for the piling to be performed under a design-build type contract. Given the required resistances provided by the Structural Engineer, Piling Contractors will provide the most economical piles depending on the equipment and material availability. The values provided above are approximate because of anticipated variability of bedrock conditions across the Site, the methods used by Piling Contractors to drive the piles and determine pile capacity could vary somewhat from one Contractor to another. However, the preliminary values provided may be considered for preliminary estimation of the number of piles required.

For short piles driven to bedrock refusal, the design is not expected to be governed by the SLS conditions. For preliminary design purposes the settlement of piles driven to sound bedrock under SLS conditions is generally expected to be less than 5 mm (excluding the elastic deformation of the piles themselves), and less than 10 mm in total settlement.

The Piling Contractor will need to confirm the estimated pile capacity considering the driving energy of their proposed equipment using approved empirical methods at the outset of the Project. The Piling Contractor's piling calculations should be carried out according to Section 18.2 of the CFEM-2006. Typical piling calculations would include the Hiley formula, wave equation, or other methods based on the Contractor's equipment. The Geotechnical Engineer must be retained to review and approve the piling calculations prior to mobilization and confirm the development of the necessary piling refusal criteria for use with this Project at the onset of piling operations.

Englobe recommends that the installation of all piles be witnessed and reviewed by a Geotechnical Engineer or a qualified Technician acting under the supervision of a Geotechnical Engineer on a full-time basis to verify the tip elevation, location, verticality, and to ensure that the design set criteria and the required pile capacity has been achieved. Pile splices will require inspection by a CWB welding inspector.

It is recommended that Pile Driving Analysis (PDA) be performed on a minimum of 10% of the piles and be completed at onset and production stages of the Project. First at the onset of pile driving to confirm the set criteria established for this Project; and secondly, on any piles that are considered suspect. In addition, re-striking of all piles is recommended for this Site to ensure that uplift of adjacent piles is avoided.

6.4.2 Option 2: Preliminary Feasibility of Raft Foundations

If the Client and Designers wish to explore the feasibility of raft foundations for this building, it would consist of a single raft foundation founded at the basement level near elevation 92.6 masl. The current architectural drawing indicates only a partial basement. **In order to make a raft foundation feasible, the entirety of the building would need to have a basement level to take advantage of some of the unloading of the basement excavation.**

All existing FILL soils, surficial topsoil, vegetation and/or other deleterious materials (e.g. any loose, wet, and/or otherwise disturbed native materials), debris, disturbed soils are not suitable for bearing of any foundation elements. Therefore, excavations for raft foundations should extend down to the native undisturbed stiff to firm silty clay, or alternatively placed on new Engineered Fill resting on native undisturbed stiff to firm silty clay deposit. Based on the limited available boreholes data, native clay deposit is expected at near ground surface at elevations near 95.0 masl.

It is important to note that the preliminary parameters presented below are only to assist the designer during the feasibility assessment. If designers wish to further explore this option, the additional geotechnical investigation including consisting of Cone Penetration Tests (SCPTu) will be required to confirm consolidation parameters. The design of raft slabs are an iterative process where the geotechnical consultant provides initial spring constants and then the Structural Engineer estimate pressure distributions below the raft. Then the geotechnical consultant can revise the modulus values based on the loads. This process is repeated several times until the moduli stabilize.

For a raft foundation founded on Engineered Fill or on native undisturbed stiff to firm silty clay, a factored Ultimate Limit States (ULS) design bearing resistance of 45 kPa is suggested. This includes for a geotechnical resistance factor of $\Phi = 0.5$. However, the design of raft foundation on compressible silty clay deposit is expected to be governed by tolerable settlement under SLS conditions. Considering the anticipated elevation of the proposed basement (3.0 m below grade) and considerable off-loading of the native silty clay deposit, Table 6-1 below presents the results of the settlement estimates for assumed SLS design bearing pressures for the building considering the expected founding elevation, and raft foundation dimensions.

Table 6-1: Preliminary Raft Resistance Values for Feasibility Screening Only

Assumed Foundation Dimensions and Elevations	
Proposed Building	27 x 80 m
Foundation Elevation	92.6 masl
Bearing Capacity	
Assumed Gross SLS design bearing resistance (considering off-loading due to basement excavation)	30 kPa
Allowance for global grade raise	0.7 m x 20 kN/m^3 = 14 kPa
Englobe Estimated Settlement Results	
Estimated settlement under suggested SLS design bearing pressure	Up to 25 mm
Estimated Modulus of Subgrade Reaction (MSR) at design pressure including grade raise and foundation dimensions	MSR= 0.5 MPa/m
Typical tabulated MSR based on 0.3 m x 0.3 m square reference area for stiff clays	K_{v1}= 10 to 30

Based on the results of consolidation testing to date and depending on the settlement tolerance of the proposed building, under assumed gross SLS design bearing pressure of 30 kPa foundation settlements of up to 16 mm of could be experienced at the center of the raft and settlements of up to 25 mm could be experiences at the corner of the raft. The typical K_{v1} 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. Designers are referred to section 7.7 of the CFEM-2006 for further information on the use of MSR values.

The clays on this Site are generally slightly over consolidated, but they are very sensitive to even minimal grade raises. Grade raises must be limited to a maximum of 0.7 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.

If the proposed foundation general arrangements and designs are not able to accommodate the estimated design bearing pressures provided above, then the available options would be to reduce the anticipated applied loads, or alternatively consider the use of deep foundation solutions such as driven H-piles or concrete filled pipe piles supported on bedrock.

Subgrade preparation below the foundation will involve removal of all fill soils, organics, disturbed/remoulded or previously excavated soils to expose a native undisturbed stiff to firm silty clay subgrade. The exposed surface should be examined by the Geotechnical Engineer to assess the competency. Any identified local anomalies, soft spots, or disturbed/remoulded areas should be subsequently excavated and replaced at the direction of the Geotechnical Engineer.

All excavations in the clay should be performed with a smooth-edged ditching bucket to ensure that the footing subgrade is undisturbed. It is recommended that Contractors employ a lean mix concrete mud-slab on the approved clay subgrade daily. This will serve as a clean and level working mat upon which to assemble the rebar.

It is recommended that Englobe be retained to complete a review for compliance with our recommendations and during construction to verify suitability of subgrade materials.

6.5 Frost Protection

All footings for heated structures must be provided with a minimum of 1.5 m of earth cover, and 1.8 m of earth cover for unheated or isolated structures in the Ottawa area. Otherwise, an equivalent insulation detail would be required in order to provide adequate protection against frost action. Where soil cover cannot be provided, an insulation detail should be designed or approved by a Geotechnical Engineer. Contractors must be aware that this detail may be such that the insulation may need to be placed below the footing and then the footing poured on top, and therefore pre-approval is recommended to ensure excavations and backfill are properly planned.

Should construction take place during winter, surfaces that support foundations or Engineered Fill must be protected by Contractors against freezing for the entire duration of construction or until adequate soil cover is in place. Backfill soils should not be placed in a frozen condition or placed on frozen subgrades.

6.6 Seismic Site Classification

In accordance with the Ontario Building Code 2012 (OBC-2012), structures designed under Part Four of the Code must be designed to resist a minimum earthquake force. Based upon the results of the drilling program, we recommend that this structure be designed to “Site Class E”, with respect to Table 4.1.8.4.A of the OBC-2012, and subject to the limitations of the code.

6.7 Lateral Earth Pressures

The following preliminary lateral earth pressure parameters are provided to assist Contractors and Designers with the design of both permanent basement walls and temporary Engineered Shoring systems, if used. Designers will need to review if hydrostatic pressures are to be included in the earth pressure calculations based on the permanent drainage designs. If a fully waterproof ‘bath-tub design without perimeter drainage is being used, then hydrostatic pressures will need to be included in the design.

6.7.1 Static Conditions

The following Rankine earth pressure coefficients are being provided to assist Designers.

Table 6-3: Recommended Lateral Earth Pressure Coefficients for Static Conditions

Soil	Bulk Density 'Y' (kN/m^3) *	Angle of Internal Friction, φ' (degrees)	Undrained Shear Strength, Su (kPa)	Rankin Earth Pressure Coefficients**		
				K_a	K_o	K_p
Native stiff to firm silty clay	16.3	28	55 to 45	0.36	0.53	2.77
New Compacted Granular Backfill OPSS "Granular B, Type II"	22	32	0	0.31	0.47	3.25

** Only the bulk unit weight is being presented, Designers will need to assess whether bulk, saturated, and/or submerged unit weights should be used based on their design conditions.*

***Assumes level/flat backfill surface. If Engineered Shoring is used, then Designers should refer to CFEM-2006 for design assistance and the Geotechnical Engineer should be retained to perform shoring design review.*

For yielding retaining walls, the active earth pressure coefficients, K_a, is recommended to be used. For non-yielding permanent walls, such as basement walls, the at-rest, K_o, is recommended to be used for design. 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.

6.7.2 Dynamic Conditions

Below grade walls subjected to lateral forces due to seismic forces can be designed using the pseudo-static approach using the Mononobe-Okabe equations, shown in Section 24.9 of CFEM-2006. In these formulas, there are both geotechnical and geometric components.

The total active thrust under seismic loading (P_{ae}) is recommended to be expressed as follows:

$$P_{ae} = ½ K_{ae}γ H^2 x (1- k_v)$$

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 x k_h to 1/3 k_h is considered but a value closer to 2/3 x 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 OBC-2012 PGA for the Site is 0.43g at Site Class E, where g is the acceleration due to gravity, and the probability of exceedance per annum is 0.000404. This value was determined using the National Building Code of Canada 2015 (NBCC-2015) Seismic Hazard Calculation document and can be found attached in Appendix E.

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

$$P_{pe} = ½\ K_{pe}γ\ H^2 \ x\ (1- k_v)$$

Where:

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

The above equation 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 (0.3 x H) above the base and the seismic force is assumed to act near (0.6 x H) 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 equation:

$$h = [(0.33HxP_a) + (0.6H\ x\ P_e)]/\ P_{ae}$$

The following soil parameters are presented to assist Designers in designing retaining walls for this Site under seismic conditions using the pseudo-static approach.

Table 6-4: Recommended Lateral Earth Pressure Coefficients under Dynamic Conditions

Soil	Bulk Density 'Y' (kN/m³) *	Angle of Internal Friction, φ' (degrees)	Undrained Shear Strength, Su (kPa)	Mononobe Okabe Earth Pressure Coefficients**	
				K_{ae}	K_{pe}
Native stiff to firm silty clay	16.9	27	55-45	1.70	2.18
New Compacted Granular Backfill OPSS "Granular B, Type II"	22	32	0	1.35	2.07

** Only the bulk unit weight is being presented, Designers will need to assess whether bulk, saturated, and/or submerged unit weights should be used based on their design conditions.*
***Assumes level/flat backfill surface. If Engineered Shoring is used, then Designers should refer to CFEM-2006 for design assistance and the Geotechnical Engineer should be retained to perform shoring design review.*

6.8 Floor Slabs

If the structure is founded on piled foundations, then the floor slab will need to be designed as a fully structural slab supported on grade beams between the pile caps. A capillary moisture barrier consisting of a layer of either 19 mm clear stone or an OPSS 1010 "Granular A" at least 200 mm thick should underlie the slab. This layer should be compacted to 100 percent of its SPMDD and placed on approved subgrade surfaces. If the structure is founded on a raft, then the raft for the basement slab will need to be appropriately waterproofed.

If floor coverings are to be used, vapour barriers are also recommended to be incorporated beneath the slab. Floor toppings may be impacted by curing and moisture conditions of the concrete. Floor finish manufacturer's specifications and requirements should be consulted, and procedures outlined in the specifications should be followed. The placement of construction and control joints in the concrete should be in accordance with generally accepted practice.

6.9 Corrosion Potential of Soils

Analytical testing was carried out on 1 soil sample collected from borehole BH23-04 to determine corrosion potential of the subsurface soils. The selected soil sample was tested for pH, resistivity, chlorides, sulphides, sulphates and redox potential. The test results are summarized in the following table.

Table 6-5: **Corrosion Parameter Results**

Parameter	Tested Value
	BH23-04, SS2
pH	7.30
Chloroide (ug/g)	<10
Sulphate (ug/g)	<10
Resistivity (Ohm-cm)	5210
Redox Potential (mV)	439
Sulphides (%)	<0.04

The American Water Works Association (AWWA) publication 'Polyethylene Encasement for Ductile-Iron Pipe Systems' ANSI/AWWA C105/A21.5-10 dated October 1, 2010 assigns points based on the results of the above tests. A soil that has a total point score of 10 or more is considered to be potentially corrosive to ductile iron pipe. Based on the results obtained for the sample submitted, the Site soils are not considered to be severely corrosive to ductile iron pipe.

The analytical results of the soil samples were compared with applicable Canadian Standards Association (CSA) A23.1-04 and are given in Table 6-6 below.

Table 6-6: **Additional Requirement for Concrete Subjected to Sulphate Attack**

Class of Exposure	Degree of Exposure	Water Soluble Sulphate in Soil Sample (%)	Cementing Material to be Used
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.20 - 2.0	HS or HSb
S-3	Moderate	0.10 - 0.20	MS, MSb, LH, HS, or HSb

The chemical sulphate content analyses for the selected soil sample tested indicate a sulphate concentration of less than 10 ug/g (0.01%) in soil, as shown in Table 6-1, indicating the soil is not a risk for sulphate attack on concrete material.

6.10 Waterproofing and Permanent Drainage

The building basement should be designed as a fully waterproof ‘bath-tub’ design (without external perimeter drains) to avoid potential adverse impacts due to moisture movements in the immediate areas around the proposed building footprint.

Full water proofing membranes such as a WR Meadows Mel-ROL PRECON or equivalent type product for walls and under-slab will be required. These types of membranes adhere to the concrete and provide a waterproof seal between the membrane and poured concrete. Their installation would require that excavations be planned large enough for safe worker accesses on the exterior of the foundation wall to allow installation. Water stops should be installed at cold joints in the foundation walls and floor-wall joint.

6.11 Backfill

All new Fill soils that underlie floor slabs, footings, in building interiors, or other structural applications are considered as Engineered Fill and must be treated as follows:

6.11.1 Engineered Fill

All new Fill soils that underlie floor slabs, footing, or other structural applications is considered as Engineered Fill. For this Project, Engineered Fill may be required to raise the grade between the approved silty clay subgrade and foundation level or floor slabs. Engineered Fill must meet the strict requirements as shown below:

- The proposed material must be tested for grain size and Proctor and reviewed and approved by the Geotechnical Engineer before being considered as Engineered Fill. Typically, a crushed well-graded material such as an OPSS 1010 “Granular A” or “Granular B Type II” type material is suitable. However, other suitable granular materials may be proposed and considered depending on the Site-specific conditions.
- Prior to placing any Engineered Fill, all unsuitable FILL materials must be removed, and the subgrade approved by the Engineer. Any deficient areas should be repaired prior to placement.

- Engineered Fill should be placed in maximum loose lifts of 300 mm and adequately compacted to achieve 100% of its Standard Proctor Maximum Dry Density (SPMDD). Engineered Fill must have full-time compaction testing by geotechnical personnel; and
- At a minimum, the Engineered Fill beneath foundations should extend laterally a distance of 0.3 m beyond the edge of the footings and then be sloped downward and outward at 1H:1V slope. Designers and contractors are cautioned that the resultant excavation can be quite large if a significant thickness of Engineered Fill is required.

6.11.2 Exterior Foundation Wall Backfill

The backfill placed against exterior foundations should be a free draining granular material meeting the grading requirements of an OPSS 1010 “Granular B, Type I” or “Granular B, Type II”. Exterior foundation backfill should be placed and compacted as outlined below:

- Backfill should not be placed in a frozen condition, or place on a frozen subgrade.
- Backfill should be placed and compacted in maximum loose lift thickness compatible with the selected construction equipment, but not thicker than 0.3 m.
- In landscaped areas the upper 0.3 m of backfill below landscape details should be a low permeable soil to reduce surface water infiltration.
- Backfill should be placed uniformly on both sides of the foundation walls to avoid build-up of unbalanced lateral pressures, or alternatively wait until basement wall are tied together with the floor above before backfilling the exterior foundation wall.
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to achieve 98 % percent of its SPMDD.
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95 % of its SPMDD.
- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall.
- Entrance slabs should be founded on frost walls or alternatively have insulation details developed to prevent frost heaving at the building entrances; and
- In areas where the building backfill underlies a pavement, sidewalk, or other hard landscaping, the excavation should have a frost taper incorporated to prevent differential heaving around the building.

6.12 Underground Utilities

The recommendations within this section are intended to be a supplement to, and not a replacement of the most recent local municipal requirements.

6.12.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials:

- Bedding for buried utilities should consist of an OPSS 1010 "Granular A" material and placed in accordance with municipal requirements, assuming the subgrade soils are not allowed to become disturbed.
- The use of clear stone is not recommended for use as pipe bedding. The voids in the stone may result in a low gradient water flow and infiltration of fines from the surrounding soils and cover materials, causing settlement and loss of support to pipes and structures.
- The cover material should be a service sand material or an OPSS 1010 "Granular A". The dimensions should comply with pertinent specification section.
- The bedding, springline, and cover should be compacted to at least 95% of its SPMDD; and
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

6.12.2 Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches underlying pavement areas, the backfill should be placed and compacted in uniform lift thickness compatible with the selected compaction equipment and not thicker than 300 mm. Each lift should be compacted to a minimum of 98% of its SPMDD.
- The backfill placed in the upper 0.3 m below the pavement subgrade elevation should be compacted to a minimum of 100% of its SPMDD.
- Excavation backfill should attempt to match texture of the existing adjacent soils. If imported materials are used, side slopes with frost tapers are recommended. Frost tapers should be a back-slope of 10H:1V through the frost zone, (i.e.,1.8 m from finished grade).
- During backfilling, care should be taken to ensure the backfill proceeds in equal stages simultaneously on both sides of the pipe; and
- No frozen material should be used as backfill; neither should the trench base be allowed to freeze.

The quality and workmanship in the construction is as important as the compaction standards themselves. It is imperative that the guidelines for the compaction be followed for the full depth of the trench to achieve satisfactory performance.

6.12.3 Clay Seals

Clay seals should be incorporated into the design of the any utility trenches. If clay seals are not used, then there is the potential for the trench to act as a drain causing additional consolidation settlement of the Site. The location of the clay seals should be at a frequency prescribed by the Civil Engineer and at the property lines.

Ontario Provincial Standard Drawing (OPSD) 1205 and OPSD 802.095 are referred to both the Designers and Contractor for guidance on clay seals. Acceptable imported clay material may be used for the construction of the clay seals.

6.13 Recommended Asphalt Pavement

All existing asphalt pavement and granular courses should be excavated down to the proposed new subgrade level. The final subgrade should be proof-rolled to look for deflection, soft spots, or local anomalies. Typically, a heavy-duty steel drum roller or a loaded dump truck is sufficient for proof rolling. Proof rolling of proposed subgrades should be witnessed by geotechnical staff. Any non-performing areas should be sub-excavated and replaced with an appropriate new FillL soil. An appropriate Fill soil would be a free-draining non-frost susceptible soil similar to a “Granular B Type I” or “Granular B Type II” material.

Newly backfilled soils should attempt to match the texture of the existing adjacent soils. Localized sub-excavations should have frost tapers to avoid concentrated frost heaves across the roadway at the transition zones between sub-excavated and un-excavated subgrades.

In order to accommodate the recommended thicknesses, designers will need to review existing and proposed grades and determine where stripping or filling is necessary. Drainage of the pavement layers is important. Surface runoff should be directed to storm sewers or surface ditches where possible. The subgrade surface and each layer of the pavement section should also be provided with a suitable cross fall (approximately 3%) to prevent water from ponding on each layer. The installation of subdrains may be recommended as designs progress based on the surrounding topography and drainage conditions to assist in the long-term performance of the pavement structures. Non-woven geotextile as a separation medium may be prudent based on the observations during proof rolling.

For the proposed pavement base and subbase courses the material should consist of a "Granular A" and "Granular B Type II" material, respectively. The material should be placed in maximum loose lifts of 300 mm and compacted to 100 % of its SPMDD.

Sufficient field-testing should be carried out during construction to assess compaction of each lift of the pavement structure layers. This should be accompanied by laboratory testing of the proposed granular materials and asphalt materials.

In the case of winter work, which is not recommended, no frozen material should be used as backfill, and backfill should not be placed on frozen subgrades.

Based on the results of the field and laboratory testing, Englobe is recommending the following preliminary minimum pavement sections. It is important to note that at the time of this investigation, Englobe has not been provided with any traffic counts, or level of service requirements or equipment loadings for pavement structures. The pavement sections being provided are what we would consider to be suitable for a private development within this part of Ottawa. Table 6-7 below summarizes proposed asphalt designs for the parking lot and fire route respectively.

Table 6-7: Recommended Minimum Pavement Sections

Material	Layer Thickness
Parking Lots – Light Duty (Parking Stalls)	
Asphalt Wearing Course	50 mm
Well Graded Granular Base Course (Granular 'A')	150 mm
Well Graded Granular Sub-Base Course (Granular 'B' Type II)	300 mm
Parking Lots – Heavy Duty (Aisles and Fire Routes)	
Asphalt Wearing Course	40 mm
Asphalt Binder Course	50 mm
Well Graded Granular Base Course (Granular 'A')	150 mm
Well Graded Granular Sub-Base Course (Granular 'B' Type II)	450 mm

Annual or regular maintenance will be required to achieve maximum life expectancy. Generally, the asphalt pavement maintenance will involve periodic crack sealing and repair of local distress.

It is important to emphasize that the pavement sections described above are for the proposed end use condition, including light vehicular traffic and occasional service trucks. It may be necessary to over-design these sections if they are intended to support heavy construction equipment throughout construction.

7 Monitoring During Construction

Englobe requests to be retained once the plans and specifications are finalized to review the documents and ensure the recommendations in this report are adequately addressed.

The recommendations presented in this report are based on the assumption that an adequate level of construction monitoring by qualified geotechnical personnel during construction will be provided. Based on our understanding of the scope of the Project, an adequate level of construction monitoring is considered to be as follows:

- Review and approval of all footing subgrades by the Geotechnical Engineer.
- Proof rolling, review, and approval of subgrades below the floor slab.
- Laboratory testing and pre-approval of Fill soils that are proposed to be used.
- Full time compaction testing of Engineered Fill and part time compaction testing of exterior foundation wall backfill; and
- Periodic testing of concrete.

An important purpose of providing an adequate level of monitoring is to check that recommendations, based on data obtained at the discrete borehole locations, are relevant to other areas of the Site.

8 Closure

A description of limitations which are inherent in carrying out Site investigation studies is given in Appendix A and forms an integral part of this report.

We trust this report meets your present requirements. Should you have any questions, please do not hesitate to contact our office.

Appendix A
Limitations

LIMITATIONS OF REPORT
GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that Englobe Consulting Engineers Inc. be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the boreholes.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid. Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

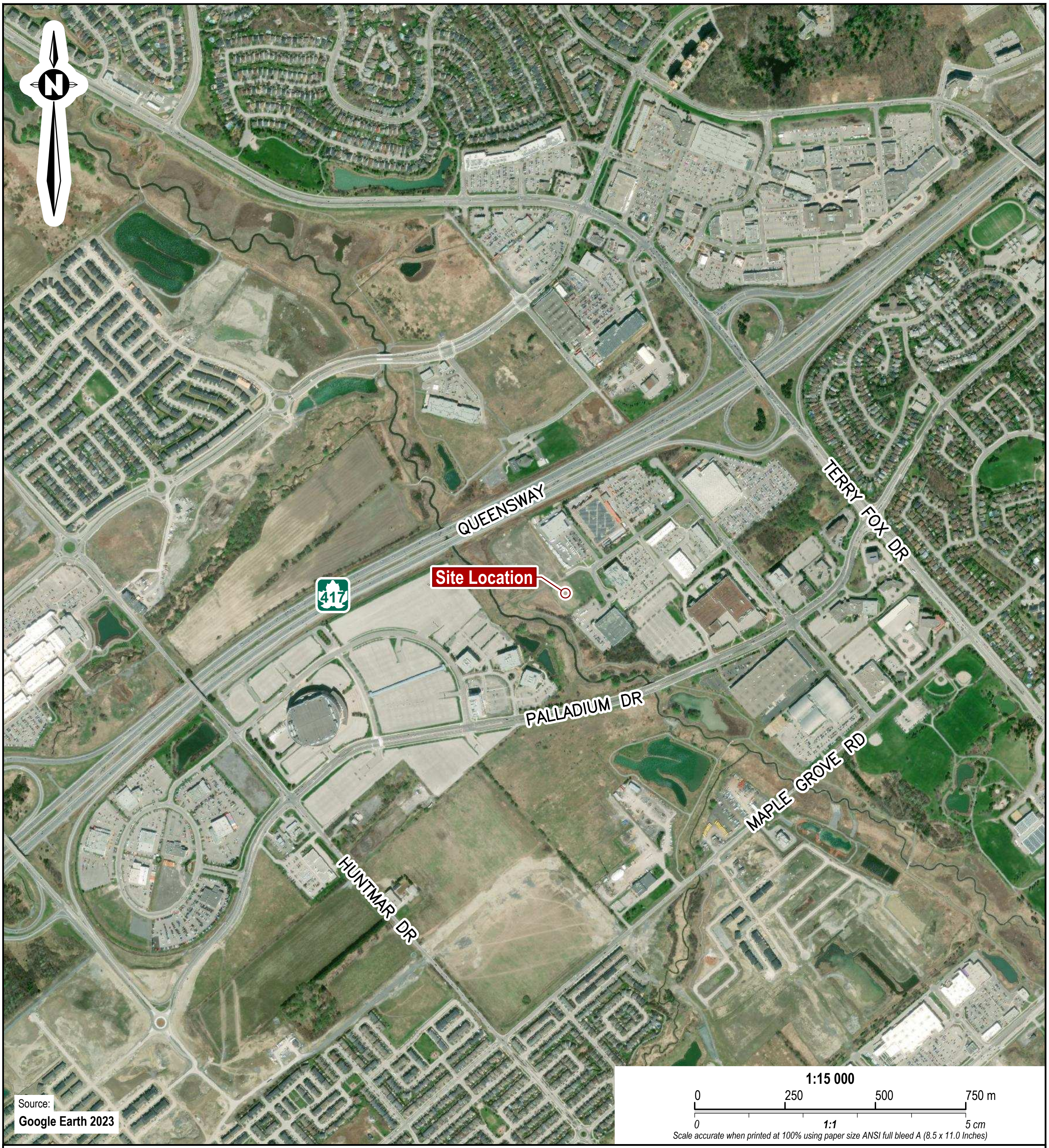
The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others and Englobe Corp. cannot warranty their accuracy. Similarly, Englobe cannot warranty the accuracy of information supplied by the Client.

Appendix B

Figure 1: Site Location Map

Figure 2: Borehole Location Plan



Source:
Google Earth 2023

1:15 000

Scale accurate when printed at 100% using paper size ANSI full bleed A (8.5 x 11.0 Inches)

Note

1. This drawing shall be read in conjunction with the associated technical report.

Revision	Date	Issue	Approval
A	2023/03/06	Preliminary	

Client
API Development Consultants Inc.

Site
40 Frank Nighbor Place, Ottawa, ON

Report Title
Preliminary Geotechnical Investigation For The Proposed 6-Storey Marriot Hotel

Drawing Title
Site Location Map

Designed By SR

Drawn By JM

Approved By

Scale As Shown

Date March 2023

Project No. 02211293.000

Figure No. 1

2713 Lancaster Road, Unit 101, Ottawa, Ontario K1B 5R6 Tel: (613) 748-1415 Fax: (613) 748-1356 Website: www.englobecorp.com/canada

Friday, February 17, 2023 @ 10:27 by Joven Mendoza

Folder: **L:\TS\CAD\Projects\Vantage Point\02211293 Frank Nighbor Pl\Geotechnical Investigation\DWGs**

Drawing: **1 Site Location.dwg**

Source:
Google Earth 2023

ENGLOBE

Note

1. This drawing shall be read in conjunction with the associated technical report.

Legend

⌖	Borehole Location
⌖	Monitoring Well Location

1:750
0 5 10 20 30 m
0 1:1 5 cm
Scale accurate when printed at 100% using paper size ANSI full bleed B (11.0 x 17.0 Inches)

Revision	Date	Issue	Approval
A	2023/03/06	Preliminary	

Client
401 Real Estate Trust C/O
API Development Consultants Inc.

Site
40 Frank Nighbor Place, Ottawa, ON

Report Title
Preliminary Geotechnical Investigation For The Proposed 6-Storey Marriot Hotel

Drawing Title
Borehole Location Plan

Designed By: SR	Scale: As Shown
Drawn By: JM	Date: March 2023
Approved By:	Project No.: 02211293.000
Figure No.: 2	

Drawing: **2 Site Plan**.dwg *Folder:* **L:\TS\CAD\Projects\Vantage Point\02211293 Frank Nighbor Pl\Geotechnical Investigation\DWGs** Monday, March 06, 2023 @ 16:08 by Joven Mendoza

Appendix C

List of Symbols and Definitions
Borehole Logs

LIST OF SYMBOLS AND DEFINITIONS FOR GEOTECHNICAL SAMPLING AND COMMON LITHOLOGIES

The following is a reference sheet for commonly used symbols and definitions within this report and in any figures or appendices, including borehole logs and test results. Symbols and definitions conform to the standard proposed by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) wherever possible. Discrepancies may exist when comparing to third-party results using the Unified Soil Classification System (USCS).

PART A – SOILS

Standard Penetration Test (SPT) 'N'

The number of blows required to drive a 50-mm (2 in) split barrel sampler 300 mm (12 in). The standard hammer has a mass of 63.5 kg (140 lbs) and is dropped vertically from a height of 760 mm (30 in). Additional information can be found in ASTM D1586-11 and in §4.5.2 of the CFEM 4th Ed.

For penetration less than 300 mm, 'N' is recorded with the penetration that was achieved.

Non-Cohesive Soils

The relative density of non-cohesive soils relates empirically to SPT 'N' as follows:

Relative Density	'N'
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	> 50

Cohesive Soils

The consistency and undrained shear strength of cohesive soils relates empirically to SPT 'N' as follows:

Consistency	Undrained Shear Strength (kPa)	'N'
Very Soft	< 12	0 – 2
Soft	12 – 25	2 – 4
Firm	25 – 50	4 – 8
Stiff	50 – 100	8 – 15
Very Stiff	100 – 200	15 – 30
Hard	> 200	> 30

PART B – ROCK

The following parameters are used to describe core recovery and to infer the quality of a rockmass.

Total Core Recovery, TCR (%)

The total length of solid drill core recovered, regardless of the quality or length of the pieces, taken as a percentage of the length of the core run.

Solid Core Recovery, SCR (%)

The total length of solid, full-diameter drill core recovered, taken as a percentage of the length of the core run.

Rock Quality Designation, RQD (%)

The sum of the lengths of solid drill core greater than 100 mm long, taken as a percentage of the length of the core run. RQD is commonly used to infer the quality of the rockmass, as follows:

Rockmass Quality	RQD (%)
Very Poor	< 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	> 90

Weathering

The terminology used to describe the degree of weathering for recovered rock core is defined as follows, as suggested by the *Geological Society of London*:

Completely weathered: All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.

Highly weathered: More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous framework or as core stone.

Moderately weathered: Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discolored rock is present ether as a continuous framework or as core stone.

Slightly weathered: Discoloration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than its fresh condition.

Fresh: No visible signs of weathering.

PART C – SAMPLING SYMBOLS

Symbol	Description
SS	Split spoon sample
TW	Thin-walled (Shelby Tube) sample
PH	Sampler advanced by hydraulic pressure
WH	Sampler advanced by static weight
SC	Soil core

PART D – IN-SITU AND LAB TESTING

SOIL NAMING CONVENTIONS

Particle sizes are described as follows:

Particle Size Descriptor		Size (mm)
Boulder		> 300
Cobble		75 – 300
Gravel	Coarse	19 – 75
	Fine	4.75 – 19
Sand	Coarse	2.0 – 4.75
	Medium	0.425 – 2.0
	Fine	0.075 – 0.425
Silt		0.002 – 0.075
Clay		< 0.002

The principle constituent of a soil is written in uppercase. The minor constituents of a soil are written according to the following convention:

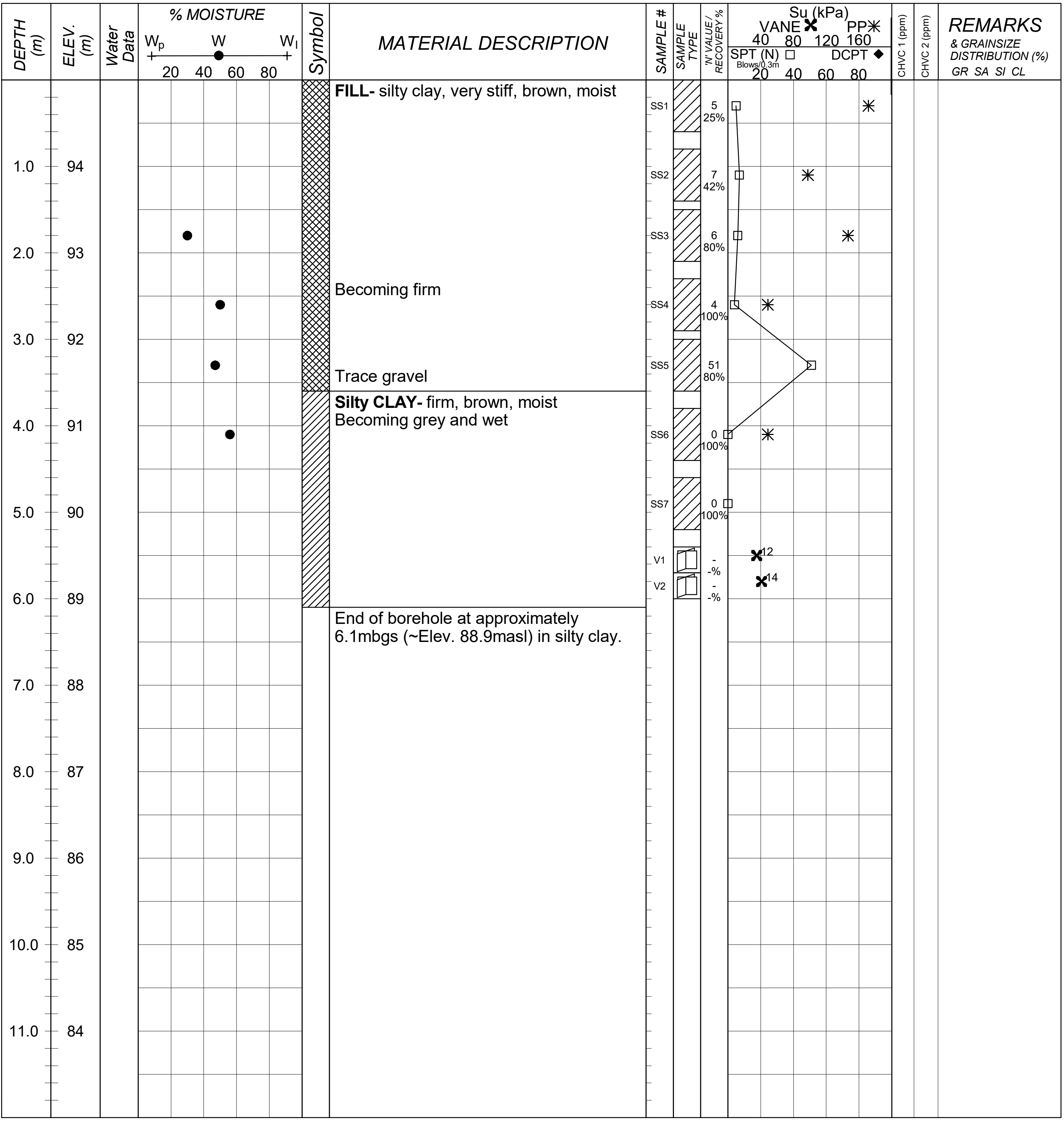
Descriptive Term	Proportion of Soil (%)
Trace	1 – 10
Some	10 – 20
(ey) or (y)	20 – 35
And	35 – 50

Eg.: A soil comprising 65% Silt, 21% Sand and 14% Clay would be described as a: Sandy SILT, Some Clay

LOG OF BOREHOLE BH23-01

DST REF. No.: **02211293**
CLIENT: **401 Real Estate Trust**
PROJECT: **Proposed 6-Storey Marriot Hotel**
LOCATION: **40 Frank Nighbor Drive, Ottawa, ON**
SURFACE ELEV.: **95.00 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **2/23/2023**
COMPLETION DATE: **2/23/2023**
COORDINATES: **428006** m N, **5016692** m E



FILL- silty clay, very stiff, brown, moist
Becoming firm
Trace gravel

Silty CLAY- firm, brown, moist
Becoming grey and wet

End of borehole at approximately 6.1mbgs (~Elev. 88.9masl) in silty clay.

SAMPLE #	'N' VALUE / RECOVERY %
SS1	5 25%
SS2	7 42%
SS3	6 80%
SS4	4 100%
SS5	51 80%
SS6	0 100%
SS7	0 100%
V1	- -%
V2	- -%

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Web: www.englobecorp.com

SAMPLE TYPE LEGEND

☰	Auger Sample	▩	Rock Core
▨	Split Spoon Sample	▥	Core Sample
▥	Bulk Sample	◫	Shelby Tube

WELL LEGEND

■	Bentonite
▒	Sand
▦	Screen

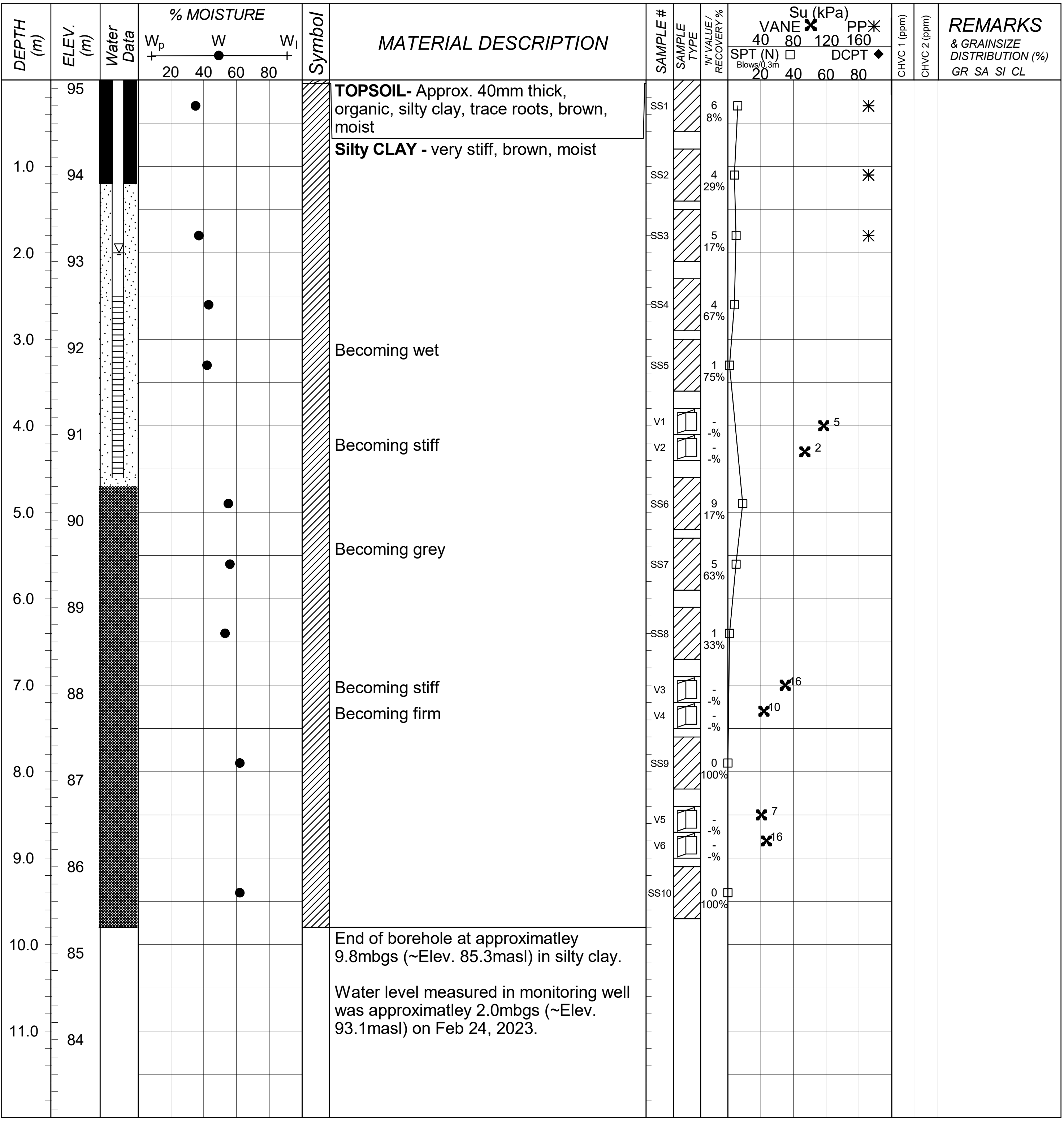
$\times^3$ Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 1

LOG OF BOREHOLE MW23-02

DST REF. No.: **02211293**
CLIENT: **401 Real Estate Trust**
PROJECT: **Proposed 6-Storey Marriot Hotel**
LOCATION: **40 Frank Nighbor Drive, Ottawa, ON**
SURFACE ELEV.: **95.10 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger and Casings**
START DATE: **2/21/2023**
COMPLETION DATE: **2/21/2023**
COORDINATES: **428069** m N, **5016723** m E



TOPSOIL- Approx. 40mm thick, organic, silty clay, trace roots, brown, moist

Silty CLAY - very stiff, brown, moist

Becoming wet

Becoming stiff

Becoming grey

Becoming stiff

Becoming firm

End of borehole at approximatley 9.8mbgs (~Elev. 85.3masl) in silty clay.

Water level measured in monitoring well was approximatley 2.0mbgs (~Elev. 93.1masl) on Feb 24, 2023.

SS1	6	8%
SS2	4	29%
SS3	5	17%
SS4	4	67%
SS5	1	75%
V1	-	-%
V2	-	-%
SS6	9	17%
SS7	5	63%
SS8	1	33%
V3	-	-%
V4	-	-%
SS9	0	100%
V5	-	-%
V6	-	-%
SS10	0	100%

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SAMPLE TYPE LEGEND

☰ Auger Sample	▩ Rock Core
▨ Split Spoon Sample	▥ Core Sample
▥ Bulk Sample	◫ Shelby Tube

WELL LEGEND

- ■ Bentonite
- ▒ Sand
- ▦ Screen

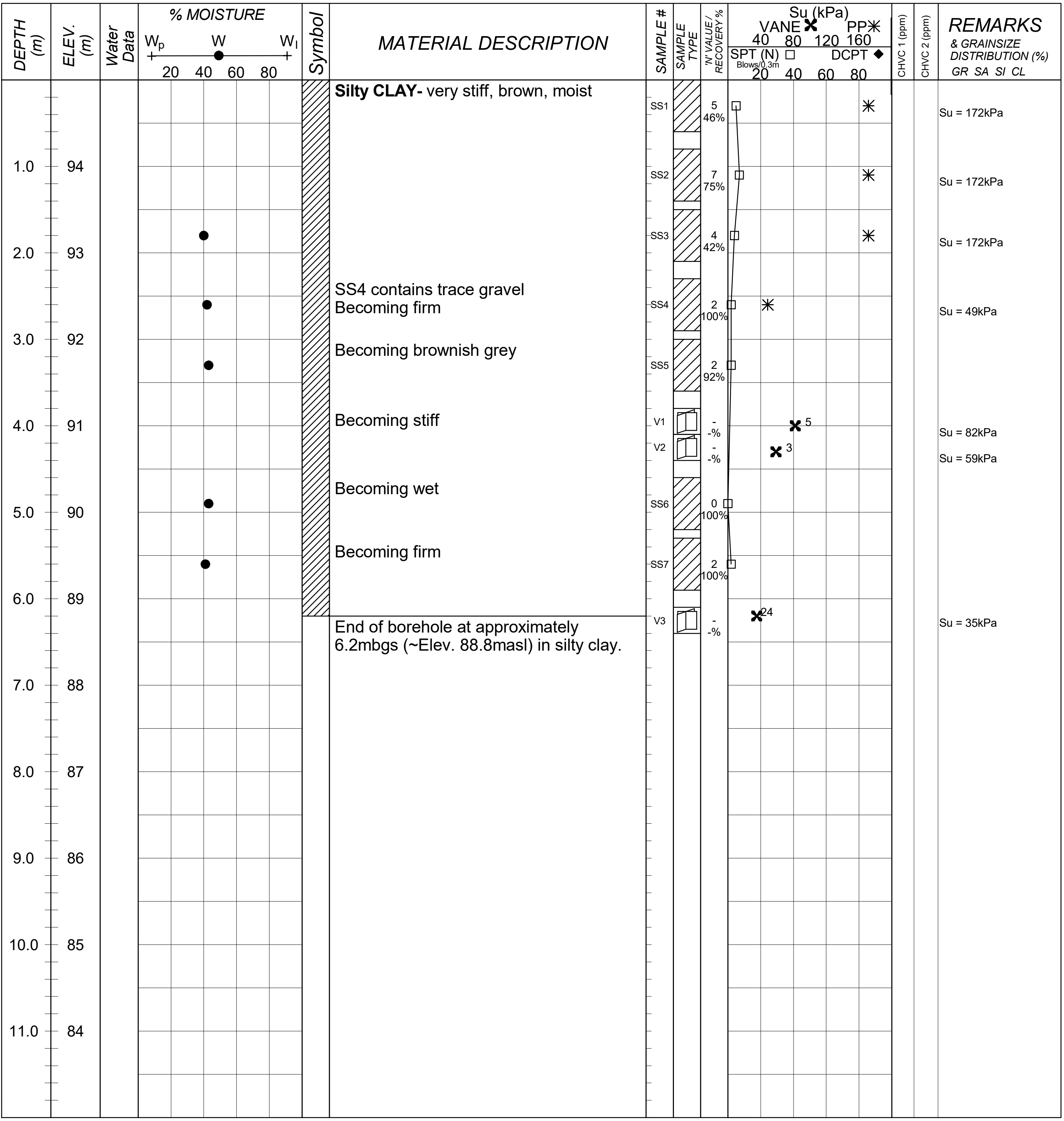
✖^{3} Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 5

LOG OF BOREHOLE BH23-03

DST REF. No.: **02211293**
CLIENT: **401 Real Estate Trust**
PROJECT: **Proposed 6-Storey Marriot Hotel**
LOCATION: **40 Frank Nighbor Drive, Ottawa, ON**
SURFACE ELEV.: **95.00 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **2/21/2023**
COMPLETION DATE: **2/21/2023**
COORDINATES: **428079** m N, **5016708** m E



Silty CLAY- very stiff, brown, moist

SS4 contains trace gravel
Becoming firm

Becoming brownish grey

Becoming stiff

Becoming wet

Becoming firm

End of borehole at approximately 6.2mbgs (~Elev. 88.8masl) in silty clay.

SS1	5	46%	Su = 172kPa
SS2	7	75%	Su = 172kPa
SS3	4	42%	Su = 172kPa
SS4	2	100%	Su = 49kPa
SS5	2	92%	
V1	-	-%	Su = 82kPa
V2	-	-%	Su = 59kPa
SS6	0	100%	
SS7	2	100%	
V3	-	-%	Su = 35kPa

BOREHOLE (THUNDER BAY) 02211293 40 FRANK NIGHBOR PLACE.GPJ DATA TEMPLATE.GDT 4/19/23

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SAMPLE TYPE LEGEND

▤ Auger Sample	▩ Rock Core
▨ Split Spoon Sample	▥ Core Sample
▥ Bulk Sample	▯ Shelby Tube

WELL LEGEND

■ Bentonite
▦ Sand
▦ Screen

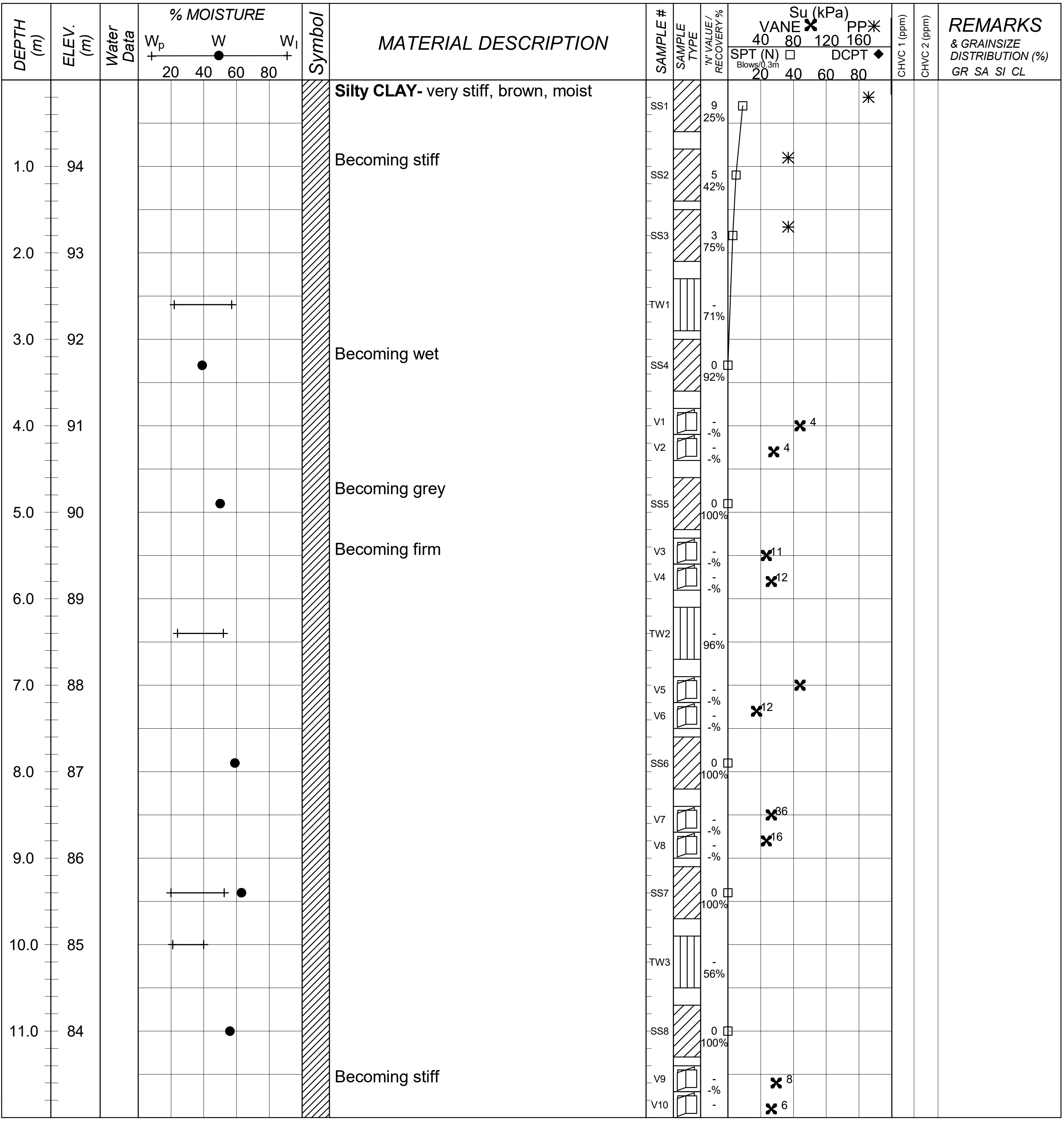
✖³ Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 2

LOG OF BOREHOLE BH23-04

DST REF. No.: **02211293**
CLIENT: **401 Real Estate Trust**
PROJECT: **Proposed 6-Storey Marriot Hotel**
LOCATION: **40 Frank Nighbor Drive, Ottawa, ON**
SURFACE ELEV.: **95.00 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger and Casings**
START DATE: **2/23/2023**
COMPLETION DATE: **2/23/2023**
COORDINATES: **428013** m N, **5016678** m E



Silty CLAY- very stiff, brown, moist

Becoming stiff

Becoming wet

Becoming grey

Becoming firm

Becoming stiff

SS1	9	25%
SS2	5	42%
SS3	3	75%
TW1	-	71%
SS4	0	92%
V1	-	-%
V2	-	-%
SS5	0	100%
V3	-	-%
V4	-	-%
TW2	-	96%
V5	-	-%
V6	-	-%
SS6	0	100%
V7	-	-%
V8	-	-%
SS7	0	100%
TW3	-	56%
SS8	0	100%
V9	-	-%
V10	-	

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SAMPLE TYPE LEGEND

☰ Auger Sample	▩ Rock Core
▨ Split Spoon Sample	▥ Core Sample
▥ Bulk Sample	◫ Shelby Tube

WELL LEGEND

- ■ Bentonite
- ⁘ Sand
- ▦ Screen

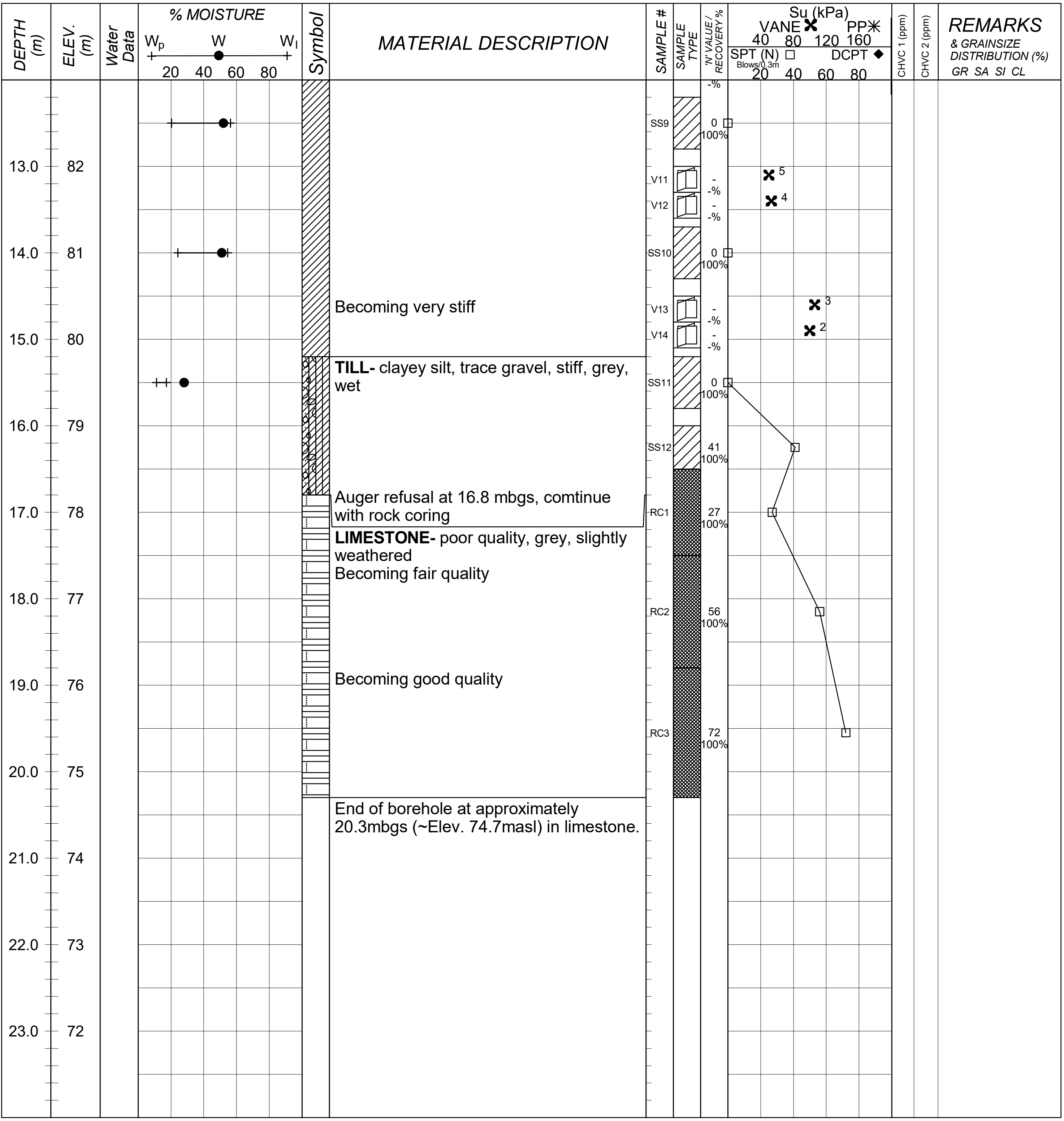
$\times^3$ Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 3

LOG OF BOREHOLE BH23-04

DST REF. No.: **02211293**
CLIENT: **401 Real Estate Trust**
PROJECT: **Proposed 6-Storey Marriot Hotel**
LOCATION: **40 Frank Nighbor Drive, Ottawa, ON**
SURFACE ELEV.: **95.00 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hollow Stem Auger and Casings**
START DATE: **2/23/2023**
COMPLETION DATE: **2/23/2023**
COORDINATES: **428013** m N, **5016678** m E



Becoming very stiff

TILL- clayey silt, trace gravel, stiff, grey, wet

Auger refusal at 16.8 mbgs, comtinue with rock coring

LIMESTONE- poor quality, grey, slightly weathered
Becoming fair quality

Becoming good quality

End of borehole at approximately 20.3mbgs (~Elev. 74.7masl) in limestone.

SAMPLE #	'N' VALUE / RECOVERY %
	-%
SS9	0 100%
V11	- -%
V12	- -%
SS10	0 100%
V13	- -%
V14	- -%
SS11	0 100%
SS12	41 100%
RC1	27 100%
RC2	56 100%
RC3	72 100%

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SAMPLE TYPE LEGEND
- ▤ Auger Sample
- ▨ Split Spoon Sample
- ▥ Bulk Sample
- ▩ Rock Core
- ▥ Core Sample
- ◫ Shelby Tube

WELL LEGEND
- ■ Bentonite
- ▒ Sand
- ▦ Screen

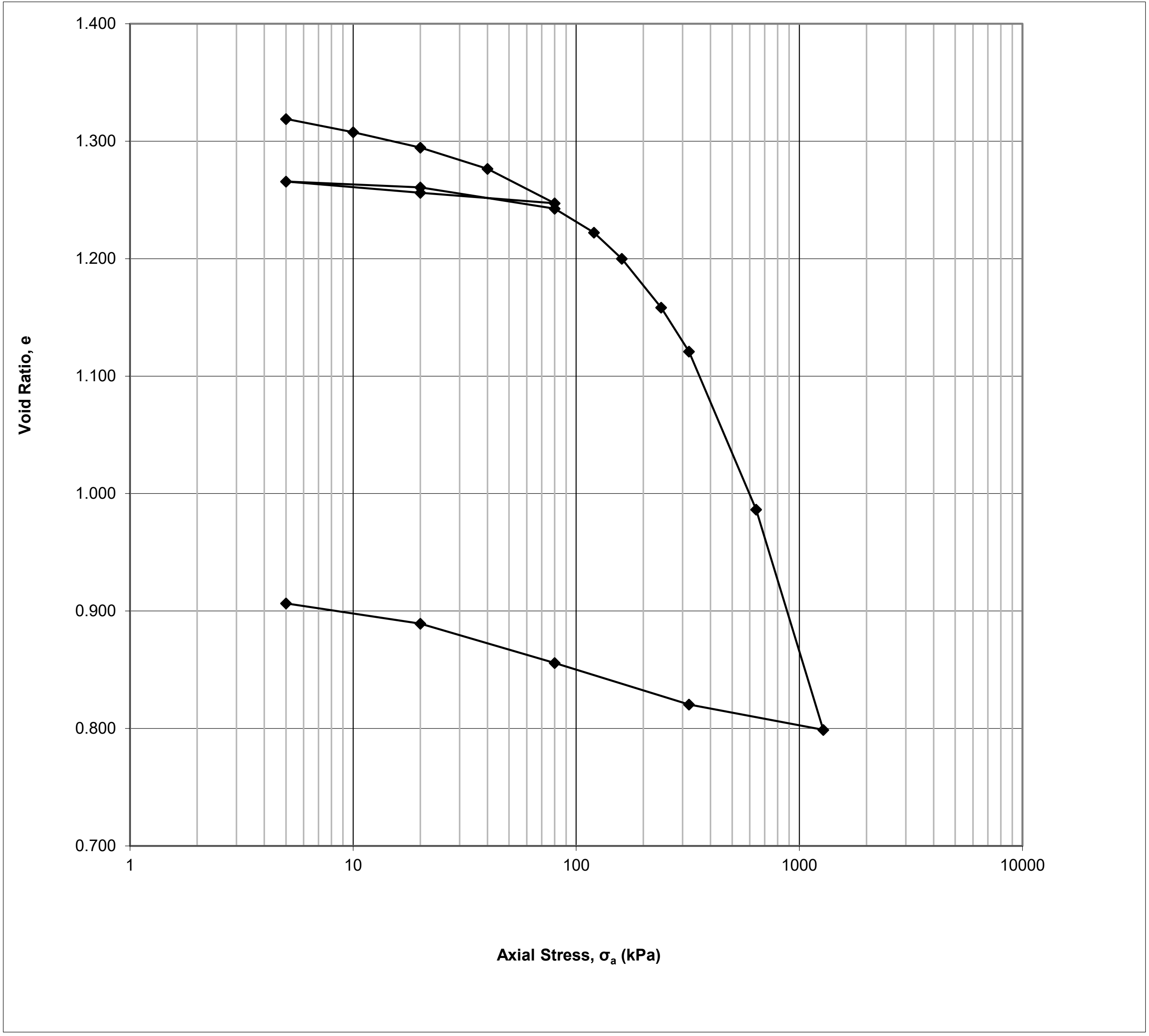
✖^{3} Numbers refers to Sensitivity
PP: Pocket Penetrometer
CHVC: Combustable Headspace Vapor Concentration
NFP: No Further Penetration

ENCLOSURE 4

Appendix D

Laboratory Test Results

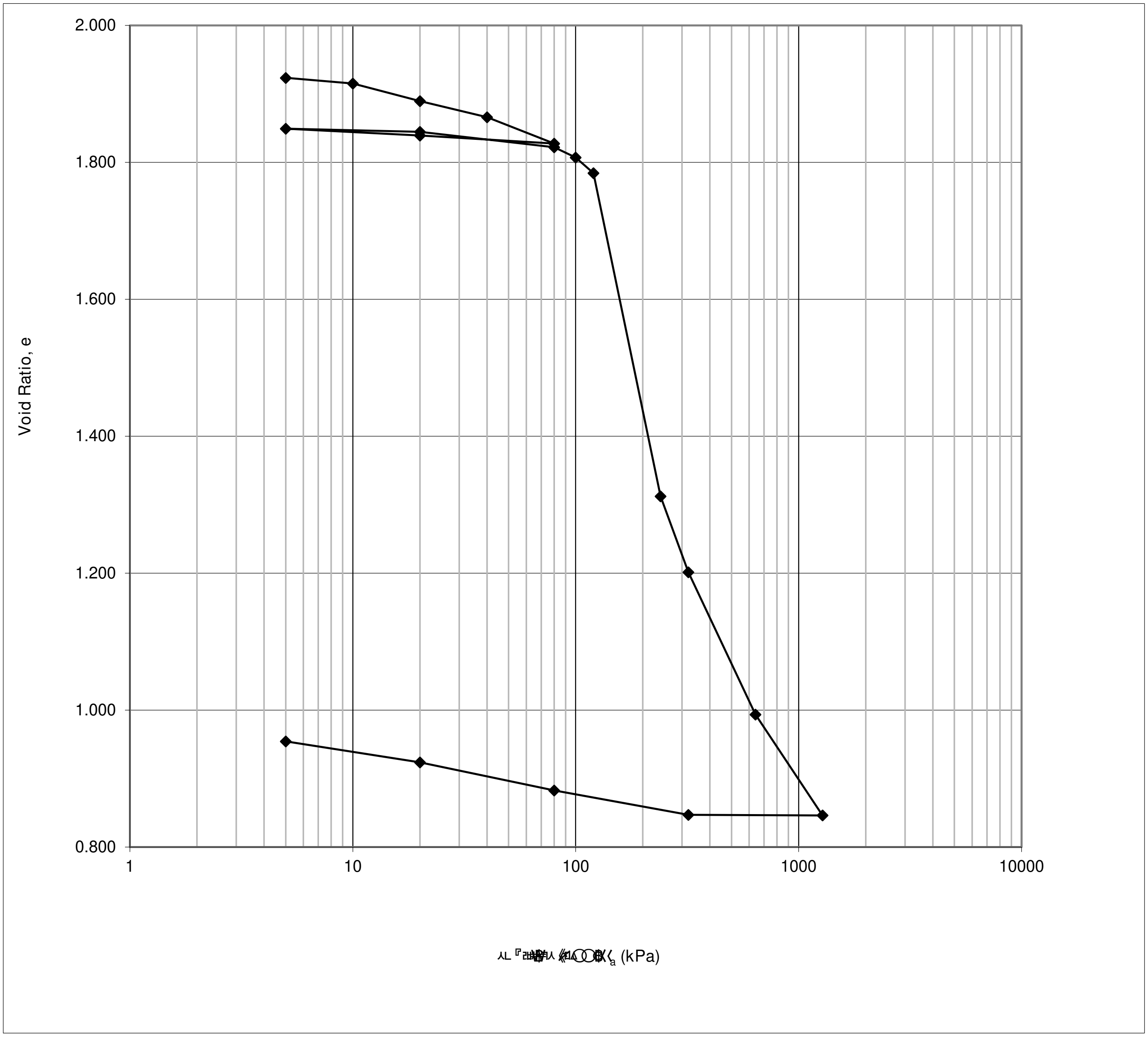
Project	Englove Corp.
Project No.	122411080
Borehole No.	BH23-04
Sample No.	TW1
Sample Depth	7.5-9.5 ft.



One-Dimensional Consolidation Properties
of Soils Using Incremental Loading

ASTM D2435/D2435M - 11(2020)

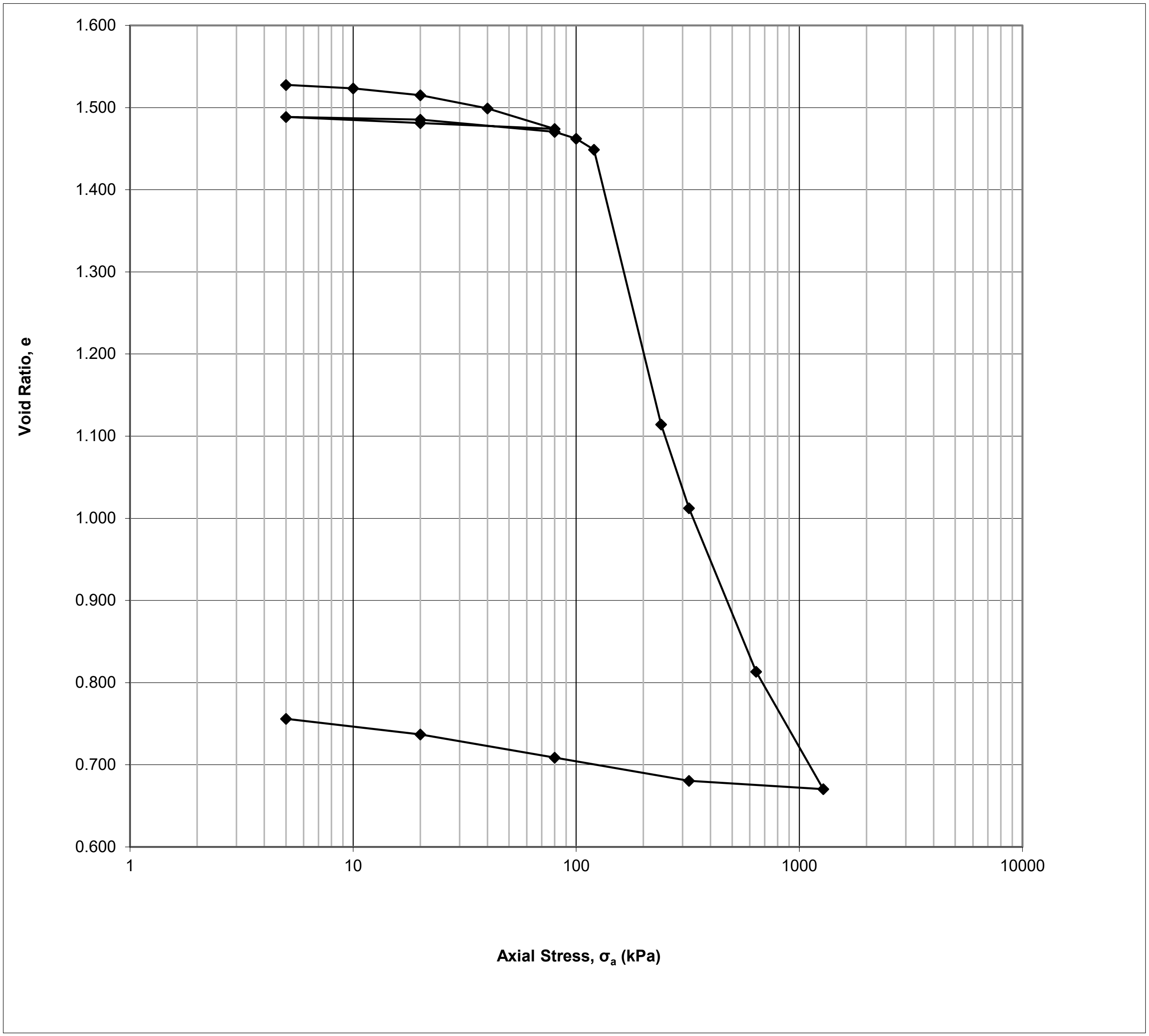
Project	EnGlobe Corp. File # 02211293.000
Project No.	122411080
Borehole No.	BH23-04
Sample No.	TW2
Sample Depth	20-22 ft.



One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D2435/D2435M - 11(2020)

Project	**EnGlobe Corp. File # 02211293.000**
Project No.	**122410864**
Borehole No.	**BH23-4**
Sample No.	**TW3**
Sample Depth	**32-34 ft.**



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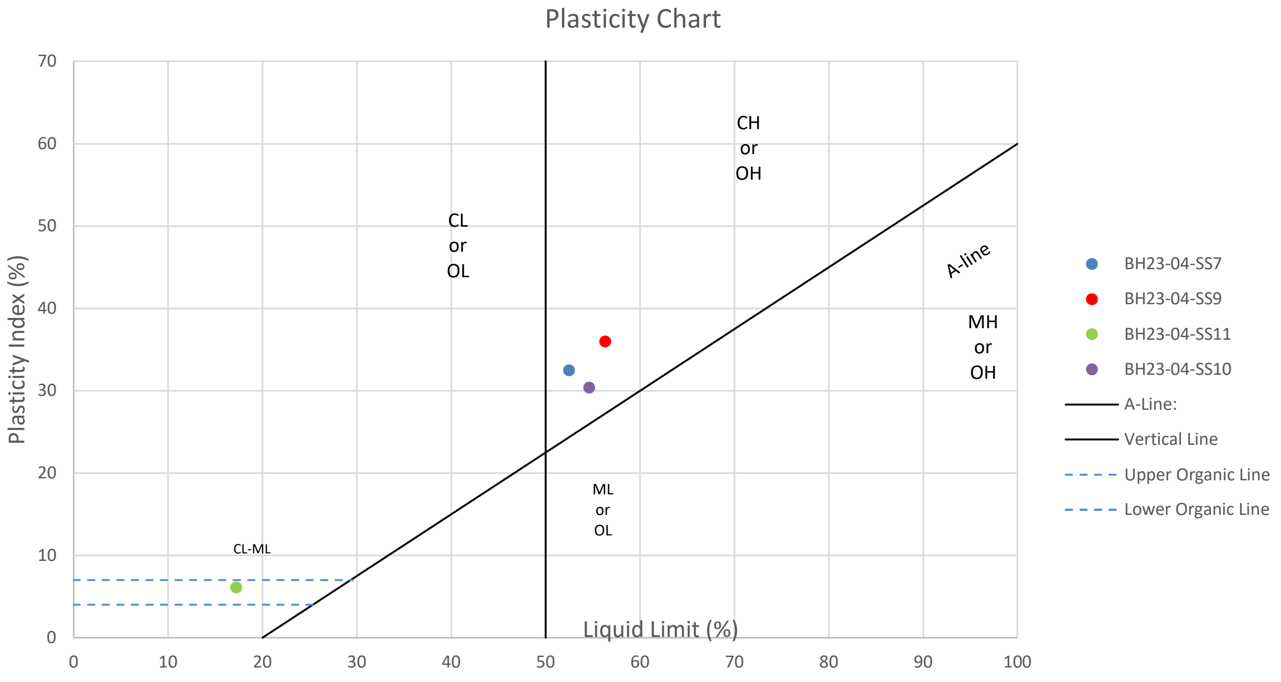
Ottawa , ON K1B 5R6
T: 1.877.300.4800
www.englobecorp.com
ottawalab@englobecorp.com

Atterberg Limits Test Results

Project:	46 Frank Neighbor Place	**Client:**	API Consultants Inc
Location:	Kanata, Ottawa	**Date:**	17/3/2023
Project No:	02211293.000		

Sample ID	Depth (m)	Soil Description	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
BH23-04-SS7	9.5	Clay of High Plasticity (CH)	62.8	52.5	20.0	32.5
BH23-04-SS9	12.5	Clay of High Plasticity (CH)	51.5	56.3	20.3	36.0
BH23-04-SS10	13.7	Clay of High Plasticity (CH)	51.3	54.6	24.2	30.4
BH23-04-SS11	15.5	Clay of Low Plasticity (CL)	27.6	17.2	11.1	6.1



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Certificate of Analysis

Englobe Corp. (Ottawa)

2713 Lancaster Road, Unit 101
Ottawa, ON K1B 5R6
Attn: Shanti Ratmono

Client PO:
Project: 02211923
Custody: 138077

Report Date: 28-Feb-2023
Order Date: 23-Feb-2023

Order #: 2308193

This Certificate of Analysis contains analytical data applicable to the following samples as submitted :

Paracel ID	Client ID
2308193-01	BH23-04-SS2

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	27-Feb-23	27-Feb-23
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	27-Feb-23	27-Feb-23
Resistivity	EPA 120.1 - probe, water extraction	27-Feb-23	27-Feb-23
Solids, %	CWS Tier 1 - Gravimetric	23-Feb-23	23-Feb-23

Certificate of Analysis

Client: Englobe Corp. (Ottawa)

Client PO:

Report Date: 28-Feb-2023

Order Date: 23-Feb-2023

Project Description: 02211923

Client ID:		BH23-04-SS2	-	-	-
Sample Date:		22-Feb-23 09:00	-	-	-
Sample ID:		2308193-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.30	-	-	-
Resistivity	0.10 Ohm.m	52.1	-	-	-
Anions					
Chloride	10 ug/g dry	<10	-	-	-
Sulphate	10 ug/g dry	<10	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	10	ug/g						
Sulphate	ND	10	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	24.2	10	ug/g	25.2			3.7	35	
Sulphate	28.7	10	ug/g	28.3			1.4	35	
General Inorganics									
pH	7.27	0.05	pH Units	7.30			0.4	2.3	
Resistivity	52.9	0.10	Ohm.m	52.1			1.6	20	
Physical Characteristics									
% Solids	85.8	0.1	% by Wt.	86.0			0.2	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	121	10	ug/g	25.2	96.1	82-118			
Sulphate	128	10	ug/g	28.3	99.4	80-120			

Qualifier Notes:

Login Qualifiers :

Sample - One or more parameter received past hold time - Redox Potential.

Applies to samples: BH23-04-SS2

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

PARACE
LABORATORIES LT

Paracel ID: 2308193

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Paracel Order Number
(Lab Use Only)
2308193

Chain Of Custody
(Lab Use Only)
№ 138077

Client Name: Englobe crop
Contact Name: Shant R
Address: 2713 lancaster Rd, Ottawa
Telephone:

Project Ref: 0221192 3
Quote #:
PO #:
E-mail: Shanti. Ratmono@englobecorp.com

Page _ of _

Turnaround Time
☐ 1 day ☐ 3 day
☐ 2 day ☐ Regular
Date Required:

☐ REG 153/04 ☐ REG 406/19
☐ Table 1 ☐ Res/Park ☐ Med/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other
☐ Table __
For RSC: ☐ Yes ☐ No

Other Regulation
☐ REG 558 ☐ PWQO
☐ CCME ☐ MISA
☐ SU - Sani ☐ SU - Storm
Mun:
☐ Other:

Matrix Type: **S** (Soil/Sed.) **GW** (Ground Water)
SW (Surface Water) **SS** (Storm/Sanitary Sewer)
P (Paint) **A** (Air) **O** (Other)

					Sample Taken		Required Analysis											
	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Date	Time	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	~~B (HWS)~~ Sulphide	PH	Chloride	Sulphate	Resistivity	Redox
1	BH23-04-SS2	S			Feb22/23	9:00 AM							✓	✓	✓	✓	✓	✓
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

Comments:

Method of Delivery: Drop Box

Relinquished By (Sign): WJ
Relinquished By (Print): Wajiha Javeed
Date/Time: Feb22/23 @ 1830

Received By Driver/Depot:
Date/Time:
Temperature: °C

Received at Lab:
Date/Time: Feb.23 2023 800
Temperature: 18.3 °C

Verified By:
Date/Time: Feb23 2023 817
pH Verified: ☐ By:

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Subcontracted Analysis

Englobe Corp. (Ottawa)
2713 Lancaster Road, Unit 101
Ottawa, ON K1B 5R6
Attn: Shanti Ratmono

Paracel Report No. **2308193**
Client Project(s): **02211923**
Client PO:
Reference: **Standing Offer**
CoC Number: **138077**

Order Date: 23-Feb-23
Report Date: 28-Feb-23

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor's report is attached

Paracel ID	**Client ID**	**Analysis**
2308193-01	BH23-04-SS2	Redox potential, soil Sulphide, solid

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

08-March-2023

Paracel Laboratories
Attn : Dale Robertson

300-2319 St.Laurent Blvd.
Ottawa, ON
K1G 4K6, Canada

Phone: 613-731-9577
Fax:613-731-9064

Date Rec. : 24 February 2023
LR Report: **CA12706-FEB23**
Reference: Project#: 2308193

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide (Na2CO3) %
1: Analysis Start Date		08-Mar-23
2: Analysis Start Time		07:09
3: Analysis Completed Date		08-Mar-23
4: Analysis Completed Time		09:11
5: QC - Blank		< 0.04
6: QC - STD % Recovery		119%
7: QC - DUP % RPD		ND
8: RL		0.02
9: BH23-04-SS2	22-Feb-23 09:00	< 0.04

RL - SGS Reporting Limit
ND - Not Detected

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

CERTIFICATE OF ANALYSIS

Client:	Dale Robertson	Work Order Number:	491465
Company:	Paracel Laboratories Ltd. - Ottawa	PO #:	
Address:	300-2319 St. Laurent Blvd. Ottawa, ON, K1G 4J8	Regulation:	[No Reg - Always Include Reg Report]
		Project #:	2308193
Phone/Fax:	(613) 731-9577 / (613) 731-9064	DWS #:	
Email:	drobertson@paracellabs.com	Sampled By:	
Date Order Received:	2/24/2023	Analysis Started:	3/1/2023
Arrival Temperature:	16.5 °C	Analysis Completed:	3/1/2023

WORK ORDER SUMMARY

ANALYSES WERE PERFORMED ON THE FOLLOWING SAMPLES. THE RESULTS RELATE ONLY TO THE ITEMS TESTED.

Sample Description	Lab ID	Matrix	Type	Comments	Date Collected	Time Collected
BH23-04-SS2	1853426	Soil	None		2/22/2023	9:00 AM

METHODS AND INSTRUMENTATION

THE FOLLOWING METHODS WERE USED FOR YOUR SAMPLE(S):

Method	Lab	Description	Reference
RedOx - Soil (T06)	Mississauga	Determination of RedOx Potential of Soil	Modified from APHA-2580B

REPORT COMMENTS

Sample received past hold time for Redox, proceed with analysis as per comments TJ 02/24/23

This report has been approved by:

Marc Creighton
Laboratory Director

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 491465

WORK ORDER RESULTS

Sample Description	BH23 - 04 - SS2
Sample Date	2/22/2023 9:00 AM
Lab ID	1853426

General Chemistry	Result	MDL	Units	Criteria: [No Reg - Always Include Reg Report]
RedOx (vs. S.H.E.)	439 [435]	N/A	mV	~

LEGEND

Dates: Dates are formatted as mm/dd/year throughout this report.

MDL: Method detection limit or minimum reporting limit.

[ ]: Results for laboratory replicates are shown in square brackets immediately below the associated sample result for ease of comparison.

~: In a criteria column indicates the criteria is not applicable for the parameter row.

Quality Control: All associated Quality Control data is available on request.

Field Data: Reports containing Field Parameters represent data that has been collected and provided by the client. Testmark is not responsible for the validity of this data which may be used in subsequent calculations.

Sample Condition Deviations: A noted sample condition deviation may affect the validity of the result. Results apply to the sample(s) as received.

Reproduction of Report: Report shall not be reproduced, except in full, without the approval of Testmark Laboratories Ltd.

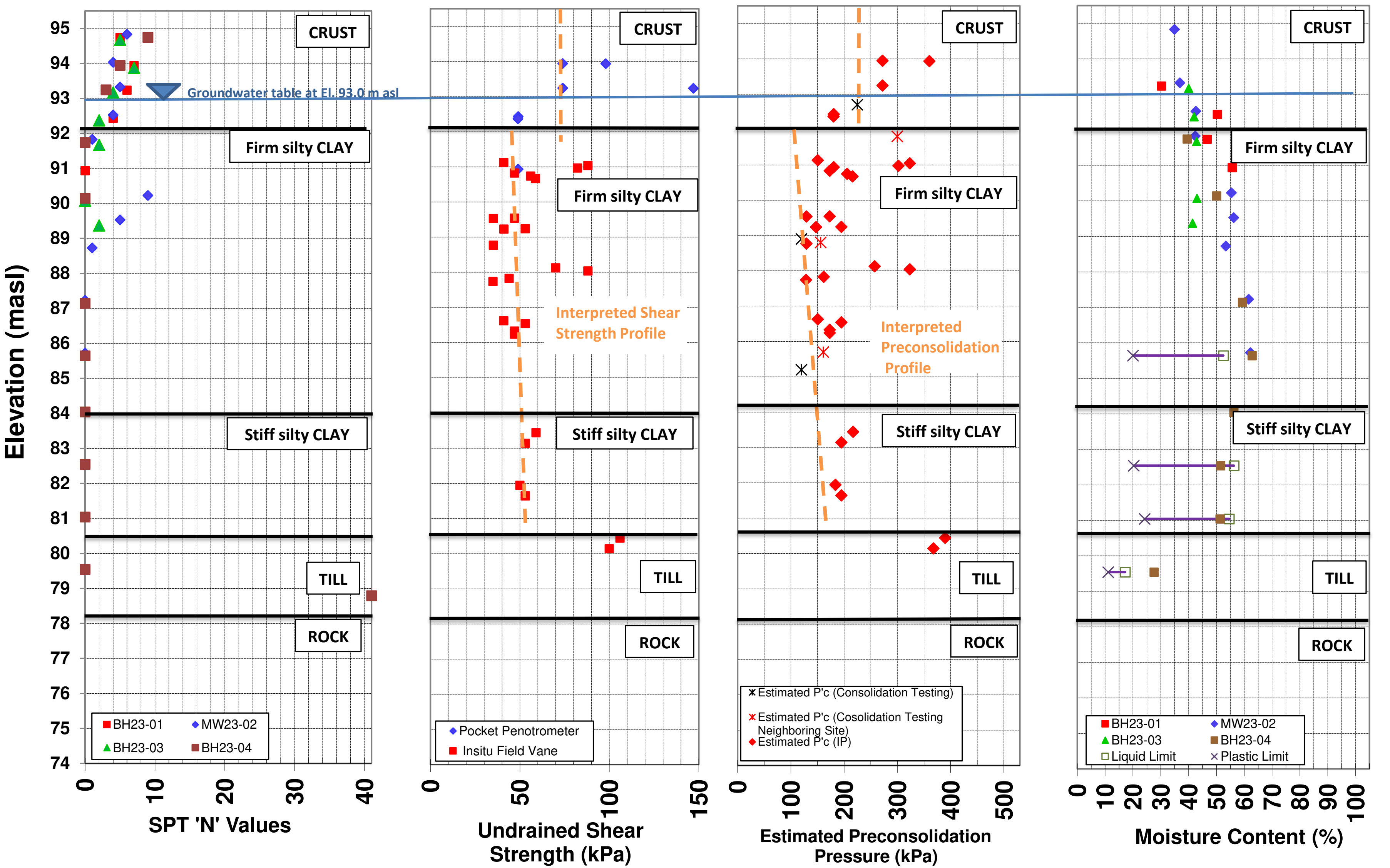
ICPMS Dustfall Insoluble: The ICPMS Dustfall Insoluble Portion method analyzes only the particulate matter from the Dustfall Sampler which is retained on the analysis filter during the Dustfall method.

Regulation Comparisons: Disclaimer: Please note that regulation criteria are provided for comparative purposes, however the onus on ensuring the validity of this comparison rests with the client.

Appendix E

Figure 3: Geotechnical Model
Rock Core Photos
2015 National Building Code Seismic Hazard Calculations

ENGLOBE



ENGLOBE

Geotechnical Model
40 Frank Nighbor Place, Kanata, ON

Figure 4

Project No. 02211293.000

Rock Core Photographs
40 Frank Nighbor Street, Kanata, ON

Project No.: 02211293.000

Rock Core No.: RC1 to RC2

Borehole: BH23-04

Date: February 2023

Rock Core No.: RC3

Borehole: BH23-04

Date: February 2023

2015 National Building Code Seismic Hazard Calculation

INFORMATION: **Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 45.300N 75.918W

2023-03-23 18:50 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.400	0.214	0.127	0.038
Sa (0.1)	0.471	0.264	0.162	0.054
Sa (0.2)	0.396	0.227	0.143	0.049
Sa (0.3)	0.302	0.175	0.112	0.040
Sa (0.5)	0.215	0.126	0.081	0.029
Sa (1.0)	0.109	0.065	0.042	0.014
Sa (2.0)	0.052	0.031	0.019	0.006
Sa (5.0)	0.014	0.008	0.004	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.254	0.144	0.089	0.029
PGV (m/s)	0.179	0.101	0.062	0.020

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

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