Geotechnical Investigation

Proposed 7-Storey Multi-Unit Apartment Building
424 Churchill Avenue North
Ottawa, ON

City of Ottawa Application No.:
D01-01-22-0011
D02-02-22-0098
D07-12-22-0152

Prepared for:
Churchill Properties Inc.
145 Select Avenue, Unit 5
Scarborough, ON
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Property and Confidentiality

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1 Introduction

Englobe Corp. (Englobe) is pleased to present the findings of our geotechnical investigation report for the proposed 7-Storey Multi-Unit Apartment Building (Project) located at 424 Churchill Avenue North in Ottawa, Ontario (Site).

Englobe, formerly DST, had previously conducted a geotechnical investigation in July 2021 and prepared a geotechnical report entitled "Preliminary Geotechnical Investigation for Proposed Multi-Storey Residential Development at 424 Churchill Avenue North, Ottawa, ON", File No. 02103035.000, dated July 2021 to evaluate the subsurface conditions at the Site of the proposed Project. At the time of the preliminary investigation, the Project was in the concept design stage and was based on a proposed 6-storey residential building with up to two levels of underground parking, founded at an approximate elevation of El. 68.3 meters above sea level (masl).

It is Englobe's understanding that the project has progressed to the detailed design phase and is now comprised of an 7-storey multi-unit apartment building with three basement levels. Based on available architectural and civil drawings, the proposed foundations will be at an approximate elevation near El 63.8 masl. Groundwater level measured during the July 2021 geotechnical report range from approximate elevations near El. 68.4 to 68.8 masl, indicating that the excavation of the new building will be approximately 4.0 meters below the measured groundwater table. Based on this understanding, Englobe recommended a hydrogeological investigation and supplementary geotechnical and environmental investigations, which was authorized by Ms. Jemmy Taing on behalf of the Client on June 5, 2023. The results of the hydrogeological investigation and supplementary environmental investigation are provided under separate cover.

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 a municipal address of 424 Churchill Avenue North in Ottawa, Ontario. The location of the Site is shown on Figure No. 1 'Site Location Map' provided in Appendix B. The Site is currently occupied with a one-storey commercial building with associated asphalt parking lot and entrances, as shown on Figure 2 'Borehole Location Plan' provided in Appendix B.

The paved area of the Site is at an approximate elevation of between El. 74.8 and 75.7 masl. The existing topography of the Site slopes downwards approximately 0.3 m to the east and south towards the adjacent streets. There is also a steep slope along the north perimeter of the Site dropping approximately 6 m from the Site down to Danforth Avenue South. The elevation along the north perimeter of the Site ranges from approximately El. 69.1 to 72.0 masl.

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

- "Topographic Plan of Survey of Lot 1 and Part of Lot 2 (South Danforth Avenue), Registered Plan 204, City of Ottawa", dated July 12, 2022, prepared by Annis, O'Sullivan, Vollebekk Ltd.
- Architectural Plan Drawings, Drawing Nos. A000 to A300 inclusive, dated January 06, 2022 to October 20, prepared by Open Plan Architects Inc.; and
- Civil Plan Drawings, Nos. C301, C401, and C601, dated October 11, 2022, prepared by LRL Associates Ltd.

The proposed building will cover an approximate area of 882.3 m^2 and will be comprised of a 7-storey multi-unit residential building with 3 basement levels. Based on the architectural and civil Site plans received, the ground floor of the building will be at an approximate elevation of El. 75.92 masl. Therefore, the B3 basement level will be approximately 10.6 meters deeper at an approximate elevation of El. 65.32 masl. The completed building will have a finished average grade of approximately El. 73.10 masl. Due to the split grade of the Site, the top two basement levels will be partially exposed along the north perimeter of the proposed building.

3 Scope of Work

Englobe's original scope of work was outlined in "Proposal for Combined Phase I/II Environmental Site Assessment and Preliminary Geotechnical Investigation", dated March 15, 2021, and was accepted by Mr. David Ho on behalf of the Client on March 19, 2021. The original scope of work for Englobe consisted of the following activities:

- Retained a subcontractor to provide both public and private underground utility clearances;
- Retained a geotechnical drilling subcontractor to drill three (3) boreholes with depths ranging from approximately 10.0 to 12.8 meters below ground surface (mbgs);
    - All boreholes were advanced through the overburden using hollow stem augers;
    - Two (2) of the boreholes were continued into the bedrock using pneumatic drilling techniques for environmental purposes;
    - One (1) borehole was continued into the bedrock using wireline diamond coring techniques.
- Installed monitoring wells in all boreholes with screens sealed into the bedrock;
- Supervised the drilling fieldwork and logged the subsoil conditions at the borehole locations based on the samples that were recovered;
- Performed geotechnical laboratory testing consisting of two unconfined compressive strength (UCS) tests on collected bedrock cores, moisture contents on all collected soil samples, and submittal of one groundwater sample for standard corrosion package laboratory analysis;

- Prepared a preliminary geotechnical investigation report based on the results of the field investigation and laboratory testing.

A supplementary geotechnical investigation was outlined in our proposal (Ref No: P2103035.000, dated June 01, 2023) and was authorized by Ms. Jemmy Taing on behalf of the Client on June 05, 2023 by means of a signed offer of services. The supplementary scope of work consisted of the following activities:

- Retain a utility subcontractor to provide both public and private underground utility clearances;
- Retain a geotechnical drilling subcontractor to drill four (4) boreholes with maximum approximate depths of up to 16.5 mbgs;
    - All boreholes were advanced through the overburden using hollow stem augers;
    - Three (3) of the boreholes were continued into the bedrock using pneumatic drilling techniques for environmental purposes;
    - One (1) borehole was advanced into the bedrock using wireline diamond coring techniques.
- Install monitoring wells in all boreholes with screens sealed into the bedrock;
- Supervise drilling fieldwork and log the subsoil conditions at the borehole locations based on the samples that were recovered;
- Perform geotechnical laboratory testing consisting of moisture contents on all collected soil samples, and submittal of one groundwater sample for standard corrosion package laboratory analysis; and
- Prepare this Geotechnical Investigation Report responding to First Round Comments from the City, focusing on:
    - Foundation waterproofing;
    - Dewatering;
    - Support of adjacent buildings; and
    - Bearing capacities at deeper founding elevations.

4 Field Investigation and Laboratory Testing

4.1 Previous Drilling Fieldwork

The drilling component of the earlier geotechnical investigation was performed on April 21 and 22, 2021. The drilling consisted of the advancement of three (3) boreholes, designated as Borehole Nos. MW21-01 through MW21-03, to approximate depths ranging from 0.0 to 12.8 meters below ground surface (mbgs). All boreholes were terminated within the bedrock underlying the overburden. The locations of the boreholes are shown on Figure No. 2, 'Borehole Location Plan' provided in Appendix B.

A geotechnical drilling subcontractor, CCC Geotechnical and Environmental Drilling Ltd., was retained to perform the drilling. All boreholes were drilled using a truck mounted drill rig. The boreholes were advanced through the overburden using hollow-stem augers. Borehole Nos. MW21-01 and MW21-03 were continued into the bedrock using pneumatic drilling methods for environmental purposes for the Phase II ESA performed in conjunction with this investigation. Borehole No. MW21-02 was advanced through the bedrock using wireline diamond coring methods. 50 mm outer diameter monitoring wells were installed in all boreholes, with screens sealed into the bedrock. The monitoring wells were backfilled with a combination of bentonite hole-plug and silica sand as necessary, and protective flush mount coverings were placed at the ground surface and sealed using asphalt cold patch.

Overburden soil samples were collected using a standard 50 mm outside diameter split-spoon sampler driven by an automatic Standard Penetration Test (SPT) hammer. The compaction of the cohesionless soils was assessed using recorded SPT N-values.

The subsurface conditions encountered in the boreholes were described by Englobe field staff based on the samples that were recovered. The recovered soil and rock core samples were labelled and submitted to Englobe’s Ottawa geotechnical laboratory for further visual review and geotechnical laboratory testing on selected soil samples. One groundwater sample was sent to an external certified environmental laboratory for standard corrosion package testing.

4.2 Current Drilling Fieldwork

The drilling component of the current geotechnical investigation was performed on July 11, 12, 19, and 20, 2023. The drilling consisted of the advancement of four (4) boreholes within the footprint of the proposed building, designated as Borehole Nos. MW23-01 through MW23-04, and were advanced to approximate depths ranging from 8.2 to 16.7 mbgs. All boreholes were terminated in the bedrock underlying the overburden. The location of the borehole is shown on the Figure No. 2, “Borehole Location Plan” provided in Appendix B.

A geotechnical drilling subcontractor, Strata Soil Sampling, was retained to perform the drilling. The borehole was drilled using a truck mounted drill rig. The boreholes were advanced through the overburden using continuous-flight hollow-stem augers. Borehole No. MW23-01 was continued into the bedrock using double-walled wireline diamond coring methods. Borehole Nos. MW23-02 through MW23-04 were continued into the bedrock using pneumatic drilling methods for environmental purposes for the Phase II ESA performed in conjunction with this investigation. 50 mm outer diameter monitoring wells were installed in all boreholes with the screen sealed into the bedrock. The monitoring wells were backfilled with a combination of bentonite hole-plug and silica sand as necessary, and protective flush mount coverings were placed at the ground surface and sealed using asphalt cold patch.

Overburden soil samples were collected in the boreholes using a standard 50 mm outside diameter split-spoon (SS) sampler driven by an automatic Standard Penetration Test (SPT) hammer. The compaction of the cohesionless soils was assessed using recorded SPT N-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 Englobe’s Ottawa geotechnical laboratory for further review and geotechnical laboratory testing on selected samples.

4.3 Geotechnical Laboratory Testing

The laboratory testing component of this investigation consisted of the determination of moisture contents on all recovered soil samples and unconfined compressive strength tests on two representative rock core samples collected. The results of the laboratory testing are presented on the Borehole Logs provided in Appendix C and as Laboratory Test Results provided in Appendix D.

In addition, one groundwater sample was collected from the Borehole No. MW21-02 and submitted to a certified environmental laboratory for standard corrosion package testing. The results of the corrosion testing are discussed in Section 6.10.

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 Asphalt/Concrete Thickness (mm)	Approx. FILL Thickness (mm)	Native Sandy Silt Depth (mbgs)	Limestone Bedrock Depth (mbgs)
MW21-01	140	320	0.5 – 1.2	
MW21-02	120	360	0.5 – 1.0	1.0 – 10.0
MW21-03	140	160	0.3 – 0.8	
MW23-01	100	200	0.3 – 0.9	0.9 – 16.7
MW23-02	25	75	-	0.1 – 9.2
MW23-03	25	300	0.3 – 0.8	0.8 – 9.2
MW23-04	-	200	-	0.2 – 8.2

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 Asphalt Pavement

Asphalt pavement was present surficially in Borehole Nos. MW21-01 through MW21-03, and MW23-01. The surficial cover at these locations consisted of asphalt pavement with approximate thicknesses ranging from 100 to 140 mm.

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 Concrete Slab

A concrete slab was present surficially in Borehole Nos. MW23-02 and MW23-03. The surficial cover at these locations consisted of a concrete slab with an approximate thickness of 25 mm.

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.

5.3 FILL Material

Cohesionless FILL material was encountered surficially in Borehole No. MW23-04 and underlying the surficial asphalt/concrete pavement at all other borehole locations, extending to approximate depths ranging from 0.3 to 0.5 mbgs. The FILL material was heterogeneous in nature and consisted of silty sand to sandy gravel, with trace to some gravel and trace to some silt.

The FILL material was brown in colour. The moisture contents of this material ranged from 5 to 8 percent. The moisture contents of the FILL material are presented on the Borehole Logs included in Appendix C.

5.4 Native Sandy Silt

A deposit of native sandy silt was encountered underlying the FILL material in Borehole Nos. MW21-01 through MW21-03, MW23-02, and MW23-04, at approximate depths ranging from 0.3 to 0.5 mbgs and extended to approximate depths ranging from 0.8 to 1.2 mbgs.

The native sandy silt deposit was described as damp and brown in colour. The natural moisture contents of this deposit ranged from 4 to 20 percent based on laboratory testing. The recorded SPT N-values for this deposit ranged from 6 to more than 50 blows per 0.3 m, indicating a loose to compact relative density.

5.5 Limestone Bedrock

Auger refusal on bedrock was encountered in all boreholes at approximate depths ranging from 0.2 mbgs to 1.4 mbgs. All boreholes were terminated within the bedrock at approximate depths ranging from 8.2 to 16.8 mbgs. Boreholes MW21-01, MW21-03, and MW23-02 through MW23-04 were continued into the bedrock using pneumatic drilling methods for environmental purposes for the Phase II ESA performed in conjunction with this investigation, therefore bedrock type and quality could not be confirmed at these locations.

Boreholes MW21-02 and MW23-04 were advanced into bedrock using HQ-size wireline diamond coring methods. The upper approximately 0.5 to 1.0 m of the bedrock consisted of highly weathered and fractured limestone. The intact portions of the bedrock consisted predominantly of grey, slightly weathered limestone becoming fresh with depth, and medium to thickly bedded. The measured Rock Quality Designation (RQD) ranged from 37 to 100 percent. The bedrock was generally in poor to good condition with areas of excellent quality. Boreholes MW21-02 and MW 23-04 were terminated within the limestone bedrock at approximate depths of 10.0 and 16.8 mbgs, respectively.

Unconfined compressive strength tests were conducted on two rock cores collected at depths of approximately 7.5 mbgs (MW21-02 RC5) and 9.1 mbgs (M21-02 RC6). The compressive strength test results are provided in the following table.

Table 5-2: Summary of Rock Core Tests

Borehole / Sample ID	Depth (m)	Unconfined Compressive Strength (MPa)
MW21-02 / RC5	7.5 - 7.6	152.7
MW21-02 / RC6	9.1 - 9.2	141.1

5.1 Groundwater

Monitoring wells were installed at all borehole locations and were sealed and screened within the bedrock at depths of between 4.6 and 16.6 mbgs. Water level readings were recorded by Englobe personnel at the monitoring well locations on April 29 and 30, 2021, and on August 14, 2023. The following table provides the details of the well construction and groundwater level readings. Additional monitoring well details and water level measurements are shown on the Borehole Logs provided in Appendix C.

Table 5-3: Summary of Monitoring Well Observations

Borehole ID	Screen Depth (m)	Screened Lithology	Sampling Date	Groundwater Level (mbgs)
MW21-01	8.1 - 11.1	Bedrock (Inferred)	April 29, 2021	6.5
MW21-02	7.0 - 10.0	Limestone Bedrock	April 30, 2021	6.8
MW21-03	9.8 - 12.8	Bedrock (Inferred)	April 30, 2021	6.9
			August 15, 2023	6.7
MW23-01	13.6 - 16.6	Limestone Bedrock	August 14, 2023	5.9
MW23-02	6.1 - 9.1	Bedrock (Inferred)	August 14, 2023	6.3
			September 13, 2023	6.3
MW23-03	6.1 - 9.1	Bedrock (Inferred)	August 14, 2023	6.2
MW23-04	4.6 - 7.6	Bedrock (Inferred)	August 14, 2023	6.0

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 should be noted that perched water may be present with the fill material and at the overburden/bedrock interface. Englobe has prepared a hydrogeological report for this Site as part of the scope of work of Englobe's proposal. The results of the hydrogeological report are available under separate cover.

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6 Discussion and Recommendations

Based on the results of the geotechnical field investigations and laboratory testing performed at the borehole locations on this Site, the following discussion and recommendations are provided to assist the Client and their Designers with the foundation 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 in the boreholes, 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:

- **Preliminary Seismic Site Classification:** In accordance with OBC 2012, structures designed under Part 4 of the Code must be designed to resist a minimum earthquake force. Englobe was not retained to conduct a seismic shear wave velocity study for the Site. An independent seismic shear wave velocity study for the Site was commissioned and conducted by the Client. The independent seismic shear wave velocity study provides a V_{S30} value which corresponds to a seismic "Site Class A" classification with respect to Table 4.1.8.4.A of OBC-2012, subject to the limitations of the code.

- **Stability of North Slope:** As discussed in Section 2, there is an existing approximately 6.0 m high slope running along the north side of the Site and sloping downward northwest to Danforth Avenue to an approximate elevation of El. 69.2 masl at the northwest corner of the Site. Englobe's current understanding of the Site plan based on the latest architectural concepts is that B3 basement level foundations are at an approximate elevation near El. 63.2 masl, and therefore the slope will be completely excavated and replaced with exposed building foundation walls. As such, Englobe has not recommended a slope stability assessment for the slope.

- **Protection of Adjacent Structures and Roadways:** Based on Englobe's current understanding of the Site plans, the current distance between the proposed structure and existing structures on the adjacent properties is less than 2.0 m. Designers and Contractors should review the geometry, depth and sloping requirements of all planned excavations to ensure that adjacent structures are not undermined. If space or property line restrictions are encountered or the spacing requirements to prevent undermining of adjacent structures cannot be satisfied, then an Engineered Shoring system and/or an underpinning program may be necessary.
- **Hydrogeological Investigation:** The recorded water levels in the installed monitoring wells were found to range from 5.9 to 6.9 mbgs. Given that excavations are expected to extend to an approximate depth of 12 mbgs, groundwater will be encountered during excavation and will need to be adequately controlled during construction. A Hydrogeological Investigation was conducted as part of the scope of work of Englobe's June 01, 2023 proposal to assess the expected groundwater infiltration expected during construction and to determine whether a permit to take water (PTTW) will be required and is provided for this Project under separate cover.

6.1 Site Preparation

All existing surficial materials, FILL soils, native overburden soils, all foundation elements associated with the existing building on site, and all weathered bedrock should be completely removed from within the footprint of the new structure to expose competent limestone bedrock capable of supporting the proposed structure.

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 storm water and runoff including an adequate pumping system, if necessary.

6.1.1 Subgrade Preparation

Subgrade preparation for footings founded on rock will involve the removal of all overburden soils and weathered bedrock to expose competent limestone bedrock to the designed subgrade level. Any pieces of rock that can be easily manipulated by conventional excavation equipment should be removed, as directed by the Geotechnical Engineer. Final subgrade surfaces should be brushed and/or air blown clean, and dry.

The exposed bedrock surface should be examined and approved by the Geotechnical Engineer to confirm the competency of foundation to support the design bearing pressures.

Confirmation of bedrock quality during construction will require the Contractor to perform probing of the bedrock using 50 mm diameter drill holes drilled to a depth of 1.5 m within the footings. These holes will need to be reviewed by the Geotechnical Engineer to confirm that no significant mud seams or voids exist. If mud seams are found, localized areas of the footings may need to be lowered below the mud seam, or footing sizes increased to lower design bearing pressures accordingly. The locations of these probe holes should be selected under the direction of the Geotechnical Engineer during construction. Contractors should plan for one probe per pad footing and a minimum or 1 probe every 6 m in strip footings.

Designers and Contractors should make some allowance for additional excavation of fractured rock to achieve a sound bedrock subgrade to the satisfaction of the Geotechnical Engineer. It is recommended that a unit price item for additional rock excavation and replacement with lean mix concrete fill be incorporated into the tender documents.

6.1.2 Interference with Existing Underground Utilities

Designers should review the proposed excavation locations and compare them to the location of any existing underground utilities. Existing utilities that are excavated or exposed as part of construction will need to be supported and/or rerouted around the building.

6.1.3 Interference with Existing Foundations

There are existing foundation elements present in the ground which correspond to the existing building on Site. Investigation of the depths of these foundations by means of test pits was not part of Englobe's scope of work. It is understood that the existing building and associated foundation elements will be excavated and removed from Site prior to commencement of construction. There will likely be existing deeper FILL soils present surrounding these foundations which should be completely removed from Site prior to commencement of construction.

6.1.4 Protection of Adjacent Structures

Based on Englobe's current understanding of the Site plans, the distance between the proposed residential building and the existing structure on the adjacent property at 352 Danforth Avenue is less than 2.0 m. Designers and Contractors should plan out the proposed excavation dimensions and compare them to the location and position of the adjacent load-bearing structures to ensure that they are not undermined during construction. Undermining is avoided by ensuring that no excavations penetrate below an imaginary line drawn outward and downwards at 10H:7V below the toe or founding level of any existing load-bearing structures. If the limitations to avoid undermining of adjacent load-bearing structures cannot be satisfied, then an Engineered Shoring system and/or an underpinning program may be necessary to provide adequate support to the adjacent structure.

6.2 Excavations

Based on Englobe's current understanding of the Project, it is anticipated that the excavations will extend to an approximate depth of 12 mbgs, based on the grade difference between the Site and Danforth Avenue to the north. Excavations will extend through the overburden soils and weathered bedrock surface and into the limestone bedrock.

6.2.1 Sloped Excavations

All excavations must be undertaken in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), Regulations for Construction O.Reg. 213/91, as amended. The comments within this subsection are intended to be in addition to, and not a replacement of the OHSA requirements.

The existing FILL material and native soils would be considered as a "Type 3 Soil" according to the OHSA regulations. However, if they become wet, muddy, is below the water table, or shows signs of seepage, they would be considered as a "Type 4 Soil". According to the OHSA, excavations which penetrate through multiple soil types should be considered as having the highest soil type.

For excavations into bedrock, there is an upper weathered rock zone that will require back sloping depending on the degree of weathering. The bedrock quality and Site-specific requirements need to be assessed during construction by the Geotechnical Engineer. For planning purposes, a weathered bedrock is recommended to be treated as a "Type 2 Soil". Sound rock, if encountered in excavations would generally be self-supporting, however, as a precautionary measure, it should be backsloped at 10V:1H.

Excavation of the bedrock may require a combination of hoe ramming, blasting, and line drilling. These operations will impart vibrations on the surrounding structures. It is recommended that a pre-construction survey be performed on the surrounding structures and roadways prior to construction, and that vibration monitoring be completed throughout the bedrock excavation process.

All rock excavations should be scaled, to remove loose rock fragments to ensure safe working conditions. All rock faces should be reviewed by a Geotechnical Engineer to look for loose pieces and wedge failures. Rock bolting for worker safety may be necessary depending on the layout and field condition at that time. The stability of the excavation side slopes will be highly dependent on the Contractor's methodology. 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 plan out the approximate excavation area and compare them to the location of the adjacent streets to ensure the pavement structures are not undermined during construction. If space or property line restrictions are encountered, then Engineered Shoring may be necessary to provide adequate support to the adjacent roadways.

6.2.2 Engineered Shoring

Engineered Shoring systems through soil often include (but are not limited to): soldier piles and lagging, interlocking sheet piles, secant and/or tangent walls, permanent diaphragm walls, etc. The appropriate method should be selected by the Project Designers and Contractors considering the space restrictions, estimated costs, and availability of materials. Engineered Shoring systems must be designed by a Professional Engineer taking into consideration the following Site-specific aspects:

- Lateral earth pressures;
- Hydraulic pressures of the groundwater;
- Loads from any adjacent structures, or infrastructure being retained;
- Seismic loadings;
- Freeze-thaw action on the face of the excavations;
- Expansion and contraction of shoring elements;
- Pre-stressing loads or post tensioning loads on tie backs;
    - Possible surcharge loads throughout construction (i.e., trucks, equipment, stockpiles, etc.);
- Vibrations induced by construction processes;
- Locked-in stresses in the bedrock; and
    - Compatibility with the design of proposed waterproofing and drainage systems for the sub-surface levels.

Soldier piles and sheet piling, if used would require predrilling to provide sufficient embedment to achieve toe fixity. It is expected that the Engineered Shoring systems would need to be provided with tie-back rock anchors to ensure their lateral stability. It is recommended that the Client retain Contractors and Designers who have significant experience with deep excavations performed under similar soil conditions. Shop drawings should be submitted to the Designers and reviewed by the Geotechnical Engineer well in advance of mobilization.

The preliminary lateral earth pressure parameters to assist Designers and Contractors with shoring designs through soil are discussed in Section 6.7 below.

6.3 Temporary Construction Dewatering

As discussed in Section 5.6, monitoring wells were installed at all borehole locations. The groundwater levels recorded on April 29 and 30, 2021 were found to range approximately from 6.5 to 6.9 mbgs, and groundwater levels recorded between August 14, 2023 and September 13, 2023 were found to range approximately from 5.9 to 6.7 mbgs. Given that excavations are expected to extend to an approximate depth of 12 mbgs, the excavation will extend below the groundwater table.

Both surface water and significant groundwater seepage are expected in the excavations and will need to be adequately controlled. Water quantities will depend on seasonal conditions, depths of excavations, presence and lateral extents of fractured rock zones, and the duration that excavations are left open. Groundwater will travel easily through the overburden soils and weathered bedrock. Existing utility trenches which join or intersect the excavations may act as a drain and supply off-Site water into the excavations. These should be plugged at the outset of construction to mitigate this possibility.

Effective groundwater control prior to and during construction and possibly permanently in this case are expected to be required. Englobe conducted a Hydrogeological Investigation as part of the scope of this project to assess the requirements for a Permit to Take Water (PTTW) or an Environmental Activity Sector Registry (EASR) application. Recommendations for appropriate dewatering measures beyond conventional sump pump techniques such as a positive dewatering system (e.g., well points or other specialized methods) to effectively lower the static groundwater level shall be provided by a specialized dewatering Contractor based on the findings and recommendations of the Hydrogeological Investigation Report (provided for this Project under a separate cover).

6.4 Foundations

It is Englobe's current understanding, based on the latest Site plans available at the time of this report, that foundations will be founded at an approximate depth of 12 mbgs (approximately near El. 63.8 masl). Based on this understanding and the soil and bedrock conditions encountered within the boreholes, it is recommended that the foundations for the proposed building be founded as conventional strip and pad footings on the sound limestone bedrock.

6.4.1 Footings on Sound Limestone Bedrock

For conventional pad and strip footings founded on sound limestone bedrock, a preliminary factored bearing resistance of 1000 kPa under Ultimate Limit States (ULS) conditions is recommended. This accounts for a geotechnical resistance factor of $\Phi = 0.5$.

There is no corresponding design bearing pressure recommended under Serviceability Limit State (SLS) conditions for bedrock as settlement under the ULS condition is expected to be minimal. Designers should limit footing dimensions to a minimum of 1.0 m for pad footings, and 0.5 m for strip footings regardless of the bearing pressure being used. Higher bearing resistances may be achievable but would require additional geotechnical investigation to confirm.

Subgrade preparation for footings founded on rock will involve the removal of all soils and weathered rock surface to expose sound bedrock. Any pieces of rock that can be easily manipulated by conventional excavation equipment should be removed, as directed by the Geotechnical Engineer. Final subgrade surfaces should be brushed and/or air blown clean, and dry. The exposed surface should be examined by the Geotechnical Engineer to assess its competency.

Confirmation of bedrock quality during construction will require probing of the bedrock at footing locations using 50 mm diameter holes drilled to a depth of 1.5 m within the footprint of footings. These holes will need to be reviewed by the Geotechnical Engineer to confirm that no significant mud seams or voids exist. If mud seams are found, localized areas of the footings may need to be lowered below the mud seam, or footing sizes increased to lower design bearing pressures accordingly. The locations of these probe holes should be provided under the direction of the Geotechnical Engineer during construction.

Designers and Contractors should make allowance for additional excavation of fractured rock to achieve a sound bedrock subgrade to the satisfaction of the Geotechnical Engineer. It is recommended that a unit price item for additional rock excavation and replacement with lean mix concrete fill be incorporated into the tender documents.

6.4.2 Lean Mix Concrete

If the grade is required to be raised between the approved sound bedrock subgrade and the design footing elevation, then it is recommended to use a lean mix concrete, as opposed to with granular fill soils. If lean mixed concrete is used below any footings it must extend a minimum of 0.3 m beyond the edge of the footing and then downward at 1H:1V. Recommended design bearing pressures on lean mix concrete would be the same as those for the bedrock, provided that the underlying subgrade has been approved by the Geotechnical Engineer.

6.5 Frost Protection

The sound limestone bedrock on this Site is not considered frost susceptible. However, due to the sloping grade of the slope of the grade, minor elements may be founded at shallow depths in the overburden soils and weathered bedrock which are considered frost susceptible. Therefore, any footings founded at shallow depths (entrance canopy footings, signs, etc.) must be provided with adequate frost protection. Depending on the location and geometry of any retaining walls, the footings in these areas may also require a frost protection detail.

Footings founded within the overburden or weathered bedrock must be provided with a minimum of 1.5 m of earth cover if the structure is heated, and 1.8 m of earth cover for unheated or isolated structures in the Ottawa area. If the required earth cover is not achievable, 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 (OBC-2012), structures designed under Part Four of the Code must be designed to resist a minimum earthquake force An independent shear wave velocity survey was undertaken by an independent Contractor on behalf of the Client, and the survey results were provided to Englobe by the Client.

The seismic MASW survey conducted by the Client provides a calculated shear wave velocity value which can be used to estimate the Seismic Site Class. In the case that footings are founded on intact bedrock, the shear wave velocity value provided corresponds with Seismic Site Class “A” with respect to Table 4.1.8.4.A of the OBC-2012, subject to the limitations of the Code.

It is important to note that the provided shear wave velocity value is measured at the existing surface grade of the Site at an approximate elevation near El. 75.3 masl. According to Commentary J of the National Building Code of Canada 2015 (NBCC-2015), the shear wave velocity should be measured from 30 m below the underside of the proposed footings. Given a proposed founding elevation near El. 63.8 masl, and in the case that footings are founded on intact bedrock, the average shear wave velocity below the footings is estimated at 1774.6 m/s in the most conservative case. This would correspond with Seismic Site Class “A” with respect to Table 4.1.8.4.A of the OBC-2012, subject to the limitations of the Code.

It should be emphasized that Englobe was not involved in designing and conducting the shear wave velocity survey. The above information is presented only to assist Designers and Contractors with the design of structures and footings, and Designers and Contractors should make their own interpretation of the information presented.

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 6-1: 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
Existing Cohesionless FILL	20	30	0	0.33	0.50	3.00
Native Loose to Compact Sandy Silt	20	30	0	0.33	0.50	3.00
New Compacted Granular Backfill OPSS "Granular B, Type II"	22	35	0	0.27	0.43	3.69

** 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:

$$P_{ae} = ½ K_{ae}\gamma H^2 \times (1- k_v)$$

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.

Based on the seismic MASW survey provided by the Client, the Site Class-adjusted OBC-2012 PGA for the Site is 0.242g at Site Class A, where g is the acceleration due to gravity, and the probability of exceedance per annum is 0.000404. This value was determined using the 2015 National Building Code Seismic Hazard Calculation document and can be found attached in Appendix E.

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

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

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:

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

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 6-2: 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}
Existing Cohesionless FILL	20	30	0	0.52	1.91
Native Loose to Compact Sandy Silt	20	30	0	0.52	1.91
New Compacted Granular Backfill OPSS "Granular B, Type II"	22	35	0	0.49	2.21

** 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

Based on the design traffic condition in the proposed underground parking levels, Designers will need to decide what type of floor structure will be necessary in the parking garage. Typical options would be a free-floating slab-on-grade, or alternatively a structural slab.

Englobe was not provided with any design criteria for floor slab loadings and traffic loadings for the floor slab of the parking garage, therefore it is assumed that floor slabs are lightly loaded with no heavy racking or process machinery that require specific support.

A typical floor slab loading for a lightly loaded slab-on-grade would be a maximum value of 24 kPa. If larger slab loadings are envisioned, then Englobe should be retained to perform additional consulting in regard to design of the floor slab. For design purposes and based upon a properly prepared subgrade surface covered with 200 mm of Ontario Provincial Standard Specification (OPSS) 1010 ‘Granular A’, a typical preliminary modulus of subgrade reaction appropriate for the slab design would be approximately 25,000 kN/m^3 on Engineered Fill and compacted to 100 percent of its Standard Proctor Maximum Dry Density (SPMDD). Alternative values would require additional analysis and testing.

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 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. The floor finish manufacturer’s specifications and requirements should be consulted and procedures outlined in the manufacturer’s specifications should be followed.

Subgrade preparation below floor slabs will involve the removal of all soils and weathered bedrock to expose the intact sound limestone bedrock. Any pieces of rock that can be easily manipulated by conventional excavation equipment should be removed, as directed by the Geotechnical Engineer. Final subgrade surfaces should be brushed and/or air blown clean, and dry. The exposed bedrock surface should be examined and approved by the Geotechnical Engineer.

Any new fill used to raise the grade between the approved bedrock subgrade and the floor slab should be considered as Engineered Fill and should be placed in strict conformance with the requirements in Section 6.12.1.

6.9 Resistance to Foundation Uplift

Resistance to foundation uplift or overturning forces can be provided by considering the dead weight of the structures and backfill soils, increasing the dead weight of the structure using additional concrete elements, or with the use of additional rock anchors.

In the case that grouted rock anchors are considered, rock anchors may be designed based on a frictional stress between grout and intact limestone bedrock. Based upon typical published values and conservative approach, Englobe recommends that a conservative allowable working stress value of 600 kPa be used to calculate the length of the required bond zone. The bond zone must be entirely within "sound bedrock" which is below the weathered zone.

Designing in accordance with the Limit States Design (LSD) method, Designers may take the approach that working stress value is approximately equivalent to the SLS value. The ULS and SLS must be based upon both performance and structural criteria. However, based upon typical published values, the unfactored ULS values may be approximately 750 kPa to more than 1000 kPa. As per CFEM-2006, a geotechnical resistance factor of $\Phi=0.3$ should be applied to the empirical unfactored ULS values. Higher stress values may be available; however, performance load testing in the field will be required to prove the capacities. If performance testing is carried out at the outset of the Project, then a resistance factor of $\Phi=0.4$ could be applied.

In order to mobilize the shear stress in the rock, the load at the top of the anchor must be properly transferred through the upper bedrock to the bond zone to prevent progressive grout fail and ensure proper performance. Therefore, a “free length” is required through the foundation element, the weathered rock zone, and down to the bond zone.

The mass of rock mobilized by a rock anchor may be assumed to be based upon a 60-degree cone drawn upward from a point located at the lower one-third point of the bond zone and spaced such that the theoretical cones do not overlap. Designers should review the spacing of anchors and account for any overlapping cones (i.e. avoid doubling-up on rock mass calculations for overlapping cones). The bulk unit weight of bedrock may be assumed to be approximately 26 kN/m^3. The corresponding buoyant unit weight would be approximately 16 kN/m^3. It is recommended that Designers consider the water level to be near the subgrade elevation, and therefore, use submerged unit weights for the rock mass calculations.

6.10 Corrosion Potential of Soils

Analytical testing was carried out on one groundwater sample collected from the borehole MW21-02 location to determine corrosion potential of the Site groundwater. The selected groundwater sample was tested for pH, resistivity, chlorides, sulphides, and sulphates. The test results are summarized in the following table.

Table 6-3: Corrosion Parameter Results

Parameter	Tested Value
	MW21-02
pH	7.92
Chloride (mg/L)	1800
Sulphate (mg/L)	210
Resistivity (Ohm-cm)	160
Redox Potential (mV)	<0.020
Sulphides (%)	284

These results are presented to assist Designers in selecting adequate corrosion protection for underground utilities at the Site. Designers should compare testing results with relevant provincial and national standards during the design process.

The analytical results of the groundwater sample were compared with applicable Canadian Standards Association (CSA) A23.1-04 and are given in Table 6-4 below.

Table 6-4: Additional Requirement for Concrete Subjected to Sulphate Attack

Class of Exposure	Degree of Exposure	Water Soluble Sulphate (SO_4) in Groundwater Sample (mg/L)	Cementing Material to be Used
S-1	Very Severe	> 10,000	HS or HSb
S-2	Severe	1,500 - 10,000	HS or HSb
S-3	Moderate	150 - 1,500	MS, MSb, LH, HS, or HSb

The chemical sulphate content analyses for representative soil sample tested indicate a sulphate concentration of 210 mg/L in the soil samples tested. The results were compared with Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and possess a “moderate” risk for sulphate attack on concrete material as shown in Table 6-4. Therefore, sulphate resistant concrete is required for concrete substructures on this Site.

6.11 Waterproofing and Permanent Drainage

As discussed in Section 5.1, the groundwater levels measured in monitoring wells at the Site varied in depth from approximately 3.9 to 6.1 mbgs. Given that the founding depth of the proposed foundations will extend up to approximately 12 mbgs, consideration should be given to designing the building basement 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.

Under floor drainage is recommended for this structure based as the groundwater level is anticipated to be above the basement floor slab.

6.12 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:

6.12.1 Engineered Fill

For this Project, Engineered Fill may be required to raise the grade between the approved intact limestone bedrock subgrade and floor slabs, and for interior foundation wall backfill. 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 and weathered bedrock must be removed, and the subgrade approved by the Engineer. Any deficient areas should be repaired prior to placement; and
- 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.

6.12.2 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.

6.13 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.

6.13.1 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.

6.13.2 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.

6.13.3 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 and channel water into the proposed building footprint. 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.

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

ENGLOBE

Folder: C:\DST\02103035.000 424 Churchill\2023 Geotechnical Investigation\DWGs Tuesday, September 05, 2023 @ 19:16 by Kris Morin

Note

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

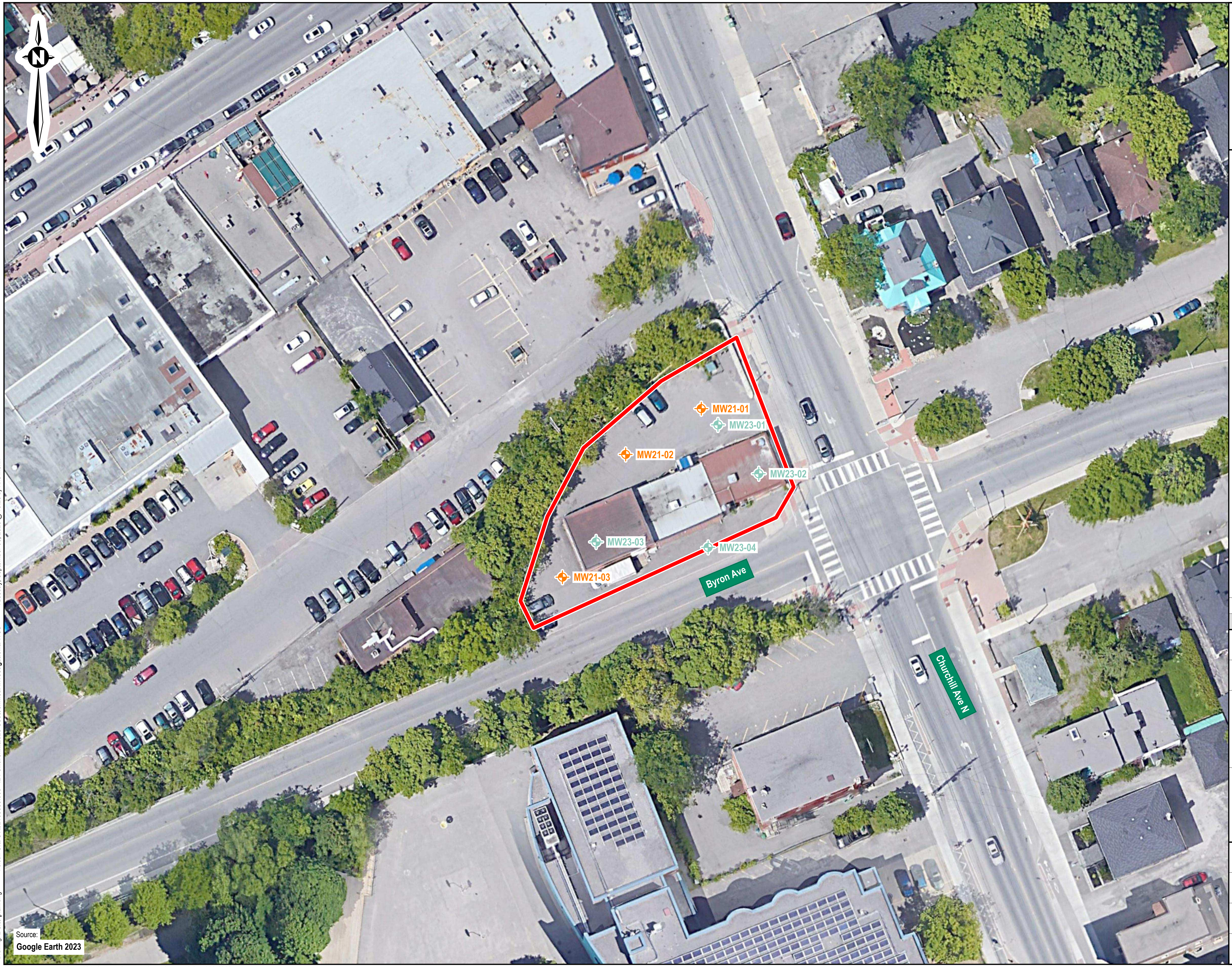
DRAFT

Revision	Date	Issue	Approval
A	09/05/2023	Preliminary	

Client: Churchill Properties Inc.	Site: 424 Churchill Avenue North, Ottawa, ON

englobe	Report Title: Geotechnical Investigation Proposed 7-Story Multi-Unit Apartment Building	Designed By: G.C.	Date: September 2023
		Drawn By: K.M.	Project No. 02103035.001
	Drawing Title: Site Location Map	Approved By:	Figure No. 1
		Scale: As shown	

Drawing: 1 site location map.dwg



ENGLOBE

Note

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

Legend

- Approximate site limits
- Previous borehole / monitoring well location
- Current borehole / monitoring well location

1:600

0 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	09/05/2023	Preliminary	

Client
Churchill Properties Inc.

Site
424 Churchill Avenue North, Ottawa, ON

Report Title
Geotechnical Investigation
Proposed 7-Story Multi-Unit Apartment Building

Drawing Title
Borehole Location Plan

Designed By: G.C.	Scale: As shown
Drawn By: K.M.	Date: September 2023
Approved By:	Project No.: 02103035.001
Figure No.: 2	DRAFT

Drawing: 2 site plan.dwg Folder: C:\DST\02103035.000 424 Churchill\2023 Geotechnical Investigation\DWGs Tuesday, September 05, 2023 @ 19:28 by Kris Morin

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Appendix C

List of Symbols and Definitions

Englobe 2023 Borehole Logs

DST 2021 Borehole Logs

ENGLOBE

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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 – 0425
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 MW23-01

DST REF. No.: **02103035.001**
CLIENT: **Churchill Properties Inc.**
PROJECT: **Geotechnical Investigation**
LOCATION: **424 Churchill Ave. N, Ottawa**
SURFACE ELEV.: **75.27 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Massenza MI3 Track Mount - HSA**
START DATE: **07/11/2023**
COMPLETION DATE: **07/11/2023**
COORDINATES: **5026692.732** m N, **441016.497** m E

DEPTH (m)	ELEV. (m)	MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE / RECOVERY %	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
	75	**APPROX. 100mm THICK ASPHALT**	GS1	43 / 100%	
		FILL: silty sand, some gravel, brown, damp	SS1	73 / 83%	
1.0	74	**SILTY SAND:** trace to some gravel, brown, damp, (very dense) Auger refusal at 0.9 mbgs, continue with rock coring			
		LIMESTONE: fair quality, grey, weathered	RC1	50%	RQD = 53% SCR = 72% TCR = 100% FFI = 4/8/1
2.0	73	Becoming good quality			
3.0	72		RC2	100%	RQD= 78% SCR= 98% TCR= 100% FFI = 8/2/2/5/1
4.0	71		RC3	96%	RQD= 48% SCR= 94% TCR= 96% FFI = 8/1/3/5/13
5.0	70				
6.0	69		RC4	100%	RQD = 86% SCR = 93% TCR = 100% FFI = 8/3/0/2/2
7.0	68		RC5	93%	RQD = 82% SCR = 92% TCR = 93% FFI = 3/0/2/1/9
8.0	67				
9.0	66		RC6	87%	RQD = 69% SCR = 87% TCR = 87% FFI = 8/3/1/4/3
10.0	65	Becoming fair quality			
11.0	64		RC7	98%	RQD = 71% SCR = 88% TCR= 98% FFI = 12/3/5/5/3

Column headings: % MOISTURE (W_p, W, W_l; 20 40 60 80); Water Data; Symbol; SAMPLE TYPE; Su (kPa) VANE ✕ PP ✳ 40 80 120 160; SPT (N) □ Blows/0.3m DCPT ◆ 20 40 60 80; CHVC 1 (ppm); CHVC 2 (ppm)

ENGLOBE
101-2713 LANCASTER ROAD
OTTAWA, ON, K1B 5R6
PH: 1-877-300-4800
FX: 1-888-979-6772
Web: www.englobecorp.com

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

LOG OF BOREHOLE MW23-01

DST REF. No.: **02103035.001**
CLIENT: **Churchill Properties Inc.**
PROJECT: **Geotechnical Investigation**
LOCATION: **424 Churchill Ave. N, Ottawa**
SURFACE ELEV.: **75.27 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Massenza MI3 Track Mount - HSA**
START DATE: **07/11/2023**
COMPLETION DATE: **07/11/2023**
COORDINATES: **5026692.732** m N, **441016.497** m E

DEPTH (m)	ELEV. (m)	MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE / RECOVERY %	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
	63		RC8	Rock Core	100%	RQD = 89% SCR = 94% TCR = 100% FFI = 0/0/0/4/1
13.0	62		RC9	Rock Core	100%	RQD = 72% SCR = 85% TCR = 100% FFI = 1/6/3/2/1
14.0	61	Becoming excellent quality				
15.0	60		RC10	Rock Core	100%	RQD = 100% SCR = 100% TCR = 100% FFI = 0/0/1/2/2
16.0	59		RC11	Rock Core	100%	RQD = 72% SCR = 81% TCR = 100% FFI = 3/4/6
17.0	58	**End of borehole at approximately 16.7 mbgs (~Elev. 58.7 masl) in limestone.** Water level measured in monitoring well was approximately 5.9 mbgs (~Elev. 69.4 masl) on August 14, 2023.				
18.0	57					
19.0	56					
20.0	55					
21.0	54					
22.0	53					
23.0	52					

Column headings: Water Data; % MOISTURE W_p W W_l 20 40 60 80; Symbol; Su (kPa) VANE ✕ PP✳ 40 80 120 160; SPT (N) □ Blows/0.3m DCPT ◆ 20 40 60 80; CHVC 1 (ppm); CHVC 2 (ppm)

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FX: 1-888-979-6772
Web: www.englobecorp.com

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

LOG OF BOREHOLE MW23-02

DST REF. No.: **02103035.001**
CLIENT: **Churchill Properties Inc.**
PROJECT: **Geotechnical Investigation**
LOCATION: **424 Churchill Ave. N, Ottawa**
SURFACE ELEV.: **73.57 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hilti Portable Core Drill - Direct Push**
START DATE: **07/19/2023**
COMPLETION DATE: **07/19/2023**
COORDINATES: **5026684.758** m N, **441023.163** m E

DEPTH (m)	ELEV. (m)	MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE / RECOVERY %
0.0	73.57	**APPROX. 25mm THICK CONCRETE SLAB**	AS1	100%
		FILL: sand and gravel, loose, brown, damp Auger refusal at 0.1 mbgs, continue with rock coring		
		BEDROCK Inferred, no recovery due to drilling method		
9.2	64.4	**End of borehole at approximately 9.2mbgs (~Elev. 64.4 masl) in inferred bedrock.** Water level measured in monitoring well was approximatley 3.9mbgs (~Elev. 69.8masl) on August 14, 2023.		

Column headings: DEPTH (m); ELEV. (m); Water Data; % MOISTURE (W_p, W, W_l; 20 40 60 80); Symbol; MATERIAL DESCRIPTION; SAMPLE #; SAMPLE TYPE; 'N' VALUE / RECOVERY %; Su (kPa) VANE ✕ PP ✳ 40 80 120 160; SPT (N) □ Blows/0.3m, DCPT ◆ 20 40 60 80; CHVC 1 (ppm); CHVC 2 (ppm); REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL

Depth scale (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0
Elevation scale (m): 73, 72, 71, 70, 69, 68, 67, 66, 65, 64, 63, 62

ENGLOBE
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101-2713 LANCASTER ROAD
OTTAWA, ON, K1B 5R6
PH: 1-877-300-4800
FX: 1-888-979-6772
Web: www.englobecorp.com

SAMPLE TYPE LEGEND
Auger Sample
Split Spoon Sample
Bulk Sample
Rock Core
Core 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

LOG OF BOREHOLE MW23-03

DST REF. No.: **02103035.001**
CLIENT: **Churchill Properties Inc.**
PROJECT: **Geotechnical Investigation**
LOCATION: **424 Churchill Ave. N, Ottawa**
SURFACE ELEV.: **75.92 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Hilti Portable Core Drill - Direct Push**
START DATE: **07/20/2023**
COMPLETION DATE: **07/20/2023**
COORDINATES: **5026673.617** m N, **440996.601** m E

DEPTH (m)	ELEV. (m)	MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE / RECOVERY %
0.0–0.025	75.92	**APPROX. 25mm THICK CONCRETE SLAB**	SS1	Split Spoon	100%
		FILL: sand and gravel, grey, damp			
		SILTY SAND: brown, damp, (dense to very dense) Auger refusal at 0.8 mbgs, continue with rock coring			
		BEDROCK Inferred, no recovery due to drilling method			
9.2	66.7	**End of borehole at approximately 9.2mbgs (~Elev. 66.7 masl) in inferred bedrock.** Water level measured in monitoring well was approximately 6.2mbgs (~Elev. 69.7masl) on August 14, 2023.			

Column headings: DEPTH (m); ELEV. (m); Water Data; % MOISTURE (W_p, W, W_l; 20 40 60 80); Symbol; MATERIAL DESCRIPTION; SAMPLE #; SAMPLE TYPE; 'N' VALUE / RECOVERY %; Su (kPa) VANE ✕ PP ✳ 40 80 120 160; SPT (N) □ Blows/0.3m DCPT ◆ 20 40 60 80; CHVC 1 (ppm); CHVC 2 (ppm); REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0
Elevation scale: 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64

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PH: 1-877-300-4800
FX: 1-888-979-6772
Web: www.englobecorp.com

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

LOG OF BOREHOLE MW23-04

DST REF. No.: **02103035.001**
CLIENT: **Churchill Properties Inc.**
PROJECT: **Geotechnical Investigation**
LOCATION: **424 Churchill Ave. N, Ottawa**
SURFACE ELEV.: **75.77 metres**
**Elevations are not geodetic, for reference within this report only.*

Drilling Data
METHOD: **Massenza MI3 Track Mount - HSA**
START DATE: **07/12/2023**
COMPLETION DATE: **07/12/2023**
COORDINATES: **5026672.722** m N, **441014.891** m E

DEPTH (m)	ELEV. (m)	MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE / RECOVERY %
0.0–0.1	75.77	**APPROX. 100mm THICK ASPHALT**			
0.1–0.9		**FILL:** sand, trace gravel, brown, damp - Auger refusal at 0.2 mbgs, continue with rock coring	SS1	Split Spoon	50%
0.9–8.2		**BEDROCK** Inferred, no recovery due to drilling method			
8.2	67.6	**End of borehole at approximately 8.2mbgs (~Elev. 67.6 masl) in inferred bedrock.** Water level measured in monitoring well was approximately 5.9mbgs (~Elev. 69.8masl) on August 14, 2023.			

Column headings: % MOISTURE (W_p, W, W_l; 20 40 60 80); Symbol; Su (kPa) VANE ✕ PP ✳ 40 80 120 160; SPT (N) □ Blows/0.3m DCPT ◆ 20 40 60 80; CHVC 1 (ppm); CHVC 2 (ppm); REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0
Elevation scale: 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64

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PH: 1-877-300-4800
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Web: www.englobecorp.com

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

MW21-01

consulting engineers
a division of Englobe

DST Project No.	02103035.000	Date	April 21, 2021
Client	GSI Group Cold Storage Ltd.	Method	Hollow Stem Auger & Pneumatic Drilling
Project	Preliminary Geotechnical Investigation		
Address	424 Churchill Avenue North, Ottawa, ON		

Depth (m)	Elevation (m)	Water level (mREL)	Well construction	Depth (m) Elevation (m)	Symbol	Material Description	Sample #	Sample Type	'N' Value/RQD %	CCGD / PID Reading: CCGD	CCGD / PID Reading: PID	Analysis (Submitted for laboratory analysis): PAHs	PHCs	Metals	VOCs	pH	Remarks
				0		ASPHALT - (140 mm thickness)	GS1										
				0.1		FILL - Silty sand, trace gravel, loose, brown, damp											
0.5				0.5		SANDY SILT - trace gravel, compact, brown, damp	SS1		6	25 ppm	0 ppm						
1.0							SS2		50+	210 ppm	1 ppm		✓		✓		
1.5				1.2		BEDROCK - Borehole advanced into bedrock using Tri-cone air drilling methods (bedrock type and quality could not be confirmed)											
2.0																	
2.5																	
3.0																	
3.5																	
4.0																	
4.5																	
5.0																	
5.5																	
6.0																	
6.5		▼															Groundwater level at 6.5 mbgs on April 30, 2021.
7.0																	
7.5																	
8.0																	
8.5																	
9.0																	
9.5																	
10.0																	
10.5																	
11.0																	
11.5						End of Borehole at 11.1 m.											
12.0																	
12.5																	

MW21-02

DST consulting engineers
a division of Englobe

DST Project No.	02103035.000	Date	April 21, 2021
Client	GSI Group Cold Storage Ltd.	Method	Hollow Stem Auger & Wireline Diamond coring
Project	Preliminary Geotechnical Investigation		
Address	424 Churchill Avenue North, Ottawa, ON		

Depth (m) / Elevation (m)	Material Description	Sample #	Sample Type	'N' Value/RQD %	CCGD	PID	PAHs	PHCs	Metals	VOCs	pH	Remarks
0	ASPHALT - (120 mm thickness)	GS1										
0.1	FILL - Sand, some gravel, compact, brown, damp											
0.5	SANDY SILT - trace gravel, compact, brown, damp,	SS1		11	360 ppm	1 ppm						
		SS2		50+	710 ppm	3 ppm		✔		✔	✔	
1	LIMESTONE - highly weathered and fractured, grey Auger refusal encountered at 1.4 mbgs											
1.4	LIMESTONE - poor quality based on RQD, slightly weathered, strong, medium to thickly bedded	RC1		43								
		RC2		37								
3.8	- 0.1m thick shale bed											
4.5	- fair quality based on RQD, fresh	RC3		68								
6.1	- excellent quality based on RQD	RC4		93								Groundwater level at 6.8 mbgs on April 29, 2021.
		RC5		92								
9	- fair quality based on RQD	RC6		50								
10.0	End of Borehole at 10.0 m.											

Analysis: Submitted for laboratory analysis

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0, 12.5

Column headings: Depth (m) | Elevation (m) | Water level (mREL) | Well construction | Depth (m) Elevation (m) | Symbol | Material Description | Sample # | Sample Type | 'N' Value/RQD % | CCGD / PID Reading (CCGD, PID) | Analysis (PAHs, PHCs, Metals, VOCs, pH) | Remarks

MW21-03

consulting engineers
a division of Englobe

DST Project No.	02103035.000	Date	April 22, 2021
Client	GSI Group Cold Storage Ltd.	Method	Hollow Stem Auger & Pneumatic Drilling
Project	Preliminary Geotechnical Investigation		
Address	424 Churchill Avenue North, Ottawa, ON		

Depth (m) / Elevation (m)	Material Description	Sample #	Sample Type	'N' Value/RQD %	CCGD	PID	PAHs	PHCs	Metals	VOCs	pH	Remarks
0	ASPHALT - (140 mm thickness)	GS1										
0.1	FILL - Silty sand, trace gravel, brown, compact, damp	SS1		50+	5 ppm	0 ppm		✓		✓		
0.3	SANDY SILT - trace gravel, brown, compact, damp											
0.8	BEDROCK - Borehole advanced into bedrock using Tri-cone air drilling methods (bedrock type and quality could not be confirmed)											Groundwater level at 6.9 mbgs on April 30, 2021.
	End of Borehole at 12.8 m.											

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0, 12.5

Analysis: Submitted for laboratory analysis

Appendix D

Laboratory Test Results

DST, a division of Englobe
203-2150 THURSTON DRIVE
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T: 1.877.300.4800
F: 1.888.979.6772
ottawalab@dstgroup.com
www.dstgroup.com

Moisture Content Test Results

DST Ref. No.: 02103035	**Date:** May 11, 2021.
Project: 424 Churchill Avenue North	**Client:** GSI Group Cold Storage Ltd.

Sample ID	Mass of Tare (g)	Mass of Wet & Tare (g)	Mass of Dry & Tare (g)	Mass of Wet Sample (g)	Mass of Dry Sample (g)	Mass of Water (g)	% MC
MW21-01 (SS1A)	13.6	62.8	59.6	49.2	46.0	3.2	6.96
MW21-01 (SS1B)	13.6	61.8	55.6	48.2	42.0	6.2	14.76
MW21-01 (SS2)	13.5	35	33.5	21.5	20.0	1.5	7.50
MW21-02 (GS1)	13.6	56.2	53.2	42.6	39.6	3.0	7.58
MW21-02 (SS1)	13.6	40	36.2	26.4	22.6	3.8	16.81
MW21-02 (SS2)	13.5	29.0	28.4	15.5	14.9	0.6	4.03
MW21-03 (GS1)	13.7	36.1	35.0	22.4	21.3	1.1	5.16
MW21-03 (SS1)	13.5	54.1	52.7	40.6	39.2	1.4	3.57

Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

May 11, 2021
File: 122411080

Attention: **DST Consulting Engineers, File #02103035**

Reference: **ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core**

The following table summarizes two rock core compressive strength results.

Location	Compressive Strength (MPa)	Description of Break
10-MW21-2 RC-5	152.7	Well-formed cones at both ends
10-MW21-2 RC-6	141.1	Well-formed cones at both ends

Sincerely,

Stantec Consulting Ltd

Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
brian.prevost@stantec.com

Appendix E

Rock Core Photos

2010 National Building Code Seismic Hazard Calculations

ENGLOBE	**Rock Core Photographs** **424 Churchill Road N, Ottawa, ON**	**Project No.:** 02103035.001

RC-01: 74.4 to 73.3 masl	RC-02: 73.3 to 71.8 masl	RC-03: 71.8 to 70.2 masl
RQD = 53%	RQD= 78%	RQD= 48%
SCR = 72%	SCR= 98%	SCR= 94%
TCR = 100%	TCR= 100%	TCR= 96%
FFI = 4/8/1	FFI = 8/2/2/5/1	FFI = 8/1/3/5/13

DRY CORE

WET CORE

Figure 3A	**Borehole:** MW23-01 **Rock Core No.:** RC-01 to RC-03	**Date:** September 01, 2023

Rock Core Photographs 424 Churchill Road N, Ottawa, ON	**Project No.:** 02103035.001

RC-04: 70.2 to 68.7 masl	RC-05: 68.7 to 67.2 masl	RC-06: 67.2 to 65.4 masl
RQD = 86%	RQD = 82%	RQD = 69%
SCR = 93%	SCR = 92%	SCR = 87%
TCR = 100%	TCR = 93%	TCR = 87%
FFI = 8/3/0/2/2	FFI = 3/0/2/1/9	FFI = 8/3/1/4/3

DRY CORE

WET CORE

Figure 3B	**Borehole:** MW23-01 **Rock Core No.:** RC-04 to RC-06	**Date:** September 01, 2023

ENGLOBE	**Rock Core Photographs** **424 Churchill Road N, Ottawa, ON**	**Project No.:** 02103035.001

RC-07: 65.4 to 63.8 masl	RC-08: 63.8 to 62.5 masl	RC-09: 62.5 to 60.9 masl
RQD = 71%	RQD = 89%	RQD = 72%
SCR = 88%	SCR = 94%	SCR = 85%
TCR= 98%	TCR = 100%	TCR = 100%
FFI = 12/3/5/5/3	FFI = 0/0/0/4/1	FFI = 1/6/3/2/1

DRY CORE

WET CORE

Figure 3C	**Borehole:** MW23-01 **Rock Core No.:** RC-07 to RC-09	**Date:** September 01, 2023

ENGLOBE	**Rock Core Photographs** **424 Churchill Road N, Ottawa, ON**	**Project No.:** 02103035.001

RC-09: 62.5 - 60.9 masl	RC-10: 60.9 to 59.5 masl	RC-11: 59.5 to 58.6 masl
RQD = 72%	RQD = 100%	RQD = 72%
SCR = 85%	SCR = 100%	SCR = 81%
TCR = 100%	TCR = 100%	TCR = 100%
FFI = 1/6/3/2/1	FFI = 0/0/1/2/2	FFI = 3/4/6

DRY CORE

WET CORE

Figure 3D	**Borehole:** MW23-01 **Rock Core No.:** RC-09 to RC-11	**Date:** September 01, 2023

consulting engineers
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Rock Core Photographs
424 Churchill Ave. N, Ottawa, ON

Project No.: 02103035.000

RC1-1.4m

RC1-2.9m

RC2-2.9m

RC2-4.5m

Rock Core Photo No.: 1

Borehole: MW21-02 (RC1 & RC2)

Depth: 1.4 to 4.5 m

RC3-4.5m

RC3-6.0m

RC4-6.0m

RC4-7.4m

Rock Core Photo No.: 2

Borehole: MW21-02 (RC3 & RC4)

Depth: 4.5 to 7.4 m

DST consulting engineers a division of Englobe	Rock Core Photographs 424 Churchill Ave. N, Ottawa, ON	Project No.: 02103035.000

RC5-7.4m

RC5/RC6-8.9m

RC6-10.0m

Rock Core Photo No.: 3	Borehole: MW21-02 (RC5 & RC6)	Depth: 7.4 to 10.0 m

2010 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: 45.391N 75.754W **User File Reference:** 424 Churchill Avenue North, Ottawa, ON 2023-09-01 13:45 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 (0.2)	0.632	0.384	0.247	0.088
Sa (0.5)	0.306	0.185	0.121	0.043
Sa (1.0)	0.137	0.087	0.055	0.017
Sa (2.0)	0.046	0.028	0.018	0.006
PGA (g)	0.322	0.200	0.121	0.038

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

GEOPHYSICS GPR INTERNATIONAL INC.

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Tel. : (450) 679-2400
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www.geophysicsgpr.com

July 4th, 2022

Transmitted by email: jemmy@gsiproperties.ca
Our Ref.: GPR-22-03855

Mrs. Jemmy Taing
GSI Properties
5 - 145 Select Avenue
Toronto ON M1V 5M8

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
424 Churchill Avenue North, Ottawa (ON)

Dear Madam,

Geophysics GPR International inc. has been mandated by GSI Properties to carry out seismic shear wave surveys at 424 Churchill Avenue N, in Ottawa (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Spatial AutoCorrelation (SPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocity values were calculated for the soil and the rock, to determine the Site Class.

The surveys were carried out on June 28th, 2022, by Mr. Elliot Lessard and Mr. Émile Thibault, trainee. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the testing methods, and the results presented in table and graph.

MASW PRINCIPLE

The *Multi-channel Analysis of Surface Waves* (MASW) and the *SPatial AutoCorrelation* (SPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface wave. The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones' spread axis. Conversely, the SPAC is considered a "passive" method, using the low frequency "signals" produced far away. The method can also be used with "active" seismic source records. The SPAC method generally allows deeper Vs soundings. Its dispersion curve can then be merged with the one of higher frequency from the MASW to calculate a more complete inversion. The dispersion properties are expressed as a change of velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding).

Figure 3 schematically outlines the basic operating procedure for the MASW method. Figure 4 illustrates an example of one of the MASW/SPAC records, the corresponding spectrogram analysis and resulting 1D V_S model.

INTERPRETATION

The main processing sequence involved data inspection and edition when required; spectral analysis ("phase shift" for MASW, and "cross-correlation" for SPAC); picking the fundamental mode; and 1D inversion of the MASW and SPAC shot records using the SeisImagerSW™ software. The data inversions used a nonlinear least squares algorithm.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_S) is of the order of 15% or better.

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

SURVEY DESIGN

The seismic acquisition spreads were laid on the Laundry Land's parking lot (Figure 2). The geophone spacing was of 2.0 metres for the main spread, using 24 geophones. Two shorter seismic spreads, with geophone spacing of 0.5 and 1.0 metre, were dedicated to the near surface materials. The seismic records were produced with a seismograph Terraloc Pro 2 (from ABEM Instrument), and the geophones were 4.5 Hz. The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and 40 µs for the seismic refraction. The records included a pre-trigged portion of 10 ms. An 8 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic spreads. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

The shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length.

RESULTS

Using seismic refraction (V_P) the rock depth was calculated between 1.8 and 2.6 metres (± 1 metre). Its seismic velocity (V_S) was calculated at 1965 m/s for the sound shallow portion. These parameters were used for the initial geophysical models, prior to the MASW results inversions.

The MASW calculated V_S results are illustrated at Figure 5.

The $\overline{V}_{S30}$ value results from the harmonic mean of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface down to 30 metres, as:

$$\overline{V}_{S30} = \frac{\sum_{i=1}^{N} H_i}{\sum_{i=1}^{N} H_i / V_i} \quad | \quad \sum_{i=1}^{N} H_i = 30 \text{ m}$$

(N: number of layers; H_i : thickness of layer "i" ; V_i : V_S of layer "i")

Thus, the $\overline{V}_{S30}$ value represents the seismic shear wave velocity of an equivalent homogeneous single layer response, between the surface and 30 metres deep.

The calculated $\overline{V}_{S30}$ value of the actual site is 1504 m/s (Table 1), corresponding to the Site Class "A".

CONCLUSION

Geophysical surveys were carried out to identify the Site Class at 424 Churchill Avenue North, Westboro, in Ottawa (ON). The seismic surveys used the MASW and the SPAC analysis, and the seismic refraction to calculate the $\overline{V}_{S30}$ value. Its calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 1504 m/s, corresponding to the Site Class "A" ($\overline{V}_{S30}$ > 1500 m/s), as determined through the MASW and SPAC methods, Table 4.1.8.4.-A of the NBC, and the Building Code, O. Reg. 332/12. It must be noted that Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated materials between the rock and the bottom of the spread footing or mat foundation.

It must also be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, very soft clays, high moisture content etc. (cf. Table 4.1.8.4.A of the NBC) can supersede the Site classification provided in this report based on the $\overline{V}_{S30}$ value.

The V_S values calculated are representative of the in situ materials and are not corrected for the total and effective stresses.

Hoping the whole to your satisfaction, we remain yours truly,

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Senior Project Manager

Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

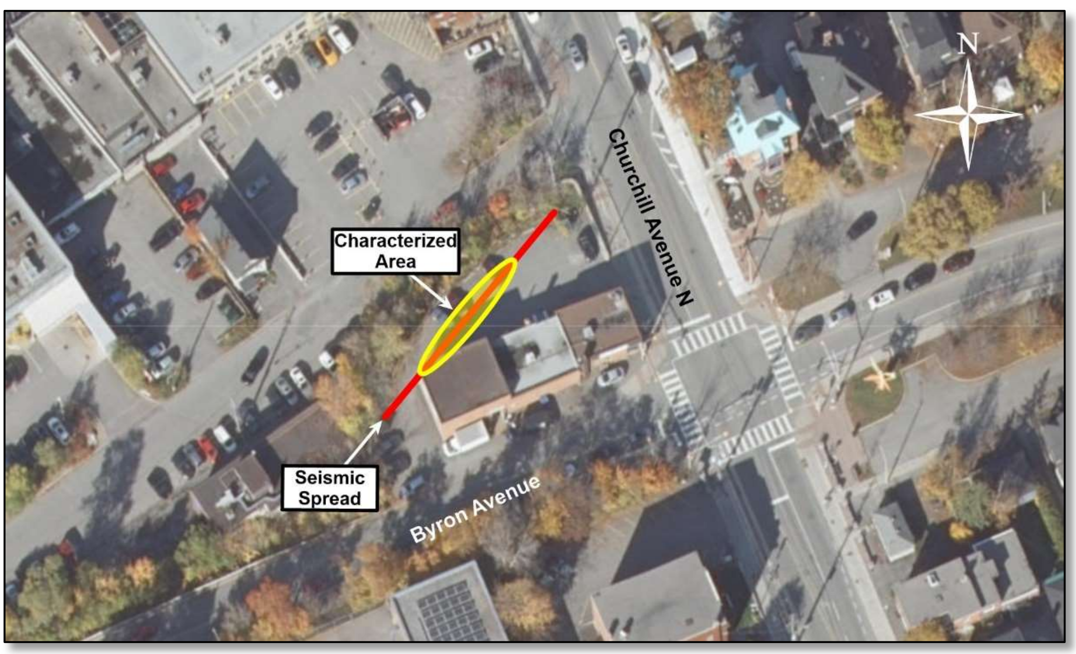

Figure 2: Location of the seismic spreads
(source: *geoOttawa*)

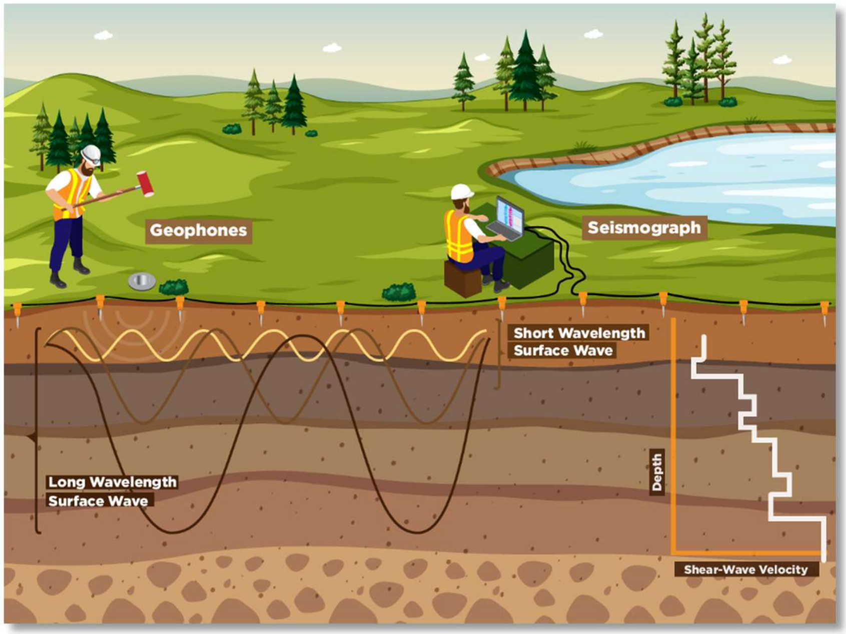

Figure 3: MASW Operating Principle

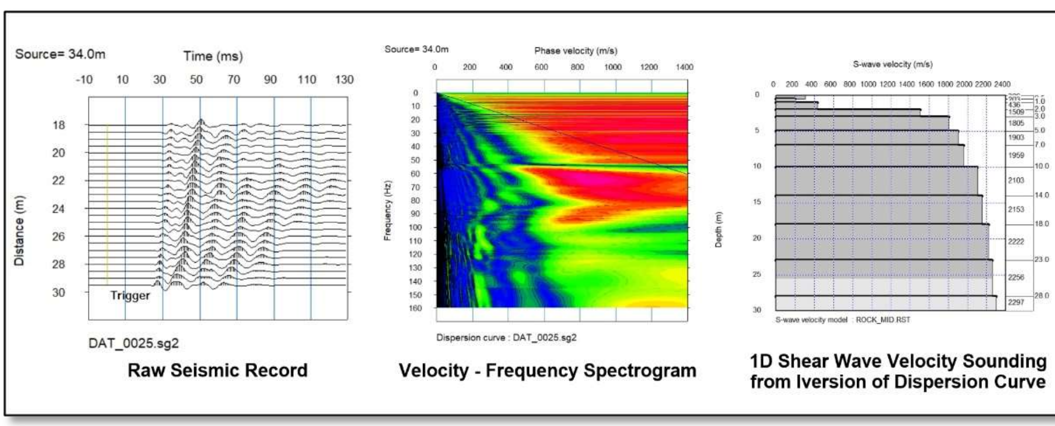

Figure 4: Example of a MASW/SPAC record, Phase Velocity - Frequency curve of the Rayleigh wave and resulting 1D Shear Wave Velocity Model

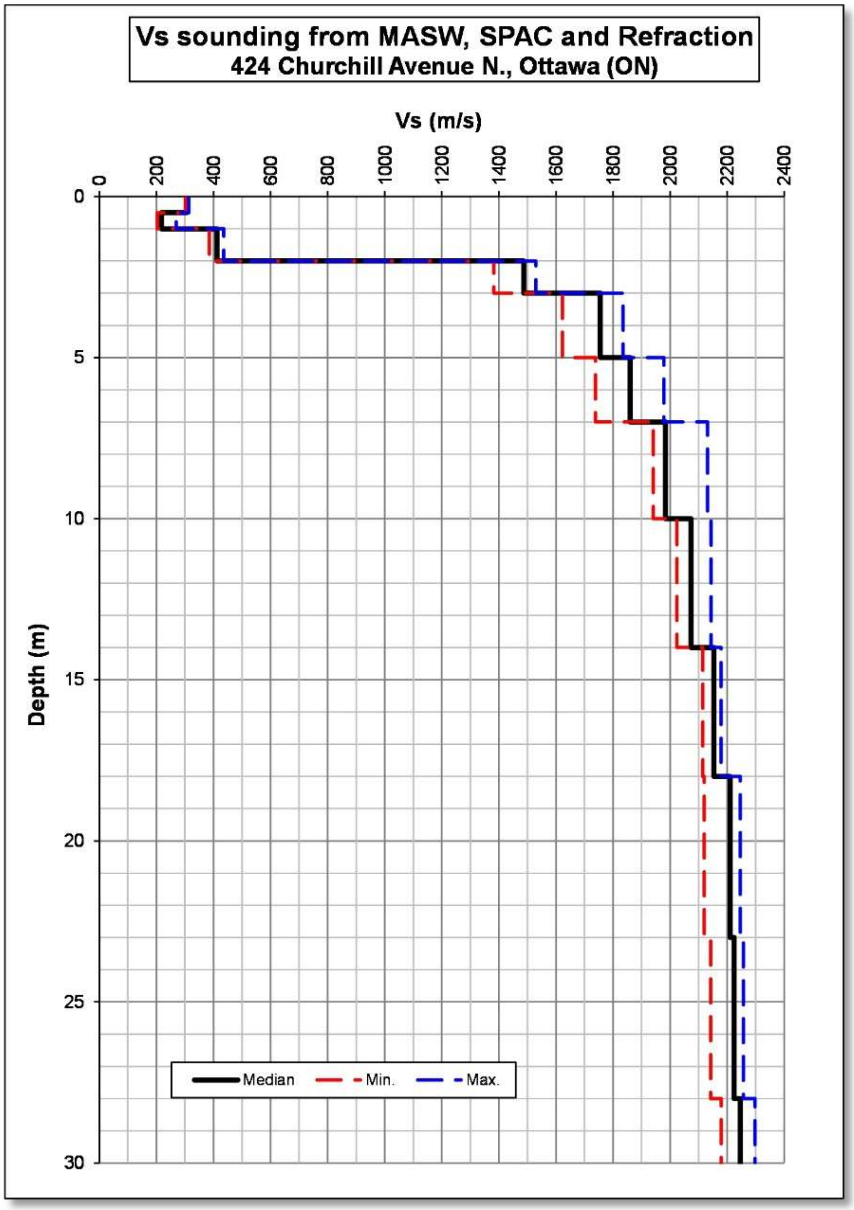

Figure 5: MASW Shear-Wave Velocity Sounding

TABLE 1

V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	302.4	305.6	313.5	**Grade Level (June 28th, 2022)**				
0.5	203.3	217.6	268.8	0.5	0.5	0.001636	0.001636	305.6
1.0	385.3	412.4	435.7	0.5	1.0	0.002298	0.003934	254.2
2.0	1382.2	1487.6	1529.8	1.0	2.0	0.002425	0.006359	314.5
3.0	1622.7	1755.2	1834.8	1.0	3.0	0.000672	0.007031	426.7
5.0	1738.8	1859.5	1977.8	2.0	5.0	0.001139	0.008170	612.0
7.0	1941.4	1983.9	2130.9	2.0	7.0	0.001076	0.009246	757.1
10.0	2023.9	2072.9	2143.3	3.0	10.0	0.001512	0.010758	929.5
14.0	2114.4	2153.4	2178.6	4.0	14.0	0.001930	0.012688	1103.4
18.0	2119.6	2210.1	2245.8	4.0	18.0	0.001858	0.014545	1237.5
23.0	2141.8	2223.8	2256.8	5.0	23.0	0.002262	0.016808	1368.4
28.0	2178.8	2245.8	2297.5	5.0	28.0	0.002248	0.019056	1469.3
30.0				2.0	30.0	0.000891	0.019947	1504.0

V_{S30} (m/s)	**1504.0**
Class	**A**

(1) The Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated materials between the rock surface and the bottom of the spread footings or mat foundation.