- **11061917 Canada Inc.**

Geotechnical Investigation

Type of Document:
Final

Project Name:
Residential Development
365 Forest Street
Ottawa, Ontario

Project Number:
OTT-00252625-A0

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Date Submitted
February 6, 2020

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Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed residential development to be located on the parcel of land identified by the following three (3) lot addresses; 365 Forest Street, 2589 Bond Street and 1420 Richmond Road, Ottawa, Ontario. For purposes of this report, the parcel of land is identified by one (1) address, 365 Forest Street. Authorization to proceed with this geotechnical investigation was provided by Carmine Zayoun of 11061917 Canada Inc. via our signed work authorization form dated April 2, 2019.

This geotechnical investigation was undertaken concurrently with Phase One and Two Environmental Site Assessments (ESAs) conducted by EXP and the results are reported under separate covers.

Design information regarding the proposed development is provided in the following drawings:

- Plan and profiles of buildings – Drawing Nos. ES02, ES22 and ES23 dated January 22, 2020 and prepared by LaPalme Rheault and acsl Architectes + Associes.

- Site grading plan – Drawing No. C200 dated January 27, 2020 and prepared by EXP.

The drawings indicate the proposed development will consist of two (2) multi-storey buildings with a five (5) level underground parking garage. The two (2) buildings are identified as a 13-storey building (Tower A) and a 12-storey building (Tower B). The elevation of the ground floor for both buildings will be Elevation 75.20 m. The elevation of the lowest floor slab in the parking garage will be at Elevation 58.60 m. The site grades will be raised by 0.9 m.

The fieldwork for the geotechnical investigation was undertaken from April 24 to 30, 2019 and consists of twelve (12) boreholes (BH Nos. 19-01 to 19-12) advanced to auger refusal and termination depths of 5.9 m to 9.6 m below existing grade. A monitoring well was installed in selected boreholes for long-term monitoring of the groundwater level and sampling of the groundwater.

The geotechnical investigation revealed the subsurface conditions consist of fill, sandy silt to silty sand, glacial till underlain by sandstone bedrock (Rockcliffe formation) contacted at 6.5 m to 7.8 m below existing grade (Elevation 68.2 m to 67.6 m). The groundwater level at the site was established at depths ranging between 1.4 m and 6.0 m (Elevation 73.8 m to 69.7 m).

The results of the MASW survey completed at the site indicate that the average seismic shear wave velocity is 1610.4 m/s for footings founded on the sound sandstone bedrock as discussed in Section 8 of this report. Therefore, a site class A for seismic site response can be used in accordance with Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC).

The investigation revealed that some of the overburden soils are susceptible to liquefaction during a seismic event. However, the liquefiable soils along with all remaining overburden soil will be excavated from the building envelopes down to and into the bedrock to accommodate the proposed construction. Therefore, since the liquefiable soils will be removed, liquefiable soils is not a concern for the proposed development.

Based on the available information, the lowest floor slab in the parking garage will be at Elevation 58.6 m and therefore the most appropriate foundation for the proposed buildings is spread and strip footings designed to bear on the sound bedrock free of soil filled seams and below any fractured and weathered zones.

Spread and strip footings founded on the competent sound sandstone bedrock, free of soil filled seams and below the weathered and fractured zones may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 4500 kPa. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement of the structure will be much larger than the recommended value for factored geotechnical resistance at ULS. Therefore, the factored geotechnical resistance at ULS will govern the design.

The lowest level floor slab in the underground parking garage may be designed as a slab-on-grade with perimeter and underfloor drainage systems. The floor slab should be set on a bed of well packed 19 mm clear stone at least 200 mm thick placed on an engineered fill pad at least 300 mm thick placed on the sound sandstone bedrock.

Excavation of the overburden soils may be undertaken using large heavy mechanical equipment capable of removing debris within the fill as well as cobbles and boulders within the fill and glacial till. Excavation of the underlying bedrock will require line drilling and blasting techniques and should be undertaken by a specialized contractor. Pre-condition survey of surrounding buildings and infrastructure (such as roadways and underground services) should be undertaken at the site prior to start of construction as well as conducting vibration monitoring during blasting and rock excavation and construction operations.

Excavations within the soils for the proposed two (2) buildings will likely have to be undertaken within the confines of a shoring system that may consist of steel H soldier pile and timber lagging, steel interlocking sheeting and/or secant pile shoring system. The shoring system may be tied back by anchors grouted into the sound bedrock.

Excavations within the bedrock may be undertaken with near vertical sides subject to review by a geotechnical engineer. The rock face may require support in the form of rock bolts to maintain the integrity of the rock face. In the upper weathered/fractured zones of the bedrock, rock bolts in combination with wire mesh system and/or shotcrete may be required. This will be best established on-site during excavation.

Excavations above the groundwater may be dewatered by conventional sump pumping techniques. Excavations below the groundwater level and the water bearing sandy silt to silty and glacial till are expected to be more problematic and may result in greater water seepage, loss of ground and disturbance of the soils. Under these conditions, it is recommended that these excavations should be undertaken within the confines of a shoring system that is also designed to cut-off groundwater flows towards the excavation and minimize groundwater flows into the shored excavation. In this regard, seepage of groundwater into the shored excavation should still be anticipated but may be removed by collecting the water at low points within the excavation and pumping from sumps.

It is anticipated that all of the fill required for construction will have to be imported to the site and conform to the Ontario Provincial Standard Specification (OPSS) requirements for Granular A, B Type II and Select Subgrade Material (SSM).

The above and other related considerations are discussed in greater detail in the main body of this report.

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1 Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed residential development to be located on the parcel of land identified by the following three (3) lot addresses; 365 Forest Street, 2589 Bond Street and 1420 Richmond Road, Ottawa, Ontario. For purposes of this report, the parcel of land is identified by one (1) address, 365 Forest Street. Authorization to proceed with this geotechnical investigation was provided by Carmine Zayoun on behalf o 11061917 Canada Inc. via our signed work authorization form dated April 2, 2019.

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This geotechnical investigation was undertaken to:

a) Establish the subsurface soil, bedrock and groundwater conditions at the twelve (12) borehole locations drilled throughout the site;

b) Classify the site for seismic site response in accordance with the requirements of the 2012 Ontario Building Code (OBC) and assess the potential for liquefaction of the subsurface soils during a seismic event;

c) Comment on grade-raise restrictions;

d) Make recommendations regarding the most suitable type of foundations, founding depth and bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance at ultimate limit state (ULS) of the founding strata and comment on the anticipated total and differential settlements of the recommended foundation type;

e) Discuss the feasibility of constructing the lowest floor slab as a slab on grade and provide comments regarding perimeter and underfloor drainage systems;

f) Provide lateral earth pressure parameters (for static and seismic conditions) for the subsurface foundation walls of the proposed buildings;

g) Comment on excavation conditions and de-watering requirements during construction;

h) Discuss backfilling requirements and suitability of on-site soils for backfilling purposes;

i) Recommend pavement structure thicknesses for paved surface parking lots and access roads; and,

j) Comment on subsurface concrete requirements and corrosion potential of subsurface soils to buried metal structures/members.

The comments and recommendations given in this report are based on the assumption that the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2 Site Description

The subject site is bounded by Richmond Road on the north side, Bond Street on the south side, Forest Street on the west side and commercial type development on the east side. The 365 Forest Street lot site is occupied by a single storey multi-tenant building. The 2589 Bond Street lot is occupied by a single-storey building. The 1420 Richmond Road lot is currently vacant. The location of the site is shown in Figure 1.

Based on the ground surface elevations at the boreholes, the topography of the site ranges from Elevation 75.74 m to Elevation 74.13 m and gradually slopes downward to the south and southeast towards Bond Street. Ground cover of the site consists asphalt and gravel surfaces with landscaped areas.

3 Geology of Site

3.1 Surficial Geology

The surficial geology map (Map 1506A – Surficial Geology, Ontario-Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1982) indicates that beneath any fill, the site is underlain by medium grained stratified sand with some silt.

3.2 Bedrock Geology

The bedrock geology map (Map 1508A – Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1979) indicates the site is underlain by sandstone bedrock (with shale) of the Rockcliffe formation.

4 Investigation Procedure

4.1 Fieldwork

The fieldwork for the geotechnical investigation was undertaken between April 24 and 30, 2019 and consists of the drilling of twelve (12) boreholes (BH Nos. 19-01 to 19-12) throughout the site advanced to auger refusal and/or termination depths of 5.9 m to 9.6 m below existing grade. The borehole locations are shown in Figure 2.

The borehole locations and elevations were established in the field by a survey crew from EXP and their locations cleared from any underground services by USL-1 cable locators.

The boreholes were drilled with a CME-75 truck-mounted drill rig equipped with continuous flight hollow-stem auger equipment and rock coring capabilities. Standard penetration tests (SPTs) was performed in all the boreholes on a continuous basis and at 0.75 m and 1.5 m depth intervals. The soil samples were retrieved by the split-barrel sampler, in accordance with the American Society for Testing and Materials (ASTM). The presence of the bedrock was proven in selected boreholes by conventional coring techniques using NQ-size core barrel. A record of wash water return, colour of wash and any sudden drop of the drill rods were kept during rock coring operations.

A 32 mm diameter monitoring well with a screened section was installed in selected boreholes for long-term monitoring of the groundwater levels and sampling of the groundwater. The installation configuration of each monitoring well is documented on the respective borehole log. All boreholes were backfilled upon completion of drilling and sampling operations.

4.2 Laboratory Testing Program

All soil samples were visually examined in the field for textural classification, logged, preserved in plastic bags and identified accordingly. Similarly, all rock cores were placed in core boxes, identified and visually examined and logged. On completion of the fieldwork, all the soil samples and rock cores were transported to the EXP laboratory located in the City of Ottawa.

The soil samples and rock cores were visually examined in the laboratory by a senior geotechnical engineer. The soil samples were classified in accordance with the Unified Soil Classification System (USCS). The rock cores were visually examined and logged in accordance with Section 3.2 of the 2006 Canadian Foundation Engineering Manual (Fourth Edition, CFEM) and photographs taken of the rock cores.

A summary of the soil and bedrock laboratory testing program is shown in Table I. The laboratory testing program for selected soil samples and rock cores were undertaken in accordance with ASTM. The testing procedures for the corrosion analysis are referenced in Appendix A.

Table I: Summary of Laboratory Testing Program	
Type of Test	**Number of Tests Completed**
Soil Samples	
Moisture Content Determination	82
Unit Weight Determination	15
Grain Size Analysis	5
Corrosion Analysis (pH, sulphate, chloride and electrical resistivity)	2
Bedrock Cores	
Unit Weight Determination	5
Unconfined Compressive Strength Test	5
Corrosion Analysis (pH, sulphate, chloride and electrical resistivity)	2

4.3 Multi-channel Analysis of Surface Waves (MASW) Survey

A multi-channel analysis of surface waves (MASW) survey was conduced on site on July 30, 2019 by Geophysics (GPR) International Inc. The MASW survey consists of one (1) survey line across the site in a north-south direction in order to measure the shear wave velocity and determine the site classification for seismic site response based on the shear wave velocity measurement. The procedure and report of the MASW survey is shown in Appendix B.

5 Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface conditions and groundwater levels from the boreholes are given on the attached Borehole Logs, Figure Nos. 3 to 14 inclusive. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

Boreholes were drilled to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions.

It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling operations. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The "Note on Sample Descriptions" preceding the borehole logs form an integral part of this report and should be read in conjunction with this report.

A review of the borehole logs indicates the following subsurface conditions with depth and groundwater level measurements.

5.1 Paved Areas - Pavement Structure

Borehole Nos. 19-03, 19-04, 19-06 and 19-07 are located in paved areas of the site. The pavement structure consists of 30 mm and 60 mm thick asphaltic concrete underlain by 150 mm to 250 mm thick granular fill base.

5.2 Unpaved Areas – Gravel Surface

The remaining boreholes are located in unpaved areas consisting of a surficial 100 mm to 500 mm thick granular fill layer.

5.3 Fill

Fill was contacted beneath the pavement structure and surficial granular layer in all the boreholes. The fill extends to depths ranging from 1.4 m to 3.0 m (Elevation 73.6 m to 71.8 m). Borehole No. 19-12 terminated within the fill at 4.4 m depth (Elevation 71.0 m). The fill consists of clayey silty sand to silty sand with gravel. The fill contains rootlets and brick debris. A petroleum odour was noted in the fill samples from Borehole Nos. 19-05 and 19-11. Based on the standard penetration test (SPT) N values of 1 to 16, the fill is in a very loose to compact state. The moisture content of the fill is 4 percent to 30 percent. The unit weight of the fill is 19.1 kN/m^3 to 22.9 kN/m^3.

Grain size analysis of one (1) sample of the fill was conducted and the results are summarized in Table II. The grain size distribution curve is shown in Figure 15.

Table II: Summary of Results from Grain-size Analysis – Fill Sample

Borehole No. - Sample No.	Depth (m)	Grain-size Analysis (%)			Soil Classification (USCS)
		Gravel	Sand	Fines (Silt and Clay)	
BH 19-10 – SS3	1.5 – 2.1	7	47	46	Silty Sand (SM)

Based on a review of the results from the grain size analysis, the fill may be classified as a silty sand (SM) in accordance with the Unified Soil Classification System (USCS).

5.4 Sandy Silt to Silty Sand

The fill in Borehole Nos. 19-01 to 19-03 is underlain by a sandy silt to silty sand layer from 1.4 m to 2.2 m depths (Elevation 73.6 m to 71.9 m). The SPT N values are 4 and 6 indicating the sandy silt to silty sand is in a loose state. The natural moisture content of the sandy silt to silty sand is 22 percent and 23 percent. The natural unit weight of the sandy silt to silty sand is 19.4 kN/m^3.

5.5 Glacial Till

The fill and sandy silt to silty sand layer are underlain by glacial till that extends to depths of 6.5 m to 7.8 m (Elevation 68.2 m to 67.6 m). The glacial till ranges from a clayey silty sand to a silty sand with gravel. The glacial till is a silty clay in Borehole Nos. 19-03 to 19-05 from 2.2 m to 5.3 m depths (Elevation 72.8 m to 69.7 m). The glacial till contains shale fragments, cobbles and boulders. The SPT N values of the cohesionless silty clayey sand till ranges from 1 to 81 indicating the glacial till is in a very loose to very dense state. The SPT N values of the cohesive portion of the silty clay till of 2 to 4 indicates the silty clay till has a soft consistency. The natural moisture content and unit weight of the cohesionless silty clayey sand to silty sand with gravel till is 5 percent to 26 percent and 23.1 kN/m^3 to 23.9 kN/m^3, respectively. The natural moisture content of the silty clay till ranges from 17 percent to 39 percent.

Grain size analysis of four (4) samples of the glacial till were conducted and the results are summarized in Table III. The grain size distribution curves are shown in Figures 16 to 19.

Table III: Summary of Results from Grain-size Analysis – Glacial Till Samples					
Borehole No. - Sample No.	**Depth (m)**	**Grain-size Analysis (%)**			**Soil Classification (USCS)**
		Gravel	**Sand**	**Fines (Silt and Clay)**	
BH 19-02 – SS5	3.8-4.4	13	45	42	Silty Sand (SM)
BH 19-04 – SS6	4.6-5.2	18	40	42	Silty Sand with Gravel (SM)
BH 19-08 – SS5	3.0-3.6	17	47	36	Silty Sand with Gravel (SM)
BH 19-11 – SS6	4.6-5.2	13	51	36	Silty Sand (SM)

Based on a review of the results from the grain size analysis, the glacial till may be classified as a silty sand (SM) to silty sand with gravel (SM) in accordance with the USCS. The glacial till contains shale fragments, cobbles and boulders.

5.6 Sandstone Bedrock

Auger refusal was met in Borehole Nos. 19-01 to 19-05, 19-08, 19-09 and 19-11 at 6.2 m to 7.8 m depths (Elevation 68.4 m to 67.9 m). Conventional core drilling techniques were used to advance Borehole Nos. 19-01, 19-03, 19-08, 19-09 and 19-11 beyond the auger refusal depths to termination depths of 8.0 m to 9.6 m (Elevation 67.0 m to 65.8 m) confirming that auger refusal in these boreholes was met on sandstone bedrock. Photographs of the bedrock cores are shown in Appendix C.

A summary of the auger refusal depth (elevation) on inferred boulders or sandstone bedrock and the bedrock depths (elevations) is shown in Table IV.

Table IV: Summary of Auger Refusal and Bedrock Depths (Elevations) in Boreholes			
Borehole No.	**Ground Surface Elevation (m)**	**Auger Refusal Depth (Elevation) (m)**	**Bedrock Depth (Elevation) (m)**
19-01	74.13	6.5 (67.6)	6.5 (67.6)
19-02	74.37	6.2 (68.2)	-
19-03	75.02	7.0 (68.0)	7.0 (68.0)
19-04	74.98	6.6 (68.4)	-
19-05	74.82	6.7 (68.1)	-
19-08	75.51	7.3 (68.2)	7.3 (68.2)
19-09	75.65	7.6 (68.1)	7.6 (68.1)
19-11	75.71	7.8 (67.9)	7.8 (67.9)

A review of the above table indicates that the depth to the bedrock surface ranges from 6.5 m to 7.8 m below existing grade (Elevation 68.2 m to 67.6 m).

The upper 500 mm to 800 mm of the bedrock appears weathered and highly fractured in Borehole Nos. 19-01, 19-03 and 19-11. The Total Core Recovery (TCR) of the bedrock is 90 percent to 100 percent. The Rock Quality Designation (RQD) ranges from 29 percent to 86 percent indicating the bedrock is of a poor to good quality.

Unit weight determination and unconfined compressive strength tests were conducted on five (5) rock core sections and the results are summarized in Table V. A review of the test results indicates the strength of the rock may be classified as very strong in accordance with the Canadian Foundation Engineering Manual (CFEM), Fourth Edition, 2006.

Table V: Summary of Unconfined Compressive Strength Test Results – Bedrock Cores

Borehole No.-Run No.	Depth (m)	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)	Classification of Rock with respect to Strength (1)
BH 19-01 – Run 2	7.5 - 7.6	24.7	120.6	Very Strong
BH 19-03 – Run 1	7.5 – 7.6	24.5	176.2	Very Strong
BH 19-08 -Run 2	8.7 – 8.8	24.3	125.7	Very Strong
BH 19-09 – Run 1	8.6 – 8.7	25.6	138.2	Very Strong
BH 19-11 - Run 1	7.8 – 7.9	26.7	199.4	Very Strong

Note:

(1) Reference: Fourth Edition – Canadian Foundation Engineering Manual (2006)

5.7 Groundwater Level Measurements

A summary of the groundwater level measurements taken on May 15 and 16, 2019 in the monitoring wells installed in some of the boreholes is shown in Table VI. The monitoring well in Borehole No. 19-01 could not be located during our site visits.

Table VI: Summary of Groundwater Level Measurements

Borehole No. (BH)	Ground Surface Elevation (m)	Date of Measurement (elapsed time in days from date of installation)	Groundwater Depth Below Ground Surface (Elevation), m
BH 19-02	74.13	May 16, 2019 (17 days)	1.9 (72.2)
BH 19-06	75.28	May 15, 2019 (20 days)	2.4 (72.9)
BH 19-07	75.21	May 16, 2019 (22 days)	1.4 (73.8)
BH 19-08	75.51	May 15, 2019 (20 days)	5.7 (69.8)
BH 19-09	75.65	May 15, 2019 (21 days)	6.0 (69.7)
BH 19-10	75.74	May 15, 2019 (20 days)	2.3 (73.4)

Based on a review of the groundwater level measurements the groundwater level ranges from 1.4 m to 6.0 m depths (Elevation 73.8 m to 69.7 m).

Water levels were determined in the boreholes at the times and under the conditions stated in the scope of services. Note that fluctuations in the level of groundwater may occur due to a seasonal variation such as precipitation, snowmelt, rainfall activities, and other factors not evident at the time of measurement and therefore may be at a higher level during wet weather periods.

6 Site Classification for Seismic Site Response and Liquefaction Potential of Soils

6.1 Site Classification for Seismic Site Response

The MASW survey report is shown in Appendix B. The results of the MASW survey indicate that the average seismic shear wave velocity is 1610.4 m/s for footings founded on the sound sandstone bedrock as discussed in Section 8 of this report. This will result in a Class A site class for seismic site response in accordance with Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC),

6.2 Liquefaction Potential of Soils

The geotechnical investigation revealed that some of the overburden soils are susceptible to liquefaction during a seismic event. Since the construction of the five (5) level underground parking garage below both towers would require the removal of all overburden soils down and into the bedrock, inclusive of the potential liquefiable soils, liquefiable soils at the site is not an issue for the proposed development.

7 Grade Raise Restrictions

A review of the site grading plan, Drawing No. C200 (dated January 27, 2020) prepared by EXP revealed that a site grade raise of 0.9 m will be realized at the site as part of the proposed development.

Since the subsurface soils consist of cohesionless sandy soils that are not susceptible to consolidation settlement, there is no restriction to raising the grades at the site from a consolidation perspective. Therefore, the proposed 0.9 m site grade raise as presented in the January 27, 2020 EXP site grading plan (Drawing No. C200) is considered acceptable from a geotechnical point of view.

8 Foundation Considerations

The borehole information indicates the subsurface conditions at the site consist of fill, sandy silt to silty sand, glacial till underlain by sandstone bedrock (Rockcliffe formation) contacted at 6.5 m to 7.8 m below existing grade (Elevation 68.2 m to 67.6 m). Based on a review of the groundwater level measurements the groundwater level ranges from 1.4 m to 6.0 m depths (Elevation 73.8 m to 69.7 m).

Since the proposed lowest floor slab of the parking garage will be Elevation 58.6 m, the proposed buildings may be supported on spread and strip footings designed to bear on the competent sound sandstone bedrock free of soil filled seams and below all weathered and fractured zones. It has been assumed that all the overburden soils will be removed from the building envelopes down and into the bedrock.

Spread and strip footings founded on the competent sound sandstone bedrock, free of soil filled seams and below the weathered and fractured zones, may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 4500 kPa. The factored ULS value includes a resistance factor of 0.5. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement of the structure will be much larger than the recommended value for factored geotechnical resistance at ULS. Therefore, the factored geotechnical resistance at ULS will govern the design.

Settlements of footing designed for the above recommended factored geotechnical resistance at ULS and properly constructed are expected to be less than 10 mm.

All the footing beds should be reviewed by a geotechnical engineer to ensure that the bedrock subgrade is capable of supporting the design ULS value. Where fractured rock is encountered, sub-excavation may be undertaken to the underlying more competent bedrock. Alternatively, the footings may be redesigned to a reduced factored geotechnical resistance at ULS. Any sub-excavation which extends below the underside of the footings would have to be raised using 15 MPa lean mix concrete. Also, if the surface of the excavated bedrock is not level, the bedrock surface may be levelled by the placement of concrete.

The resistance to sliding of the building footings will be provided by friction between the footing concrete and the sound sandstone bedrock. The unfactored ULS coefficient of friction is 0.70.

A minimum of 1.5 m of earth cover should be provided to exterior footings of heated structures to protect them from damage due to frost penetration. The frost cover should be increased to 2.1 m for unheated structures if snow will not be removed from the vicinity of the footing and 2.4 m of earth cover if snow will be removed from the vicinity of the footing. In areas where earth cover will be less than the required, rigid insulation may be used to protect the footings. Alternatively, a combination of earth cover and rigid insulation may also be used to protect the footings. For this project it is anticipated that the required earth cover for the footings of the proposed buildings will be satisfied, since the footings are anticipated to be at depths greater than 1.5 m below the final grade.

The recommended factored geotechnical resistances at ULS have been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field monitoring provided by an experienced geotechnical engineer to validate the information for use during the construction stage.

9 Floor Slab Construction and Drainage Requirements

The lowest floor slab of the parking garage below the proposed two (2) tower buildings may be designed as a slab-on-grade. The slab-on-grade should be set on a bed of well packed 19 m clear stone at least 200 mm thick placed on an engineered fill pad at least 300 mm thick placed on the sound sandstone bedrock. The required engineered fill pad should comprise of Ontario Provincial Standard Specification (OPSS)1010 Granular B Type II placed on top of the bedrock in 300 mm thick lifts and each lift compacted to 98 percent standard Proctor maximum dry density (SPMDD). The clear stone will prevent the capillary rise of moisture from the underlying soil to the floor slab. Adequate saw cuts should be provided in the floor slab to control cracking.

It is recommended that perimeter as well as underfloor drains should be provided for the slab-on-grade of the parking garage. The underfloor drainage system may consist of 100 mm diameter perforated pipe or equivalent placed in parallel rows at 5 m to 6 m centres and at least 300 mm below the underside of the floor slab. The drains should be set on 100 mm of pea-gravel and covered on top and sides with 150 mm of pea-gravel and 300 mm of CSA Fine Concrete Aggregate. The CSA Fine Concrete Aggregate may be replaced by an approved porous geotextile membrane, such as Terrafix 270R or equivalent. The perimeter drains may also consist of 100 mm diameter perforated pipe set on the footings and surrounded with 150 mm of pea-gravel and 300 mm of CSA Concrete Aggregate. The perimeter and underfloor drains should be connected to separate sumps so that at least one system would be operational should the other fail.

The finished exterior grade should be sloped away from the buildings to prevent surface ponding close to the exterior walls.

10 Lateral Earth Pressure Against Subsurface Walls

10.1 Lateral Earth Pressures Due to Backfill Against Subsurface Walls

If the space between the subsurface walls and the rock face is to be backfilled, the subsurface walls will be subjected to lateral static earth pressure as well as lateral dynamic earth pressure during a seismic event. The lateral static earth pressure that the subsurface walls would be subjected to may be computed from equations (i) and (ii) and the lateral dynamic earth force from equation (iii) given below.

The equations given below assume that the backfill against the subsurface walls will be free-draining granular material and that subsurface drains will be provided to prevent build-up of hydrostatic pressure. Equation (i) will be applicable to the portion of the subsurface wall in the overburden soil. Equation (ii) will be applicable to the portion of the subsurface wall in the bedrock where the earth pressure will be considerably reduced due to the narrow backfill between the subsurface wall and the rock face resulting in an arching effect (Spangler & Handy, 1984). The weight of the overburden soil and any surcharge load stress (such as traffic load at ground surface and foundations of existing adjacent buildings) should be considered as surcharge when computing lateral pressure using equation (ii).

The lateral static earth pressure against the subsurface walls may be computed from the following equation:

$$P = K_0 (\gamma h + q) \quad \text{(i)}$$

where

P = lateral earth pressure acting on the subsurface wall; kN/m^2

K_0 = lateral earth pressure coefficient for 'at rest' condition for Granular B Type II backfill material = 0.50

γ = unit weight of free draining granular backfill; OPSS Granular B Type II = 22 kN/m^3

h = depth of point of interest below top of backfill, m

q = surcharge load stress, kPa

Lateral static earth pressure due to narrow earth backfill between subsurface wall and rock face at depth z; σ_n:

$$\sigma_n = \frac{\gamma B}{2\tan\delta}\left(1 - e^{-2k\frac{z}{B}\tan\delta}\right) + kq \text{ -------------- (ii)}$$

where

γ = unit weight of backfill = 22 kN/m^3

B = backfill width (m)

z = depth from top of wall (m)

δ = friction angle between the backfill and wall and rock (assumed to be equal) = 17 degrees

k = lateral earth pressure coefficient for 'at rest' condition = 0.50

q = surcharge pressure including pressures from overburden soil, traffic at ground surface and foundations from existing adjacent buildings (kPa)

The lateral dynamic (seismic) thrust may be computed from the equation given below:

$$\Delta_{Pe} = \gamma H^2 \frac{a_h}{g} F_b \quad \text{.......} \quad \text{(iii)}$$

where

Δ_{Pe}	=	dynamic thrust in kN/m of wall
H	=	height of wall, m
γ	=	unit weight of free draining granular backfill; OPSS Granular B Type II = 22 kN/m^3
$\frac{a_h}{g}$	=	seismic coefficient = 0.32
F_b	=	thrust factor = 1.0

The dynamic thrust does not take into account the surcharge load. The resultant force acts approximately at 0.63H above the base of the wall.

All subsurface walls should be properly waterproofed.

Where the basement walls will be poured against the bedrock or temporary shoring, vertical drainage board must be installed on the face of the excavation wall or temporary shoring to provide necessary drainage. Vertical drainage board such as Alidrain, Geodrain, Miridrain or equivalent may be used for this purpose. A schematic of the location of the drainage board is shown in Figure 20. Full coverage using drainage boards can be considered to minimize the risk of water penetration through the subsurface basement walls.

Where the upper portion of the subsurface basement wall is backfilled with granular material, the vertical drainage board should extend into the backfill to provide drainage of the backfill. The top of the drainage board should be covered with a fabric filter to prevent the loss of overlying soil into the drainage board. The vertical drainage board should be connected to a solid discharge pipe that passes through the foundation wall and outlets to a solid pipe inside the building that leads to a sump, as shown in Figure 20. The solid pipe inside the building should be connected to a separate sump from the sumps used for the the perimeter and underfloor drains, so that this system would be operational should one of the other drainage systems fail.

11 Excavations and De-Watering Requirements

Excavations for the proposed development are expected to extend to approximate 16.0 m to 18.0 m depth below existing grade with excavation depths in the soil to approximately 7.0 m to 8.0 m and to depths of approximately 9.0 m to 10.0 m below the bedrock surface. The excavations will extend to approximately 11.0 m to 15.0 m depths below the groundwater level.

11.1 Overburden Soil Excavation

The excavations in the soil may be undertaken using large heavy mechanical equipment capable of removing debris within the fill as well as cobbles and boulders within the fill and underlying glacial till.

All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils on site are considered to be Type 4 and as such must be cut back at 3H:1V from the bottom of the excavation. It is anticipated that due to the significant depth of the excavation and the proximity of the excavation to existing buildings and infrastructure, the excavations will likely have to be undertaken within the confines of a shoring system. The shoring system may consist of steel H soldier pile and timber lagging system, interlocking sheeting system and/or secant pile shoring system.

The type of shoring system required would depend on a number of factors including:

- Proximity of the excavation to existing structures and infrastructure;
- Type of foundations of the existing adjacent buildings and the difference in founding levels between the foundations of new buildings and existing adjacent buildings; and
- The subsurface soil, bedrock and groundwater conditions.

A conventional shoring system consisting of soldier pile and timber lagging is more flexible compared to the interlocking steel sheeting system and the secant pile shoring system. In areas where there is concern for lateral yielding of the soils and the potential of settlement of nearby structures and infrastructure, the use of a steel interlocking sheeting system or secant pile system can be considered. The steel interlocking sheeting and secant pile system also provide cut-off to groundwater flows into the excavation. In areas where the potential of settlement of the nearby structures is low, soldier pile and timber lagging system may be used. The shoring system will require lateral restraint provided by tiebacks consisting of rock anchors. Due to the presence of cobbles and boulders in the subsurface soils, pre-drilling may be required for the installation of the soldier piles and a thickened section may be required for the interlocking steel sheeting system.

The need for a shoring system, the appropriate shoring system and the design and installation of the shoring system should be determined/conducted by a professional engineer experienced in shoring design and by a contractor experienced in the installation of shoring systems. The shoring system should be designed and installed in accordance with OHSA and the 2006 CFEM (Canadian Foundation Engineering Manual (Fourth Edition)).

11.1.1 Soldier Pile and Timber Lagging System

A conventional steel H soldier pile and timber lagging shoring system must be designed to support the lateral earth pressure given by the expression below:

P = k (γh + q)

where P = the pressure, at any depth, h, below the ground surface
k = applicable earth pressure coefficient; active lateral earth pressure coefficient = 0.33
'at rest' lateral earth pressure coefficient = 0.50
γ = unit weight of soil to be retained, estimated at 21 kN/m^3
h = the depth, in metres, at which pressure, P, is being computed
q = the equivalent surcharge acting on the ground surface adjacent to the shoring system

The pressure distribution assumes that drainage is permitted between the lagging boards and that no build-up of hydrostatic pressure may occur.

The shoring should be designed using appropriate 'k' values depending on the location of any settlement-sensitive infrastructure (roadways and underground services) and building structures. The traffic loads on the streets should be considered as surcharge. It may be necessary to toe the soldier piles into the sound rock below the soils. For guidance, if there is room to permit at least a 1.0 m of rock ledge around the perimeter of the excavation, the soldier piles could be toed into the upper levels of the rock provided that a rock bolt and plate arrangement is installed on the rock face to support the toe. The rock bolt should be designed to take the full toe pressure.

11.1.2 Secant Pile Shoring System

Groundwater cut-off shoring systems such as steel interlocking sheeting and secant pile shoring system should be designed to resist 'at rest' lateral earth thrust in addition to the hydrostatic thrust as given by the expression below:

$$P_0 = K_O\, q\,(h_1 + h_2) + \frac{1}{2} K_0\, \gamma\, h_1^2 + K_0\, \gamma\, h_1 h_2 + \frac{1}{2} K_0\, \gamma'\, h_2^2 + \frac{1}{2} \gamma_w\, {h_2}^2$$

Where:

P_0 = 'at rest' earth and water thrusts acting against secant pile wall (kN/m)

K_0 = 'at rest' lateral earth pressure coefficient = 0.50

q = surcharge acting adjacent to the excavation (kPa)

h_1 = height of shoring from the ground surface to groundwater table (m)

h_2 = height of shoring from groundwater table to the bottom of excavation (m)

γ = estimated unit weight of the soil to be retained = 21 kN/m^3

γ' = submerged unit weight of soil = 11 kN/m^3

γ_w = unit weight of water = 9.8 kN/m^3

If the secant walls are incorporated into the design of the structures, they should also be designed to resist soil dynamic thrust and hydrodynamic thrust due to a seismic event.

Secant pile walls consist of overlapping concrete piles that form a stronger watertight barrier compared with the sheeting system. They can be constructed with conventional drilling methods. Secant pile walls typically include both reinforced primary and un-reinforced secondary piles. The primary piles overlap the secondary piles, with secondary piles essentially acting as concrete lagging. The reinforcement in the primary piles generally consists of steel reinforcing bar cages or steel beams. The result is a continuous intersecting line of concrete piles that are placed before any excavation is performed.

11.1.3 Additional Comments

The shoring system as well as adjacent settlement sensitive structures and infrastructure should be monitored for movement (deflection) on a periodic basis during construction operations.

Many geologic materials deteriorate rapidly upon exposure to meteorological elements. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from moisture, desiccation, and frost action throughout the course of construction.

11.1.4 Rock Anchors

The shoring system will require lateral restraint by tiebacks in the form of grouted rock anchors. It is noted that if permission is not received from adjacent property owners for the installation of tiebacks, the shoring system may have to be supported by cross bracing or the use of rakers on the inside of the excavation.

Grouted rock anchors may fail in one or more of the following manners:

a) Failure of the grout/tendon bond;
b) Failure of the steel tendon or top anchorage;
c) Failure of the rock/grout bond; or
d) Failure of the rock mass.

Failure modes a) and b) require review by the structural engineer. Geotechnical related failure modes c) and d) for vertical grouted anchors are discussed below:

Failure of the rock/grout bond:

- The unfactored ultimate limit state (ULS) bond strength of the sound rock/grout interface may be taken as 1500 kPa (1.5 MPa). For semi-empirical analysis, the factored ULS bond strength is 450 kPa, using a resistance factor of 0.3. The factored ULS bond strength may be taken as 600 kPa and includes a resistance factor of 0.4 based on conducting proof test on all anchors. The unconfined compressive strength of the grout is assumed to be 30 MPa.

Failure of the rock mass:

- A 90-degree apex (45 degrees from the vertical) should be used to calculate the rock volume using the theoretical rock cone. The apex is located at the middle of the bonded length.
- The submerged unit weight of the bedrock equal to 15.0 kN/m^3 should be used in the calculations.
- The weathered and highly fractured zone of the sandstone bedrock should not be included in the bond length when calculating the anchor capacity. The recommended minimum unbonded length is 3.0 m and should include the weathered and highly fractured zone of the bedrock.
- The minimum bonded length is 3 m.
- For groups of rock anchors, the anchor group resistance to rock mass failure should be reduced to reflect the theoretical rock cone overlap.

Pre-production or design performance tests on selected rock anchors should be conducted in accordance with the 2006 CFEM. Proof load tests should be conducted on all anchors and should be in accordance with the 2006 CFEM.

11.2 Rock Excavation

Excavation of the underlying bedrock will require line drilling and blasting techniques.

11.2.1 Rock Support

Excavations within the bedrock may be undertaken with near vertical sides subject to review by a geotechnical engineer. The rock face may require support in the form of rock bolts to maintain the integrity of the rock face. In the upper weathered/fractured zones of the bedrock, rock bolts in combination with wire mesh system and/or shotcrete may be required. The excavation will have to be undertaken in a staged approach with the rock excavated in a pre-determined depth interval (for example every 3 m). The exposed rock face in each stage will have to be examined by a geotechnical engineer to determine the number of rock bolts required. The rock bolt system should be installed in this manner to the bottom of the excavation. The design of the rock bolts should be in accordance with the design of the rock anchors in Section 11.1.1.4 of this report.

11.2.2 Vibration Control

The vibration limits for blasting should be in accordance with City of Ottawa Special Provisions (SP No. 1201).

It is recommended that a pre-construction survey of adjacent building(s) and infrastructure be undertaken prior to any earth (soil) and rock excavation work as well as vibration monitoring during excavation, blasting and construction operations. Prior to the commencement of blasting, a detailed blast methodology should be submitted by the Contractor.

11.3 De-Watering Requirements and Impact of Groundwater Lowering on Adjacent Structures

Excavations above the groundwater may be dewatered by conventional sump pumping techniques. Excavations below the groundwater level and the water bearing sandy silt to silty and glacial till are expected to be more problematic and may result in greater water seepage, loss of ground and disturbance of the soils. Under these conditions, it is recommended that these excavations should be undertaken within the confines of a shoring system that is also designed to cut-off groundwater flows towards the excavation and minimize groundwater flows into the shored excavation. In this regard, seepage of groundwater into the shored excavation should still be anticipated but may be removed by collecting the water at low points within the excavation and pumping from sumps. In areas of high infiltration, a higher seepage rate should be anticipated and the need for high capacity pumps to keep the excavation dry should not be ignored.

It has been assumed that the excavation depth at the site will be approximately 18.0 m and would necessitate groundwater removal from the site. It is noteworthy to mention that new legislation came into force in Ontario on March 29, 2016 to regulate groundwater takings for construction dewatering purposes. Prior to March 29, 2016, a Category 2 Permit to Take Water (PTTW) was required from the Ontario Ministry of the Environment and Climate Change (MOECC) for groundwater takings related to construction dewatering, where taking volumes in excess of 50 m^3/day, but less than 400 m^3/day, and the taking duration was no more than 30 consecutive days. The new legislation replaces the Category 2 PTTW for construction dewatering with a new process under the Environmental Activity and Sector Registry (EASR). The EASR is an on-line registry, which allows persons engaged in prescribed activities, such as water takings, to register with the MOECC instead of applying for a PTTW.

To be eligible for the new EASR process, the construction dewatering taking must be less than 400 m^3/day under normal conditions. The water taking can be groundwater, storm water, or a combination of both. It should be noted that the 30-consecutive day limit on the water taking under the old Category 2 PTTW process has been removed in the new EASR process. Also, it should be noted that the EASR process requires two technical studies be prepared by a Qualified Person, prior to any water taking. These studies include a Water Taking Report, which provides assurance that the taking will not cause any unacceptable impacts, and a Discharge Plan, which provides assurance that the discharge will not result in any adverse impacts to the environment. EXP has qualified persons who can prepare these types of reports, if required. A significant advantage of the new EASR process over the former Category 2 PTTW process, is that the groundwater taking may begin immediately after completing the on-line registration of the taking and paying the applicable fee, assuming the accompanying technical studies have been completed. The former PTTW process typically took more than 90 days, which had the potential to impact construction schedules.

Although this investigation has estimated the groundwater levels at the time of the fieldwork, and commented on dewatering and general construction problems, conditions may be present which are difficult to establish from standard boring and excavating techniques and which may affect the type and nature of dewatering procedures used by the contractor in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile, thin layers of soil with large or small permeabilities compared with the soil mass, etc. Only carefully controlled tests using pumped wells and observation wells will yield the quantitative data on groundwater volumes and pressures that are necessary to adequately engineer construction dewatering systems.

12 Impact of Groundwater Lowering at the Site on Municipal Services in Adjacent Roadways and Adjacent Structures

The impact of groundwater lowering at the site on the municipal underground services was assessed by reviewing municipal service drawings along the sections of Forest Avenue, Bond Street and Richmond Road that border the site and conducting settlement calculations. The settlement calculations were based on the assumption that the subsurface soil conditions encountered in the boreholes on site are similar to the subsurface soil conditions in the assessed sections of the roadways noted above. The settlement calculations indicated the impact of groundwater lowering at the site on the municipal services is negligible.

The impact of groundwater lowering at the site on adjacent surrounding building structures will have to be assessed. Information required for this assessment includes the location or distance of the existing buildings to the excavation at the site, the type of foundation and depth of foundation of the existing buildings.

13 Backfilling Requirements and Suitability of On-Site Soils for Backfilling Purposes

The soils to be excavated from the site will comprise topsoil, silty sand with gravel fill, sandy silt to silty sand, glacial till and sandstone bedrock. From a geotechnical perspective, these soils and the sandstone bedrock are not considered suitable for reuse as backfill material in the interior or exterior of the building and should be discarded. It may be possible to use portions of the fill, sandy silt to silty sand and glacial till above the groundwater level in landscaped areas, subject to further examination and testing at time of construction. However, these soils are subject to moisture absorption due to precipitation and must be protected at all time from the elements.

Therefore, it is anticipated that all the material required for backfilling purposes in the interior and exterior of the proposed buildings and in the underground service trenches will need to be imported and should preferably conform to the following specifications:

- Engineered fill, underfloor fill including backfilling in service trenches inside the building - OPSS 1010 (as amended by SSP110S13) for Granular B Type II (50 mm minus) placed in 300 mm thick lifts with each lift compacted to 98 percent SPMDD beneath the floor slab;
- Backfill against exterior subsurface walls - OPSS 1010 Granular B Type II placed in 300 mm thick lifts and compacted to 95 percent SPMDD;
- Trench backfill outside building area, and fill placement to subgrade level for pavement - OPSS 1010 Select Subgrade Material (SSM), free of organics, debris and with a natural moisture content within 2 percent of the optimum moisture content. It should be placed in 300 mm thick lifts compacted to minimum 95 percent SPMDD; and,
- Landscaped areas - Clean fill that is free of organics and deleterious material and is placed in 300 mm thick lifts with each lift compacted to 92 percent of the SPMDD.

To minimize settlement of the pavement structure over services trenches, the trench backfill material within the frost zone, to 1.8 m depth below final grade, should match the existing material along the trench walls to minimize differential frost heaving of the subgrade soil, provided this material is compactible. Otherwise, frost tapers may be required.

14 Access Roads and Parking Areas

The subgrade for the parking lots and access roads at the site is anticipated to consist of imported granular fill (compacted to 95 percent SPMDD) used to raise the grades at the site. Pavement structure thicknesses required for light and heavy-duty traffic on the access roads and in parking lots were computed and are shown in Table VII. The pavement structure thicknesses are based upon an estimate of the properties of the imported granular fill subgrade and functional design life of eight (8) to ten (10) years. The proposed functional design life represents the number of years to the first rehabilitation., assuming regular maintenance is carried out.

Table VII: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Light Duty Parking Areas	Heavy Duty Parking Areas and Access Roads
Asphaltic Concrete (PG 58-34)	92% to 97 % MRD	65 mm – SP12.5 Cat B or HL3	40 mm – 12.5 Cat B/HL3 50 mm – 19 Cat B/HL8
Granular A Base (OPSS 1010) (crushed limestone)	100% SPMDD	150 mm	150 mm
Granular B Sub-base, Type II (OPSS 1010)	100% SPMDD	300 mm	450 mm

SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698-12e2
MRD denotes Maximum Relative Density, ASTM D2041

The foregoing design assumes that construction is carried out during dry periods and that the subgrade is stable under the load of construction equipment. If construction is carried out during wet weather, and heaving or rolling of the subgrade is experienced, additional thickness of granular material and/or geotextile may be required.

Additional comments on the construction of the access roads and parking lots are as follows:

(1) As part of the subgrade preparation, the proposed access road and parking lot areas should be stripped of unsuitable fill and other obviously unsuitable material. Fill required to raise the grades to design elevations should be organic-free and at a moisture content which will permit compaction to the densities indicated. After all the underground services have been installed, the subgrade should be properly shaped, crowned and proofrolled with a heavy roller in the full-time presence of a representative of this office. Any soft or spongy subgrade areas detected should be subexcavated and properly replaced with suitable approved backfill compacted to 95 percent SPMDD.

(2) The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved. The need for adequate drainage cannot be over-emphasized. Sub-drains must be installed on both sides of the proposed access roadways. In the parking areas, they should be installed at low points and should be continuous between catch basins to intercept excess surface and subsurface moisture and to prevent subgrade softening. This will ensure no water collects in the granular course, which could result in pavement failure during the spring thaw. The location and extent of subdrainage required within the paved areas should be reviewed by this office in conjunction with the proposed site grading.

(3) To minimize the problems of differential movement between the pavement and catchbasins/ manhole due to frost action, the backfill around the structures should consist of free-draining granular preferably conforming to OPSS Granular B Type II material. Weep holes should be provided in the catchbasins and manholes to facilitate drainage of the granular fill.

(4) The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum cross fall of 2 percent) to provide effective surface drainage towards catch basins. Surface water should not be allowed to pond adjacent to the outside edges of paved areas.

(5) The granular materials used for pavement construction should conform to Ontario Provincial Standard Specifications (OPSS) for Granular A and Granular B Type II and should be compacted to 100 percent SPMDD. The asphaltic concrete and its placement should meet OPSS 1151 requirements. It should be placed and compacted to OPSS 311 and 313.

It is recommended that EXP be retained to review the final pavement structure design and drainage plans prior to construction to ensure that they are consistent with the recommendations of this report.

15 Corrosion Potential of Subsurface Soils and Bedrock

Chemical tests limited to pH, sulphate, chloride and resistivity were undertaken on two (2) soil samples (fill and glacial till) and two (2) sections of bedrock cores. A summary of the results is shown in Table VIII. The laboratory certificate of analysis is shown in Appendix A.

Table VIII: pH, Sulphate, Chloride and Resistivity Test Results on Soil Samples and Bedrock Cores

Borehole No. (BH); Sample/Run No. (SS)	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
Soil Samples					
BH 19-03: SS5	3.0 – 3.6	8.00	0.0050	0.0120	2480
BH 19-12: SS4	2.3 – 2.9	8.17	0.0121	0.0114	1950
Bedrock Cores					
BH 19-01: Run 1	6.5 – 6.6	8.55	0.0042	0.0025	2490
BH 19-11: Run 1	7.8 – 7.9	8.59	0.0026	0.0021	2110

The sulphate test results indicate the soil samples and bedrock core sections have a negligible sulphate attack on subsurface concrete. The concrete should be in accordance with CSA A.23.1-14.

Based on a review of the resistivity test results, the soil samples and bedrock core sections are considered to be corrosive to mildly corrosive to bare steel as per the National Association of Corrosion Engineers (NACE). Appropriate measures should be undertaken to protect buried steel elements from corrosion.

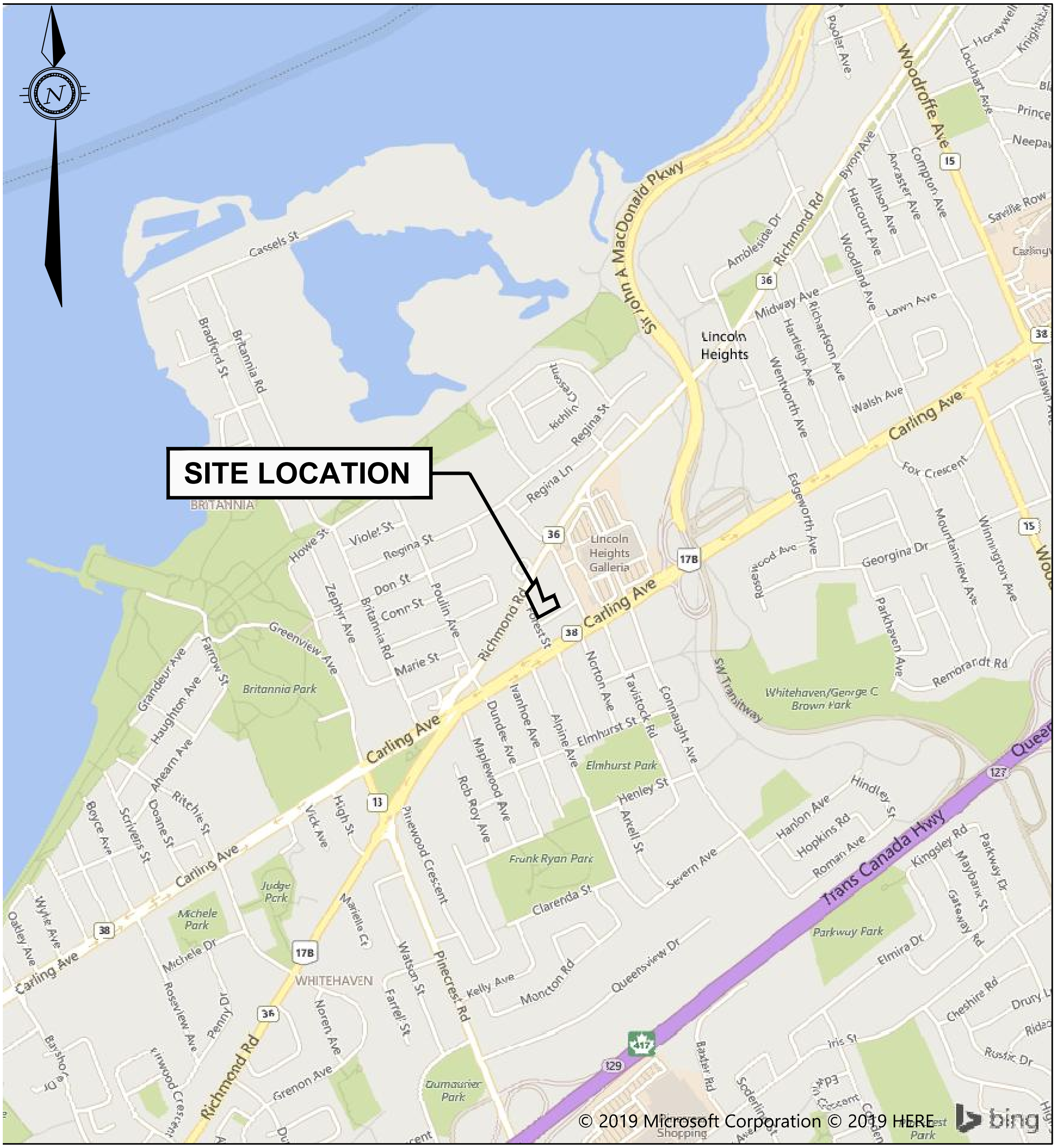
16 General Comments

The comments given in this report are intended only for the guidance of the design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should in this light, decide on their own investigations, as well as their own interpretation of the factual borehole results to draw their own conclusions as to how the subsurface conditions may affect them.

The information contained in this report is not intended to reflect on environmental aspects of the soils. Reference is made to the Phase One and Two Environmental Site Assessments completed by EXP and reported under separate covers.

We trust this report will be satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Figures



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Ottawa, ON K2B 8H6, Canada

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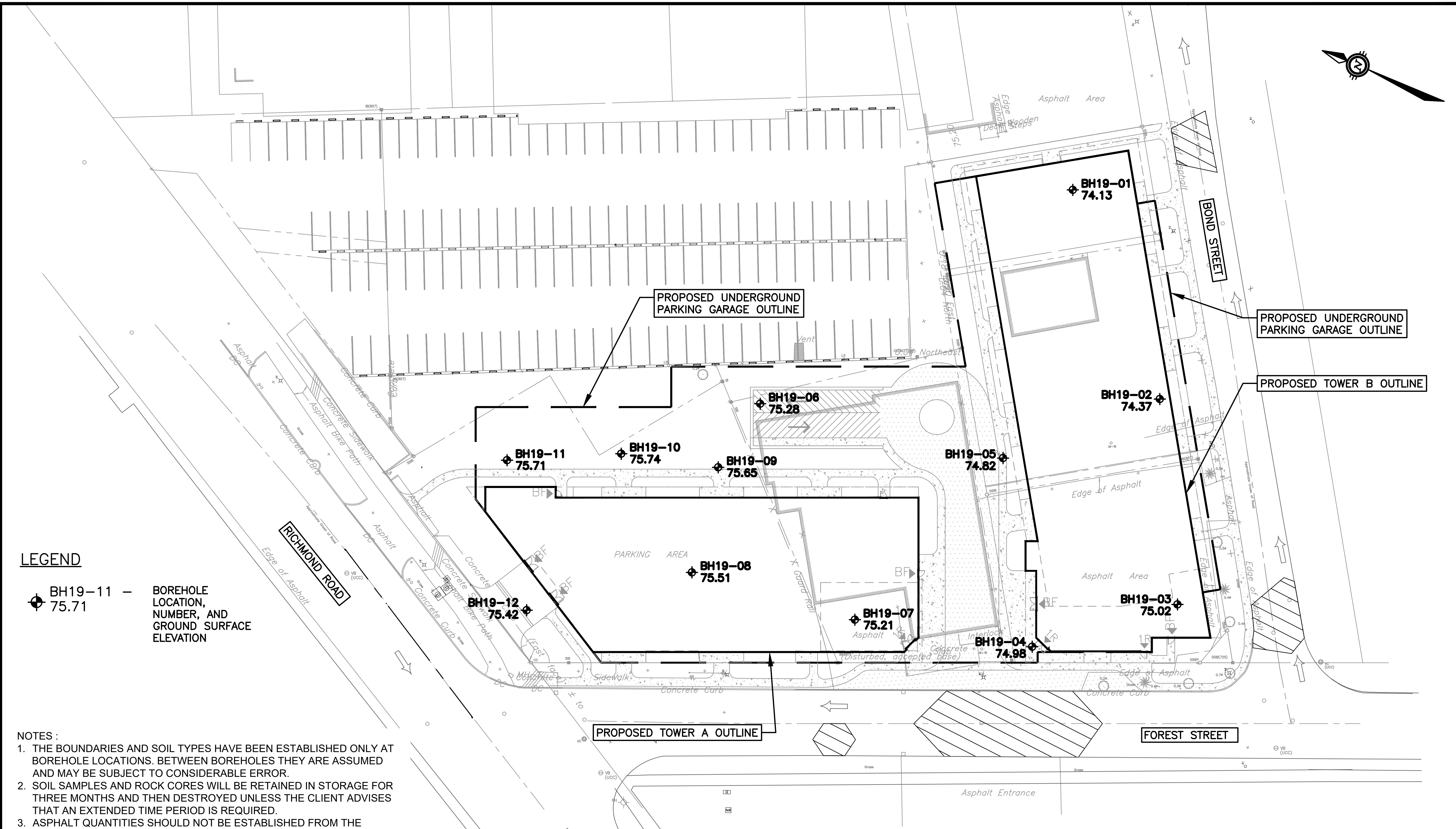

LEGEND

BH19-11 – 75.71 — BOREHOLE LOCATION, NUMBER, AND GROUND SURFACE ELEVATION

NOTES :

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND ROCK CORES WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. ASPHALT QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. BASE PLAN OBTAINED FROM ARCHITECTURAL SITE PLAN.

0 5m 10m 20m
HORIZONTAL 1:500

exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6
www.exp.com

DESIGN EXP	365 FOREST STREET OTTAWA, ONTARIO	SCALE 1:500
DRAWN MNP		
DATE JAN 2020	BOREHOLE LOCATION PLAN	SKETCH NO FIG 2
FILE NO 252625		

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Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Societv for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by **exp** Services Inc. also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Log of Borehole BH 19-01

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Date Drilled: 'April 30, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

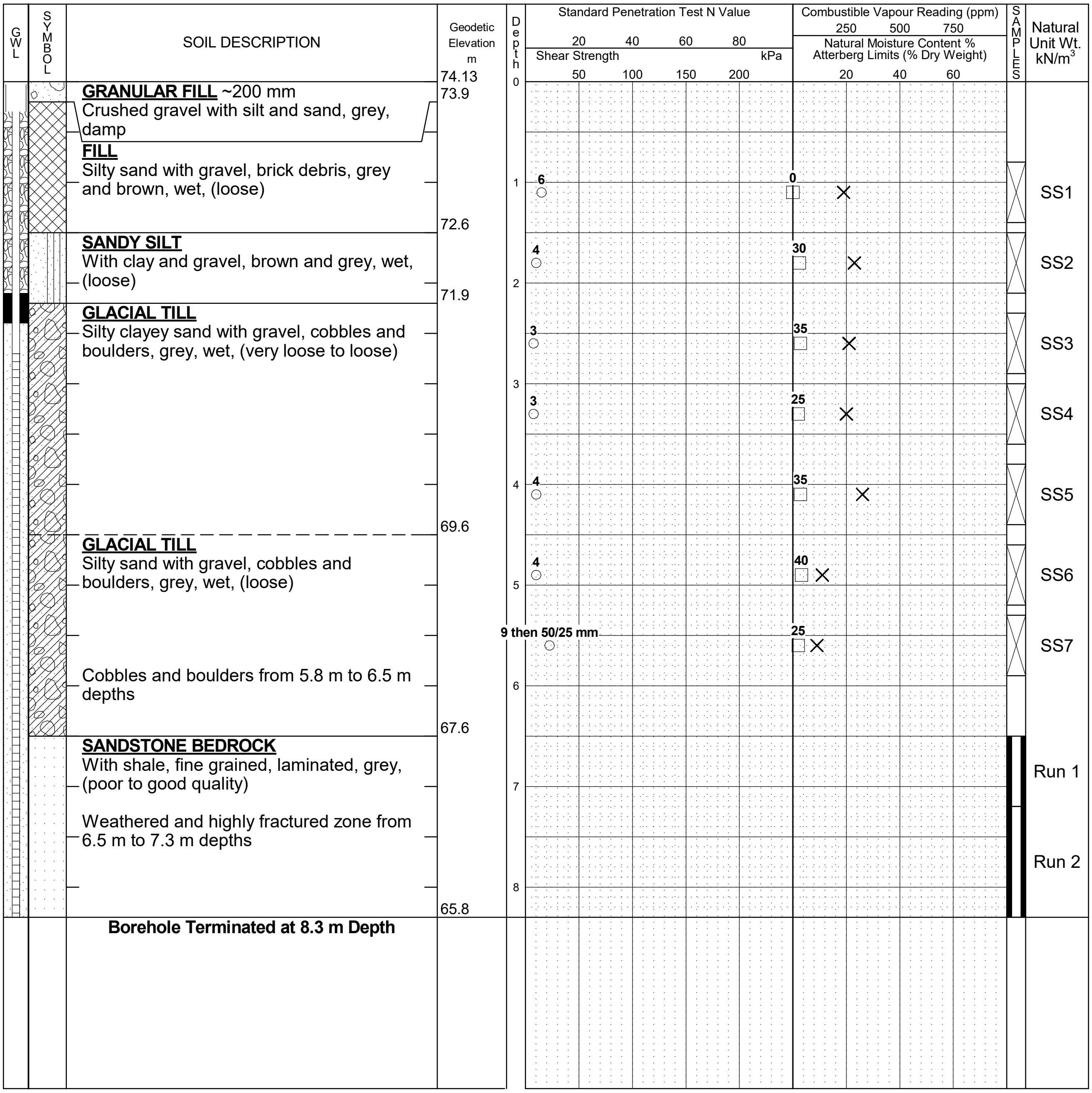
Datum: Geodetic Elevation

Logged by: M.L.   Checked by: I.T.

Figure No. 3

Page. 1 of 1

Symbol		Symbol	
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample		Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	|—○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	N/A	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.5 - 7.2	90	29
2	7.2 - 8.3	100	84

Log of Borehole BH 19-02

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 4

Page. 1 of 1

Date Drilled: 'April 29, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Split Spoon Sample
Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Shear Strength by Vane Test

Combustible Vapour Reading
Natural Moisture Content
Atterberg Limits
Undrained Triaxial at % Strain at Failure
Shear Strength by Penetrometer Test

Depth (m)	Soil Description	Geodetic Elevation (m)	SPT N Value	Combustible Vapour Reading (ppm)	Samples	Natural Unit Wt. kN/m³
0		74.37				
0–0.5	**GRANULAR FILL** ~500 mm Crushed gravel with silt and sand, grey, damp	73.9				
0.5–1.4	**FILL** Silty sand with gravel, brown and grey, wet, (loose)	73.0	6	5	SS1	20.6
1.4–2.2	**SANDY SILT** With clay and gravel, grey, wet, (loose)	72.47 72.2	8	10	SS2	19.4
2.2–3.7	**GLACIAL TILL** Silty clayey sand with gravel, cobbles, and boulders, grey wet, (very loose to loose)	70.7	5 1	20 25	SS3 SS4	
3.7–6.2	**GLACIAL TILL** Silty sand with gravel, cobbles and boulders, grey, wet, (very loose to loose)	68.2	4 3 50 for 75 mm	20 15	SS5 SS6 SS7	
	Auger Refusal at 6.2 m Depth					

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	5.2	
11 days	1.8	
17 days	1.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 19-03

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 5

Page. 1 of 1

Date Drilled: 'April 30, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Legend	
Split Spoon Sample	☒
Auger Sample	◧
SPT (N) Value	○
Dynamic Cone Test	—
Shelby Tube	■
Shear Strength by Vane Test	+ S
Combustible Vapour Reading	□
Natural Moisture Content	×
Atterberg Limits	├──○
Undrained Triaxial at % Strain at Failure	⊕
Shear Strength by Penetrometer Test	▲

Geodetic Elevation (m)	Depth (m)	Soil Description	SPT N Value	Combustible Vapour Reading (ppm)	Samples	Natural Unit Wt. (kN/m³)
75.02	0					
74.9		**ASPHALTIC CONCRETE** ~60 mm				
74.8		**GRANULAR FILL (BASE)** ~150 mm Crushed gravel with silt and sand, grey, damp	10	0	SS1	
		FILL Silty sand and gravel, dark brown, moist, (compact)	15	0	SS2	20.5
73.6		**SANDY SILT TO SILTY SAND** Brown, wet, (loose)	6	5	SS3	
72.8		**GLACIAL TILL** Silty clayey sand with gravel, cobbles and boulders, grey, wet, (loose)	4	5	SS4	23.1
			8	15	SS5	
71.3		**GLACIAL TILL** Silty clay with gravel, grey, wet, (soft)	3	20	SS6	
			2	15	SS7	
69.7		**GLACIAL TILL** Silty sand with gravel, cobbles and boulders, grey, damp, (very dense)	65	15	SS8	
			81	15	SS9	
		Coring through cobbles and boulders from 6.6 m to 7.0 m depths				
68.0		**SANDSTONE BEDROCK** With shale, fine grained, laminated, grey, (poor quality) Weathered and highly fractured zone from 7.0 m to 7.5 m depths			Run 1	
67.0	8	**Borehole Terminated at 8.0 m Depth**				

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	N/A	8.0

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7 - 8	100	40

Log of Borehole BH 19-04

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 6

Page. 1 of 1

Date Drilled: 'April 29, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Legend	Symbol	Legend	Symbol
Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

Depth (m)	Geodetic Elevation (m)	Soil Description	SPT N Value	Natural Moisture Content	Sample	Natural Unit Wt. kN/m³
0	74.98					
	74.9	**ASPHALTIC CONCRETE** ~60 mm				
	74.7	**GRANULAR FILL (BASE)** ~250 mm Crushed gravel with silt and sand, grey, damp	30	×	SS1	
		FILL Clayey silty sand to silty sand with gravel, brown, moist to wet, (loose)	5	×	SS2	19.1
			5	×	SS3	
2	72.8	**GLACIAL TILL** Silty clay with sand and gravel, cobbles and boulders, grey, wet, (soft)	4	×	SS4	
		Petroleum odour from 3.0 m to 3.6 m depths	3	×	SS5	
4	70.9	**GLACIAL TILL** Silty sand with gravel, cobbles and boulders, grey, wet, (compact to very dense)	22	×	SS6	23.9
		Petroleum odour from 5.5 m to 6.1 m depths	67	×	SS7	
6.6	68.4	**Auger Refusal at 6.6 m Depth**				

Chart scales: Standard Penetration Test N Value 20, 40, 60, 80; Shear Strength (kPa) 50, 100, 150, 200; Combustible Vapour Reading (ppm) 250, 500, 750; Natural Moisture Content % / Atterberg Limits (% Dry Weight) 20, 40, 60.

LOG OF BOREHOLE BH LOGS - 252625 FOREST.GPJ TROW OTTAWA.GDT 2/5/20

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	5.5	6.6

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 19-05

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 7

Page. 1 of 1

Date Drilled: 'April 30, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

SOIL DESCRIPTION	Geodetic Elevation m	Depth (m)	SPT N Value	Combustible Vapour Reading (ppm)	Samples	Natural Unit Wt. kN/m³
	74.82	0				
GRANULAR FILL ~100 mm Crushed gravel wtih silt and sand, grey, damp	74.7		11	40	SS1	
FILL Clayey silty sand to silty sand with gravel, rootlets, brown and grey, moist to wet, petroleum odour, (loose)		1	9	40	SS2	20.5
			6	200	SS3	
		2	7	60	SS4	22.9
GLACIAL TILL Silty clay with sand and gravel, grey, wet, petroleum odour, (soft)	71.8	3	2	45	SS5	
GLACIAL TILL Silty clayey sand with gravel, cobbles and boulders, grey, wet, petroleum odour, (very loose)	71.1	4	3	30	SS6	
		5	1	15	SS7	
	68.1	6				
Auger Refusal at 6.7 m Depth						

Chart axes: Standard Penetration Test N Value 20, 40, 60, 80; Shear Strength 50, 100, 150, 200 kPa; Combustible Vapour Reading (ppm) 250, 500, 750; Natural Moisture Content % / Atterberg Limits (% Dry Weight) 20, 40, 60.

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	4.6	5.5

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 19-06

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 8

Page. 1 of 1

Date Drilled: 'April 25, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Legend	Symbol	Legend	Symbol
Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

Soil Description	Geodetic Elevation (m)	Depth (m)	SPT N Value	Combustible Vapour Reading (ppm)	Sample	Natural Unit Wt. kN/m³
	75.28	0				
ASPHALTIC CONCRETE ~30 mm	75.2					
GRANULAR FILL (BASE) ~200 mm Crushed gravel with silt and sand, grey, damp	75.0		5	0	SS1	
FILL Silty clayey sand to silty sand with gravel, brick debris, brown, moist to wet, (loose)		1	5	5	SS2	
			9	0	SS3	
	72.88	2				
GLACIAL TILL Silty sand with gravel, cobbles and boulders, grey, wet, (very loose to loose)	72.7		4	0	SS4	
		3	3	0	SS5	
		4	5	0	SS6	
			3	0	SS7	
		5	9	0	SS8	
	69.4					

Borehole Terminated at 5.9 m Depth

Chart scales: Standard Penetration Test N Value 20, 40, 60, 80; Shear Strength (kPa) 50, 100, 150, 200; Combustible Vapour Reading (ppm) 250, 500, 750; Natural Moisture Content % / Atterberg Limits (% Dry Weight) 20, 40, 60.

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	4.6	
15 days	2.5	
20 days	2.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH19-07

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Date Drilled: 'April 24, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Figure No. 9

Page. 1 of 1

Legend	Symbol	Legend	Symbol
Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

SOIL DESCRIPTION	Geodetic Elevation (m)	Depth (m)	SPT N Value	Combustible Vapour Reading (ppm)	Natural Moisture Content	Samples	Natural Unit Wt. kN/m³
	75.21	0					
ASPHALTIC CONCRETE ~60 mm	75.1						
GRANULAR FILL (BASE) ~220 mm Crushed gravel with silt and sand, grey, damp	74.9						
FILL Silty sand with gravel, brown, moist to wet, (loose)	73.81	1	9	0	×	SS1	19.4
			8	0	×	SS2	
GLACIAL TILL Silty sand with gravel, cobbles and boulders, grey, wet, (loose to compact)	73.0	2	13	0	×	SS3	
Shale fragements from 3.0 m to 3.6 m depths		3	8	0	×	SS4	
		4	13	0	×	SS5	
		5	8	0	×	SS6	
	69.3		8	0	×	SS7	

Borehole Terminated at 5.9 m Depth

Standard Penetration Test N Value: 20, 40, 60, 80; Shear Strength (kPa): 50, 100, 150, 200

Combustible Vapour Reading (ppm): 250, 500, 750; Natural Moisture Content %; Atterberg Limits (% Dry Weight): 20, 40, 60

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	5.2	6.1
16 days	1.4	
22 Days	1.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 19-08

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 10

Page. 1 of 1

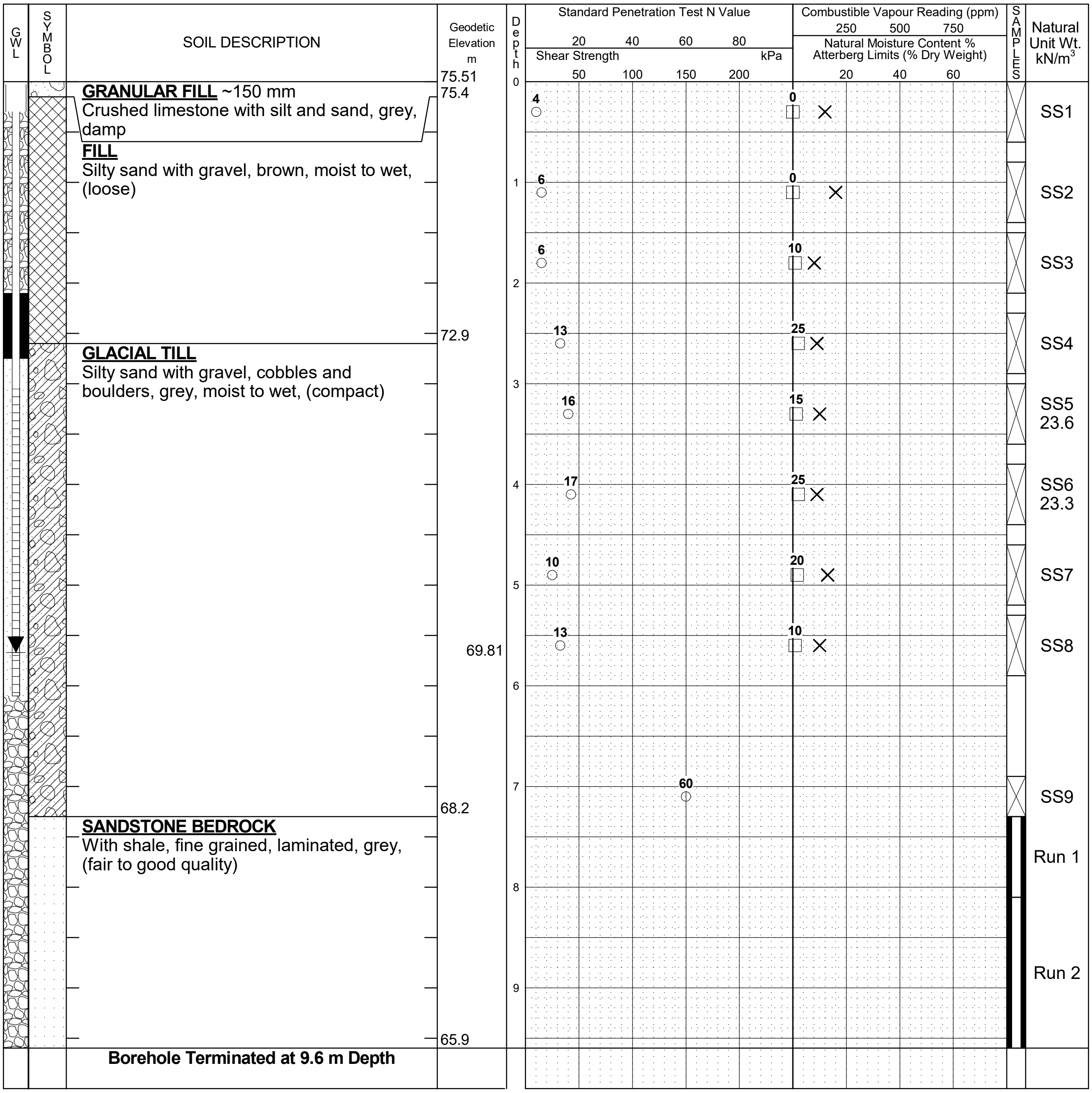
Date Drilled: 'April 25, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Legend		Legend	
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample		Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



Soil Description	Geodetic Elevation (m)	Depth (m)	SPT N	Vapour (ppm)	Sample	Natural Unit Wt. kN/m³
	75.51	0				
GRANULAR FILL ~150 mm Crushed limestone with silt and sand, grey, damp	75.4		4	0	SS1	
FILL Silty sand with gravel, brown, moist to wet, (loose)		1	6	0	SS2	
		2	6	10	SS3	
	72.9		13	25	SS4	
GLACIAL TILL Silty sand with gravel, cobbles and boulders, grey, moist to wet, (compact)		3	16	15	SS5	23.6
		4	17	25	SS6	23.3
		5	10	20	SS7	
	69.81		13	10	SS8	
		6				
		7	60		SS9	
SANDSTONE BEDROCK With shale, fine grained, laminated, grey, (fair to good quality)	68.2				Run 1	
		8			Run 2	
	65.9	9				
Borehole Terminated at 9.6 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	N/A	N/A
15 days	5.6	
20 days	5.7	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.3 - 8.1	94	53
2	8.1 - 9.6	100	78

Log of Borehole BH 19-09

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 11

Page. 1 of 1

Date Drilled: 'April 24, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Legend	Symbol	Legend	Symbol
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

Depth (m)	Geodetic Elevation (m)	SOIL DESCRIPTION	SPT N Value	Combustible Vapour Reading (ppm)	Natural Moisture Content	Sample	Natural Unit Wt. kN/m³
0	75.65	**GRANULAR FILL** ~300 mm Crushed gravel with silt and sand, grey, damp	7	0	×	SS1	
	75.4	**FILL** Silty sand with gravel, brown, moist, (very loose to loose)	1	5	×	SS2	
			9	0	×	SS3	21.9
						SS4	
	73.1	**GLACIAL TILL** Silty sand with gravel, cobbles and boulders, grey, moist to wet, (loose to compact)	10	0	×	SS5	
			18	0		SS6	
			7	0	×	SS7	
			6	0	×	SS8	
	69.65		13	0		SS9	
	68.1	**SANDSTONE BEDROCK** With shale, fine grained, laminated, grey, (fair quality)				Run 1	
	67.0	**Borehole Terminated at 8.7 m Depth**					

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	N/A	N/A
21 days	6.0	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.6 - 8.7	95	61

Log of Borehole BH 19-10

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 12

Page. 1 of 1

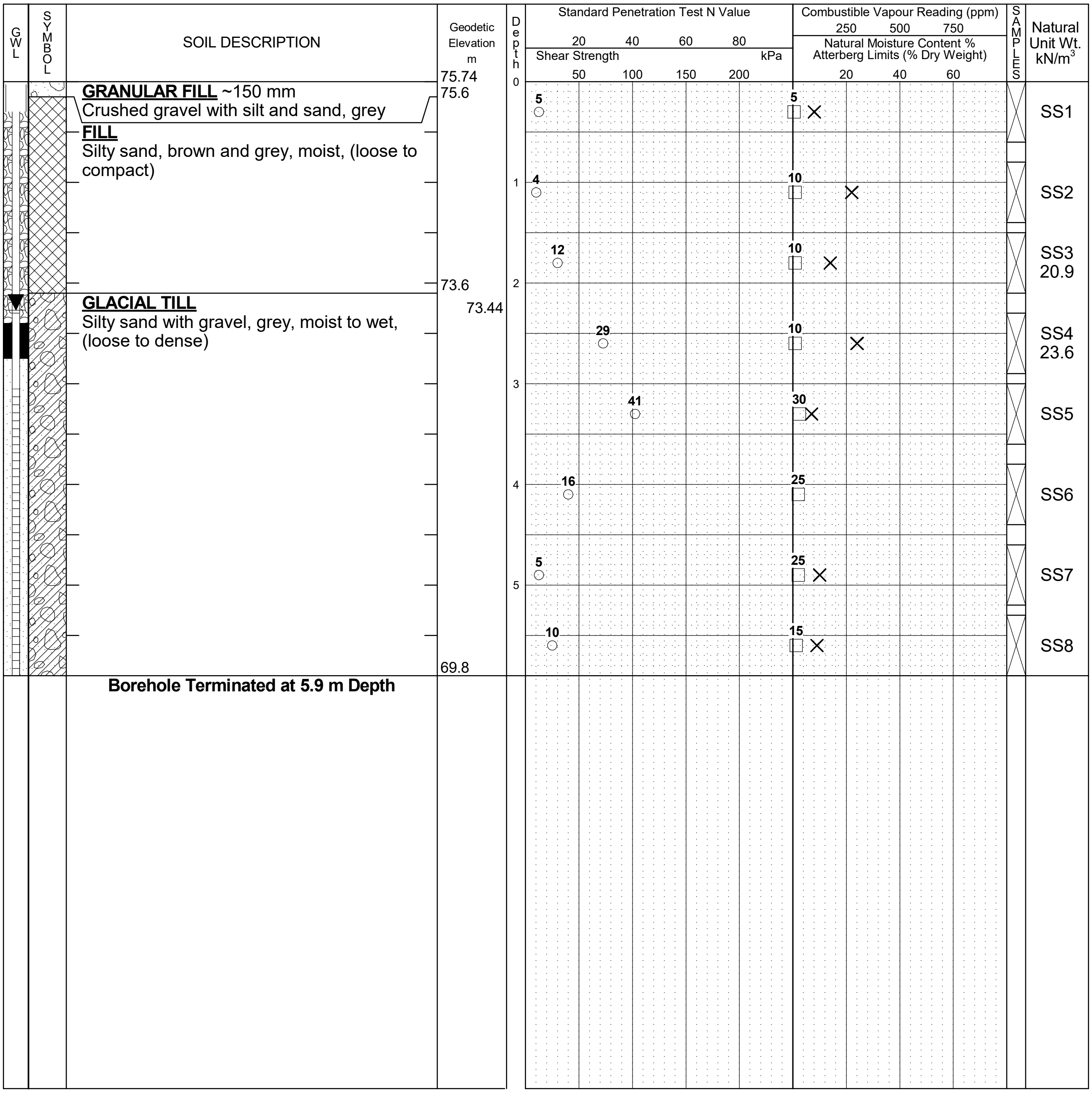
Date Drilled: 'April 25, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Symbol		Symbol	
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



LOG OF BOREHOLE BH LOGS - 252625 FOREST.GPJ TROW OTTAWA.GDT 2/5/20

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	5.2	N/A
15 days	2.2	
20 days	2.3	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH 19-11

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 13

Page. 1 of 1

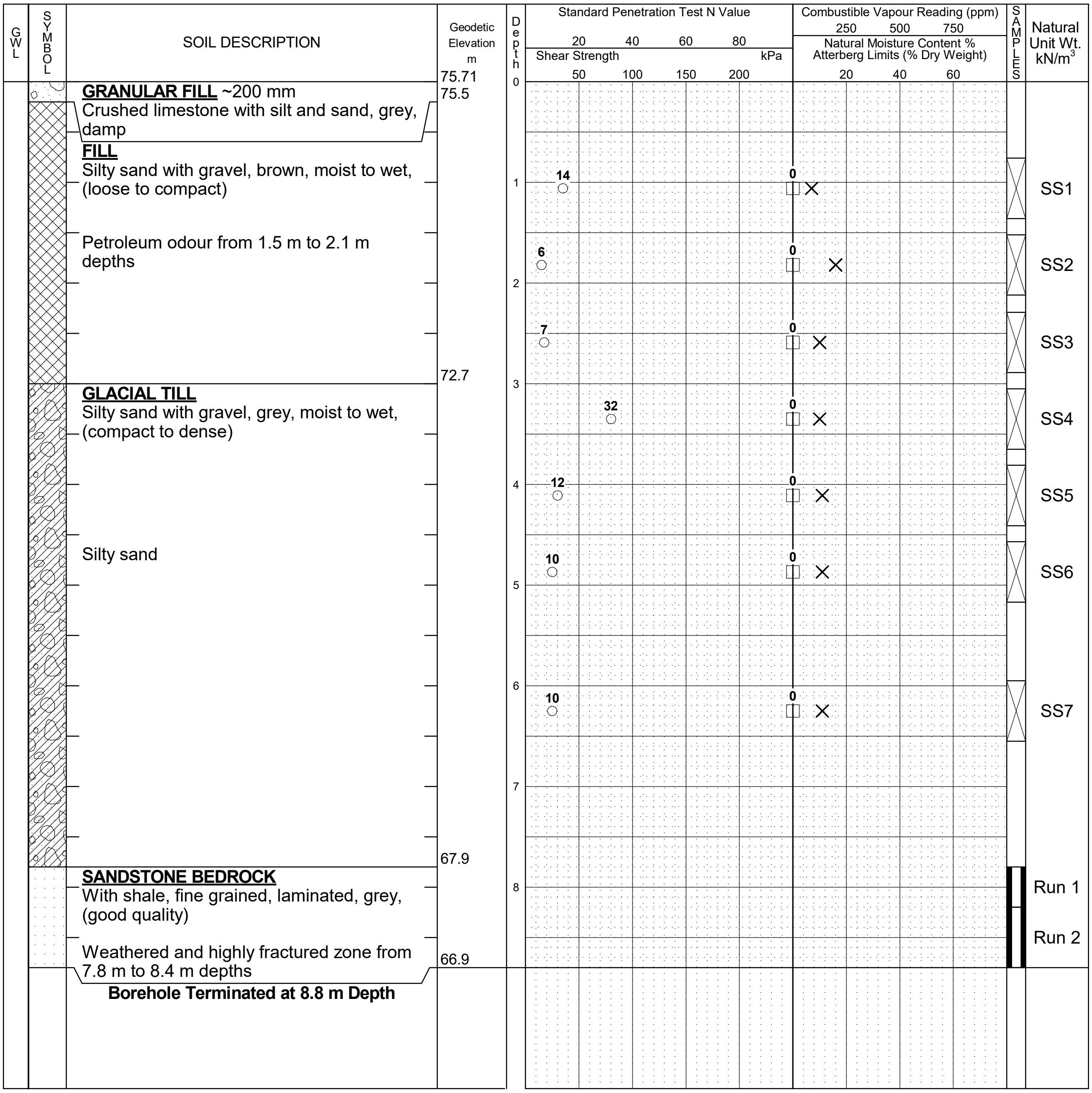
Date Drilled: 'April 29, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample		Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	
Dynamic Cone Test		Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



LOG OF BOREHOLE BH LOGS - 252625 FOREST.GPJ TROW OTTAWA.GDT 2/5/20

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	N/A	8.8

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.8 - 8.2	94	77
2	8.2 - 8.8	100	86

Log of Borehole BH 19-12

Project No: OTT-00252625-A0

Project: Residential Development

Location: 365 Forest Street, Ottawa, Ontario

Figure No. 14

Page. 1 of 1

Date Drilled: 'April 24, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. Checked by: I.T.

Legend	Symbol	Legend	Symbol
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢—○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth (m)	SPT N Value (Shear Strength kPa)	Combustible Vapour Reading (ppm)	Natural Moisture Content % / Atterberg Limits (% Dry Weight)	SAMPLES	Natural Unit Wt. kN/m³
		GRANULAR FILL ~150 mm Crushed gravel with silt and sand, damp, grey	75.42 75.3	0	4	0		SS1	
		FILL Silty sand with gravel, brown, moist to wet, (loose to compact)		1	12	0		SS2	
					16	0		SS3	
				2	7	0		SS4	22.4
				3	7	0		SS5	
			71.0	4	5	0		SS6	
		Borehole Terminated at 4.4 m Depth							

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00252625-A0

WATER LEVEL RECORDS

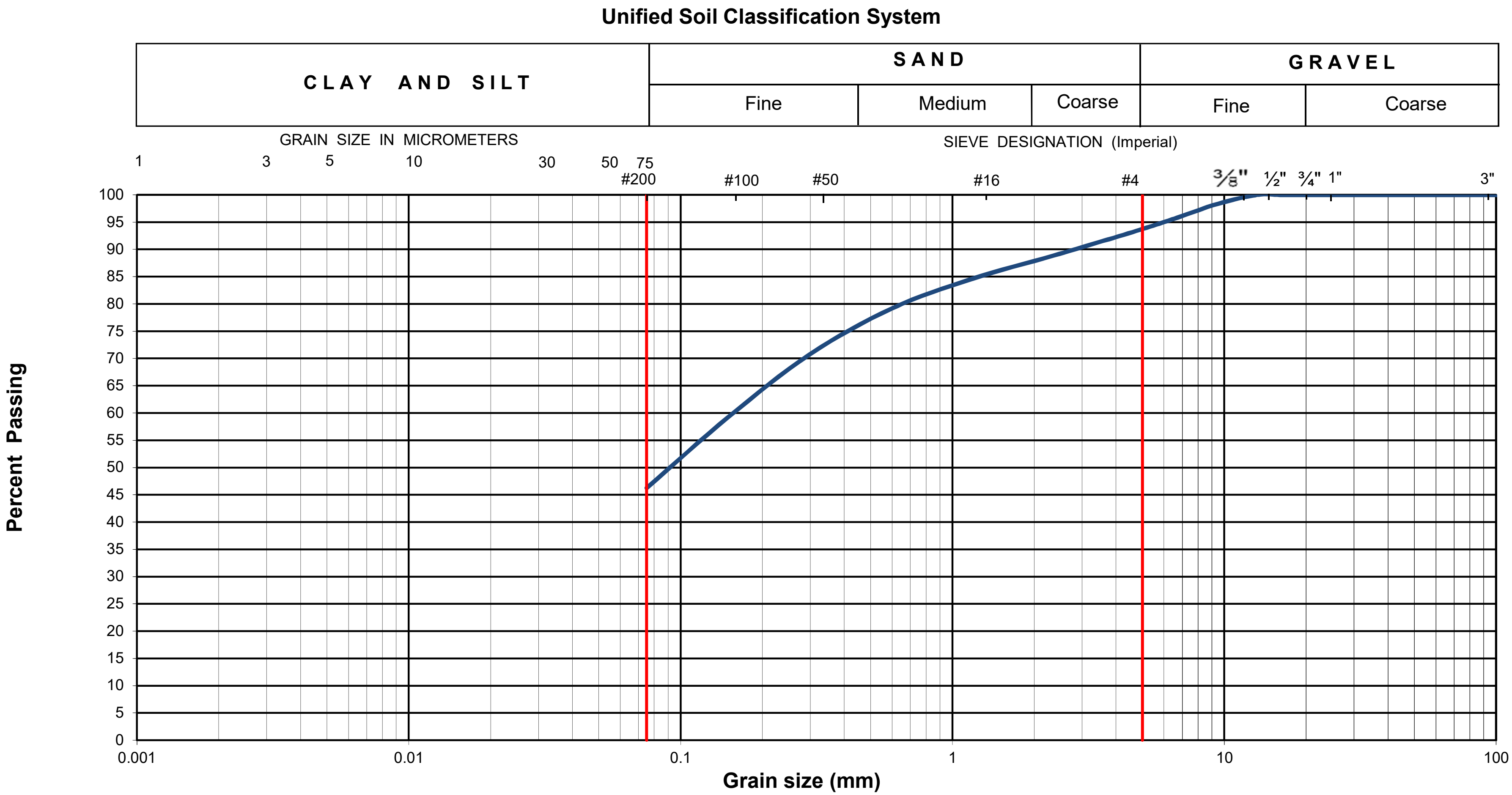
Date	Water Level (m)	Hole Open To (m)
Completion	Dry	3.8

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

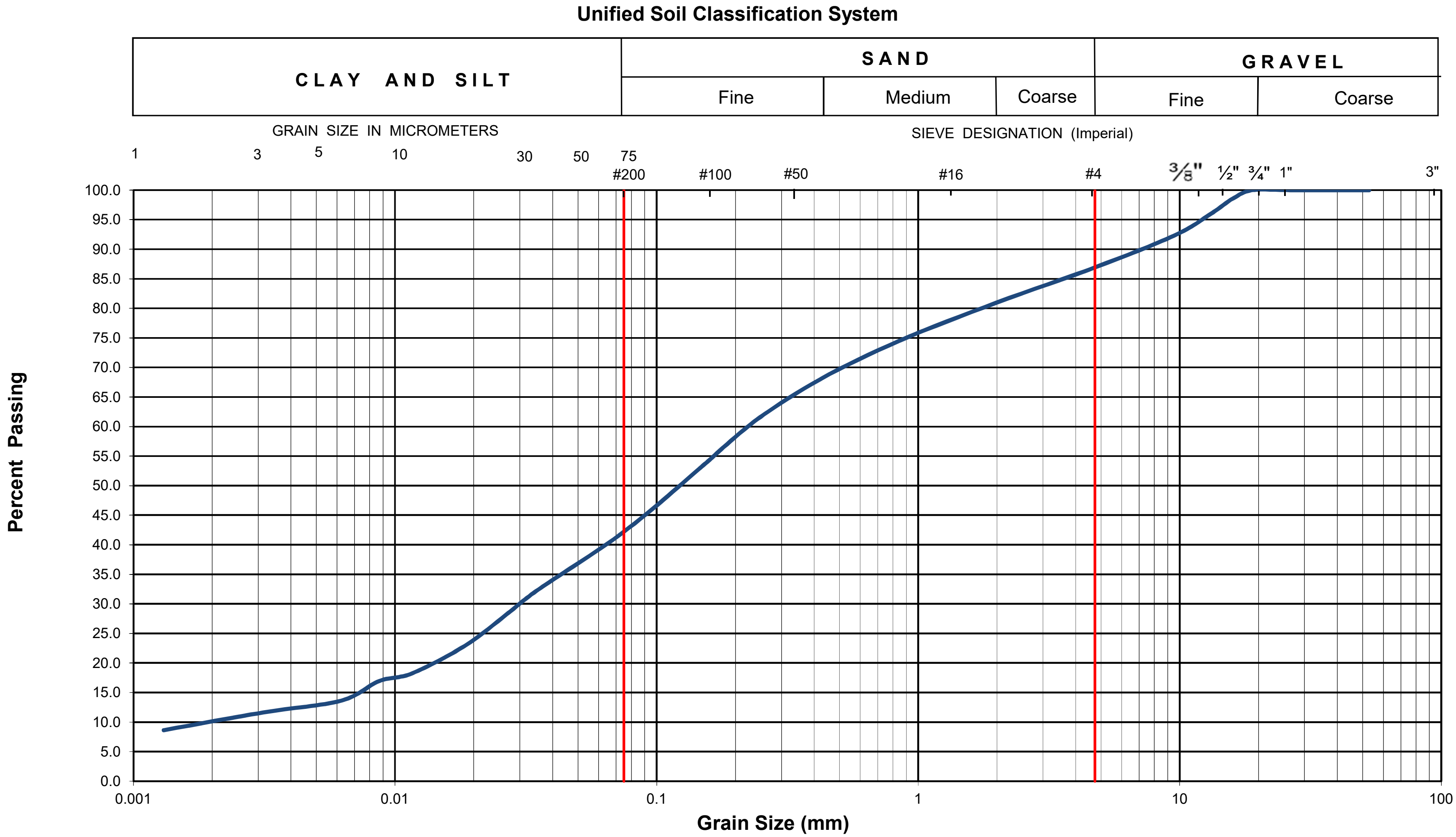
Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136



EXP Project No.:	OTT-00252625-A0	Project Name :	Residential Development				
Client :	11061917 Canada Inc.	Project Location :	365 Forest Ave, Ottawa, ON.				
Date Sampled :	April 25, 2019	Borehole No:	19-10	Sample:	SS3	Depth (m) :	1.5-2.1
Sample Composition :		Gravel (%)	7	Sand (%) 47	Silt & Clay (%) 46	Figure :	15
Sample Description :	FILL: Silty Sand (SM)						

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422

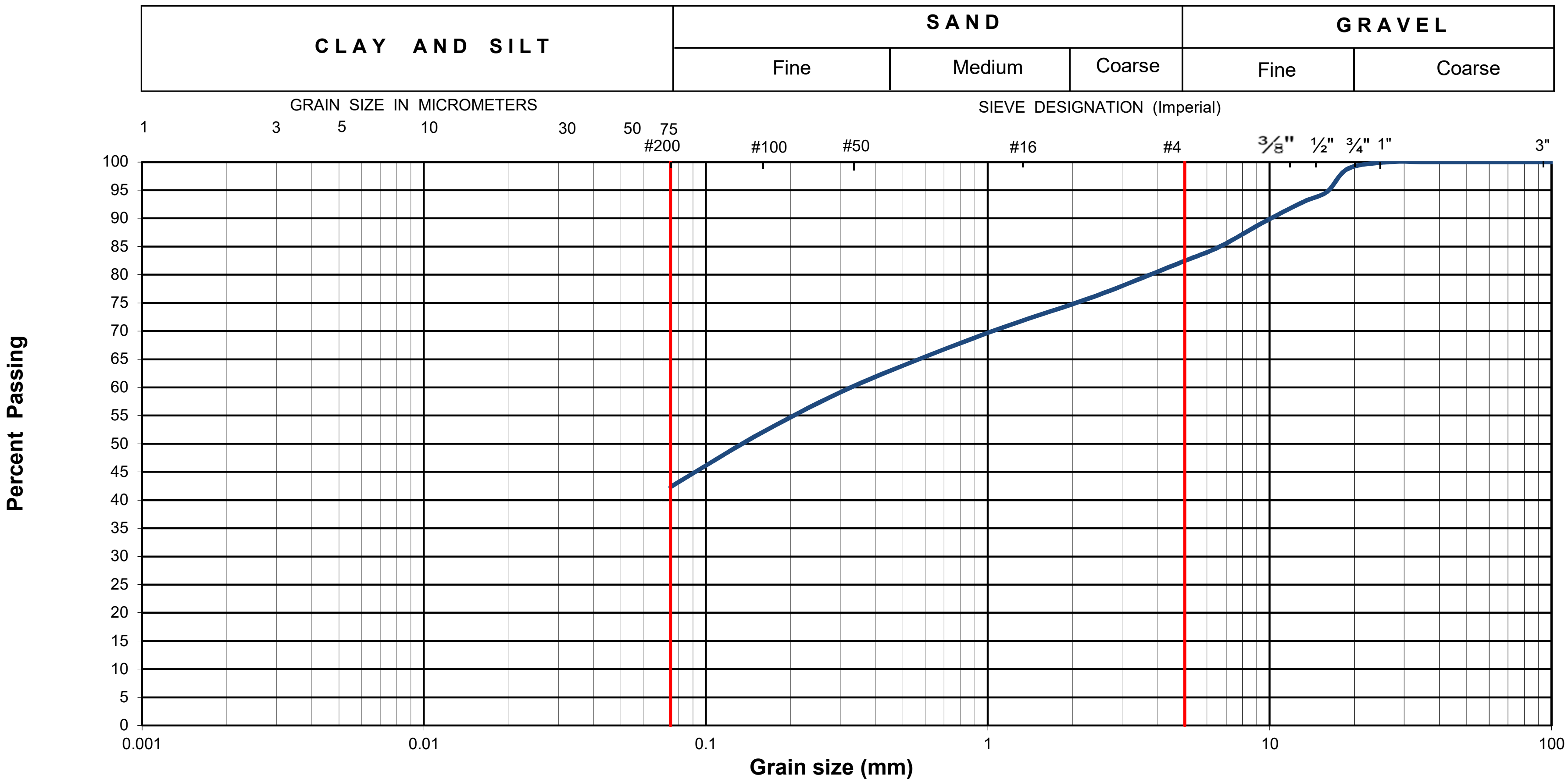


EXP Project No.:	OTT-00252625-A0	Project Name :	Residential Development					
Client :	11061917 Canada Inc.	Project Location :	365 Forest Ave, Ottawa, ON.					
Date Sampled :	April 29, 2019	Borehole No:	BH 19-02		Sample No.: SS5		Depth (m) :	3.8-4.4
Sample Description :		% Silt and Clay	42	% Sand	45	% Gravel	13	Figure : 16
Sample Description :	GLACIAL TILL: Silty Sand (SM)							

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136

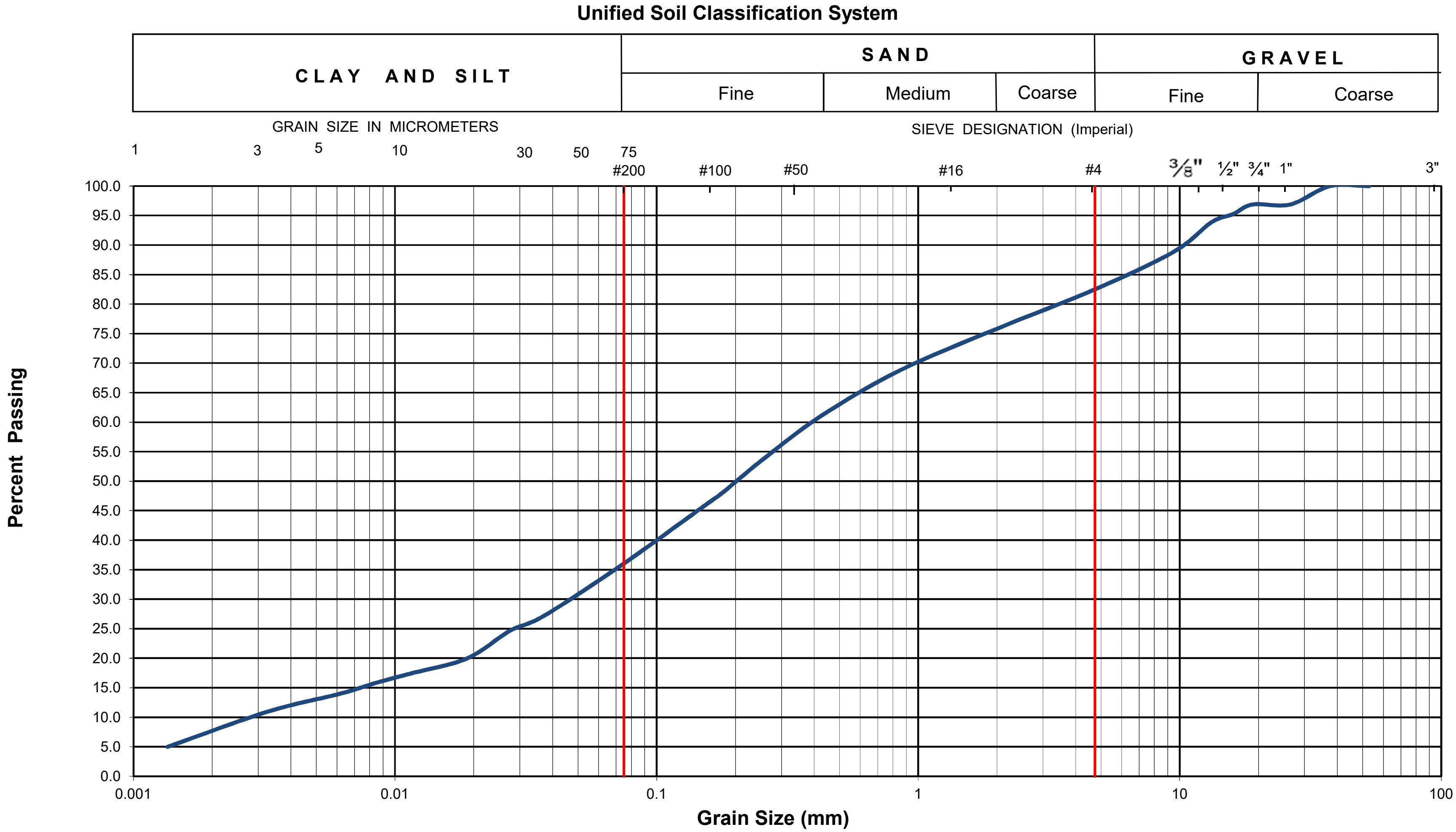
Unified Soil Classification System



EXP Project No.:	**OTT-00252625-A0**	**Project Name :**	**Residential Development**				
Client :	**11061917 Canada Inc.**	**Project Location :**	**365 Forest Ave, Ottawa., ON.**				
Date Sampled :	**April 25, 2019**	**Borehole No:**	**BH 19-04**	**Sample:**	**SS6**	**Depth (m) :**	**4.6-5.2**
Sample Composition :		**Gravel (%)**	**18**	**Sand (%)**	**40**	**Silt & Clay (%)**	**42**
Sample Description :	**GLACIAL TILL: Silty Sand with Gravel (SM)**					**Figure :**	**17**

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

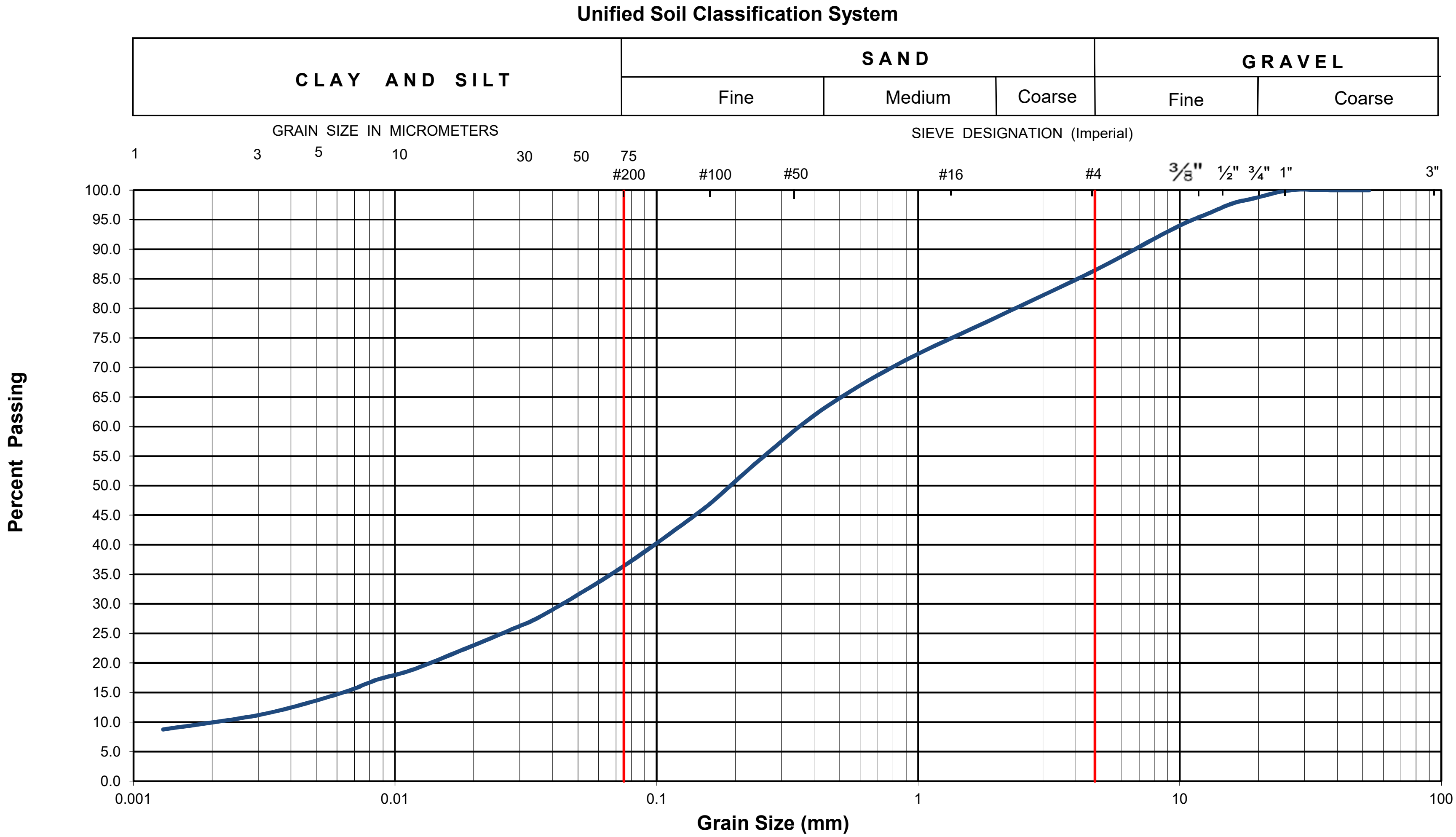
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



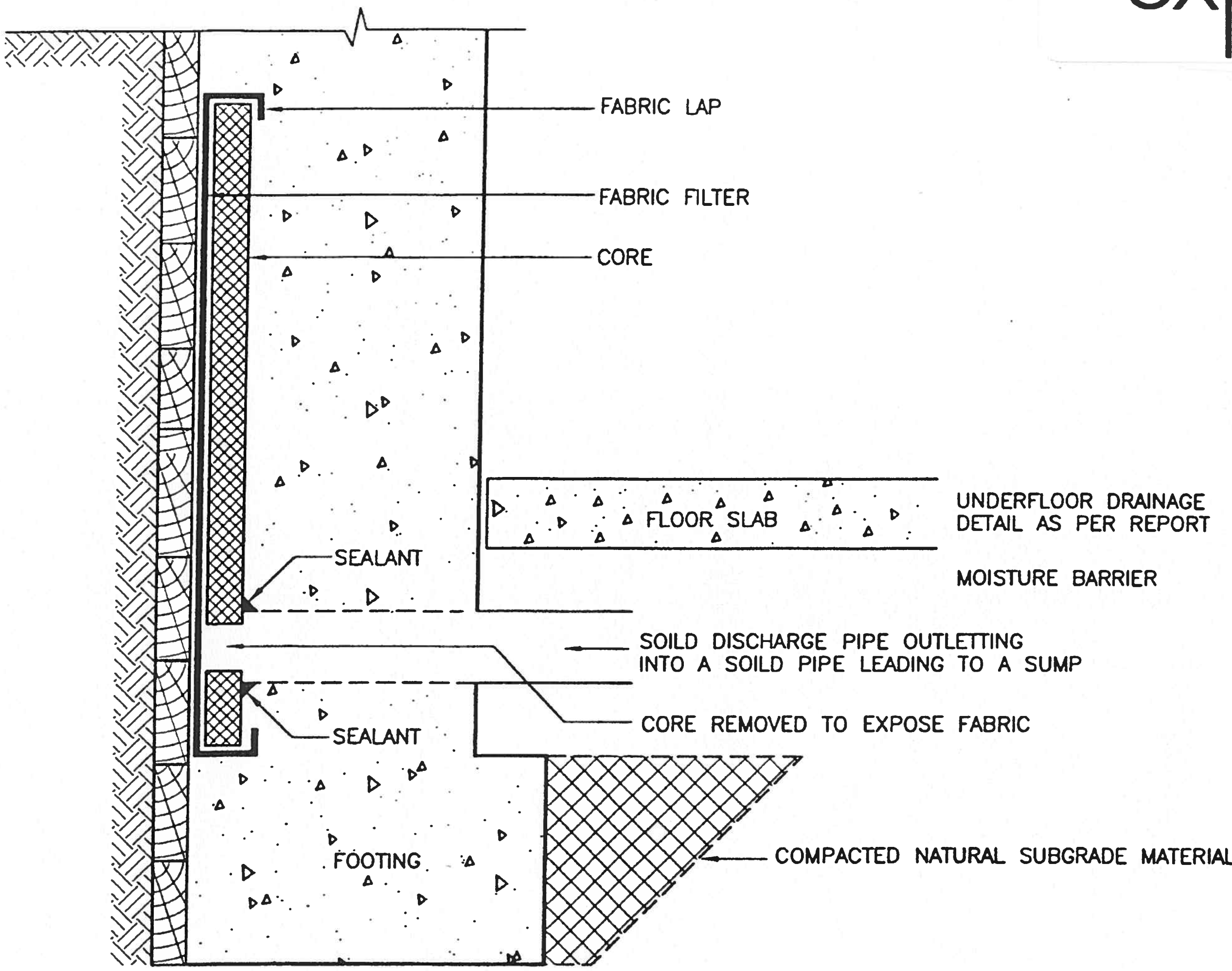
EXP Project No.:	OTT-00252625-A0	Project Name :	Residential Development					
Client :	11061917 Canada Inc.	Project Location :	365 Forest Ave, Ottawa, ON.					
Date Sampled :	April 25, 2019	Borehole No:	BH 19-08	Sample No.:	SS5		Depth (m) :	3.0-3.6
Sample Description :		% Silt and Clay	36	% Sand	47	% Gravel	17	Figure : 18
Sample Description :	GLACIAL TILL: Silty Sand with Gravel (SM)							

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-00252625-A0	Project Name :	Residential Development					
Client :	11061917 Canada Inc.	Project Location :	365 Forest Ave, Ottawa, ON.					
Date Sampled :	April 24, 2019	Borehole No:	BH 19-11	Sample No.:	SS6		Depth (m) :	4.6-5.2
Sample Description :		% Silt and Clay	36	% Sand	51	% Gravel	13	Figure : 19
Sample Description :	GLACIAL TILL: Silty Sand (SM)							



SECTION AT DISCHARGE PIPE

NOTES:

1. DRAINAGE CORE AND CLOTH TO BE TERRADRAIN 200 OR EQUIVALENT.
2. INSTALLATION INSTRUCTIONS AS PER MANUFACTURES SPECIFICATION.
3. TO BE FULL WIDTH UNLESS OTHERWISE RECOMMENDED BY THE ENGINEER.
4. FINAL DETAIL MUST BE APPROVED BEFORE SYSTEM IS CONSIDERED ACCEPTABLE.
5. TERRADRAIN 200 SHOULD BE KEPT A MINIMUM OF 1.2 m BELOW EXTERIOR FINISHED GRADE.

SUGGESTED EXTERIOR DRAINAGE AGAINST SOLDIER PILE AND LAGGING SHORING SYSTEM

Appendix A:
Laboratory Certificate of Analysis

AGAT Laboratories

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: EXP SERVICES INC
2650 QUEENSVIEW DRIVE, UNIT 100
OTTAWA, ON K2B8H6
(613) 688-1899

ATTENTION TO: Susan Potyondy

PROJECT: OTT-252625-A0

AGAT WORK ORDER: 19Z484141

SOIL ANALYSIS REVIEWED BY: Nivine Basily, Inorganics Report Writer

DATE REPORTED: Jul 02, 2019

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: EXP SERVICES INC

ATTENTION TO: Susan Potyondy

SAMPLING SITE:365 Forest Street

SAMPLED BY:exp

Inorganic Chemistry (Soil)

DATE RECEIVED: 2019-06-25

DATE REPORTED: 2019-07-02

			SAMPLE DESCRIPTION:	BH1 Run1 21'3"-23'7"	BH3 SS5 10'-12'	BH11 Run1 25'8"-26'9"	BH12 SS4 7'6"-9'6"
			SAMPLE TYPE:	Soil	Soil	Soil	Soil
			DATE SAMPLED:	2019-04-30	2019-04-30	2019-04-30	2019-04-30
Parameter	Unit	G / S	RDL	301670	301671	301672	301673
pH (2:1)	pH Units		N/A	8.55	8.00	8.59	8.17
Resistivity (2:1)	ohm.cm		1	2490	2480	2110	1950
Chloride (2:1)	µg/g		2	25	120	21	114
Sulphate (2:1)	µg/g		2	42	50	26	121

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

301670-301673 pH was determined on the 0.01M CaCl2 extract obtained from 2:1 leaching procedure (2 parts extraction fluid:1 part wet soil).

Chloride & Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).

Resistivity is a calculated parameter.

Certified By: Nivine Basily

AGAT Laboratories
5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Quality Assurance

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-252625-A0
SAMPLING SITE:365 Forest Street

AGAT WORK ORDER: 19Z484141
ATTENTION TO: Susan Potyondy
SAMPLED BY:exp

Soil Analysis

RPT Date:			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
Inorganic Chemistry (Soil)															
pH (2:1)	301670	301670	8.55	8.50	0.6%	NA	101%	90%	110%						
Chloride (2:1)	301670	301670	25	26	3.9%	< 2	109%	70%	130%	99%	70%	130%	98%	70%	130%
Sulphate (2:1)	301670	301670	42	43	2.4%	< 2	101%	70%	130%	105%	70%	130%	109%	70%	130%

Comments: NA signifies Not Applicable.
pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

Certified By: Nivine Basily

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Method Summary

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-252625-A0
SAMPLING SITE:365 Forest Street

AGAT WORK ORDER: 19Z484141
ATTENTION TO: Susan Potyondy
SAMPLED BY:exp

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	EC METER
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH

AGAT Laboratories

Imeel blue

5835 Coopers Avenue
Mississauga, Ontario L4Z 1Y2
Ph: 905.712.5100 Fax: 905.712.5122
webearth.agatlabs.com

Chain of Custody Record

If this is a Drinking Water sample, please use Drinking Water Chain of Custody Form (potable water consumed by humans)

Laboratory Use Only

Work Order #: 19Z484141

Cooler Quantity: One - no ice

Arrival Temperatures: 21.3 | 21.0 | 21.1
LT-9.6 | 8.9 | 9.2

Custody Seal Intact: ☐Yes ☐No ☐N/A

Notes: (on ice)

Report Information:

Company: Exp Services
Contact: Susan Potyondy
Address: 2650 Queensview dr Suite 100
Ottawa On K2B 8H6
Phone: 613-688-1899 Fax:
Reports to be sent to:
1. Email: Susan.Potyondy@exp.com
2. Email:

Regulatory Requirements:

(Please check all applicable boxes)

☐ No Regulatory Requirement

☐ Regulation 153/04
Table ___ Indicate One
☐Ind/Com
☐Res/Park
☐Agriculture
Soil Texture (Check One)
☐Coarse
☐Fine

☐ Sewer Use
☐Sanitary
☐Storm
Region ___ Indicate One
☐ MISA

☐ Regulation 558
☐ CCME
☐ Prov. Water Quality Objectives (PWQO)
☐ Other
___ Indicate One

Is this submission for a **Record of Site Condition?** ☐ Yes ☐ No

Report Guideline on *Certificate of Analysis* ☐ Yes ☐ No

Turnaround Time (TAT) Required:

Regular TAT ☒ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

☐ 3 Business Days ☐ 2 Business Days ☐ Next Business Day

OR Date Required (Rush Surcharges May Apply): ___

Please provide prior notification for rush TAT
**TAT is exclusive of weekends and statutory holidays*

For 'Same Day' analysis, please contact your AGAT CPM

Project Information:

Project: OTT-252625-AO
Site Location: 365 Forest street
Sampled By: exp
AGAT Quote #: ___ PO: ___
Please note: If quotation number is not provided, client will be billed full price for analysis.

Invoice Information:

Bill To Same: Yes ☒ No ☐

Company:
Contact:
Address:
Email:

Sample Matrix Legend

B	Biota
GW	Ground Water
O	Oil
P	Paint
S	Soil
SD	Sediment
SW	Surface Water

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI (Y/N)	Metals and Inorganics (O. Reg 153)	☐ All Metals ☐ 153 Metals (excl. Hydrides) ☐ Hydride Metals ☐ 153 Metals (Incl. Hydrides) (O. Reg 153)	ORPs: ☐B-HWS ☐Cl ☐CN ☐Cr6+ ☐EC ☐FOC ☐Hg ☐pH ☐SAR (O. Reg 153)	Full Metals Scan	Regulation/Custom Metals	Nutrients: ☐TP ☐NH3 ☐TKN ☐NO3 ☐NO2 ☐NO3+NO2	Volatiles: ☐VOC ☐BTEX ☐THM	PHCs F1 - F4	ABNs	PAHs	PCBs: ☐Total ☐Aroclors	Organochlorine Pesticides	TCLP: ☐M&I ☐VOCs ☐ABNs ☐B(a)P ☐PCBs	Sewer Use	pH	Sulphate	Chloride	Electro Resistivity		
BH 1 Run 1 21'3"-23'7"	Ap 30/19																				✓	✓	✓	✓		
BH 3 SS5 10'-12'	Ap 30/19																				✓	✓	✓	✓		
BH 11 Run 1 25'8"-26'9"	Ap 29/19																				✓	✓	✓	✓		
BH 12 SS4 7'6"-9'6"	Ap 24/19																				✓	✓	✓	✓		

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Ryan D:Giuseppe	June 24/19	5:45pm	Bernhelet	2019-06-25	8h41
Bernhelet	19-06-25	16h00			
			Shanmin	June 26/19	10:00am

Page 1 of 1

Nº: T078057

Appendix B:
Multi-channel Analysis of Surface Waves (MASW) Survey

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7

Tel. : (450) 679-2400
Fax : (514) 521-4128
info@geophysicsgpr.com
www.geophysicsgpr.com

August 23th, 2019

Transmitted by email: ismail.taki@exp.com
Our Ref.: GPR-19-01572-D

Mr. Ismail Taki, M.Eng., P.Eng.
Manager, Geotechnical Services
exp Services inc.
100 - 2650 Queensview Drive
Ottawa (ON) K2B 8H6

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
365 Forest Street, Ottawa (ON)

[Project: OTT-00252625-A0]

Dear Sir,

Geophysics GPR International Inc. has been requested by **exp** Services Inc. to carry out seismic shear wave surveys on a parking lot located east of the intersection of Forest Street and Richmond Road, in Ottawa (ON). The geophysical investigations used the Multi-channel Analysis of Surface Waves (MASW), the Extended SPatial AutoCorrelation (ESPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocities values were calculated for the soil and the rock, and the Site Class was identified.

The surveys were carried out on July 30th, 2019, by Mr. Dominic Déraps and Mr. Jacques-Olivier Joly. 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 principle of the test method, and the results in graphic and table format.

MASW PRINCIPLE

The *Multi-channel Analysis of Surface Waves* (MASW) and the *Extended SPatial AutoCorrelation* (ESPAC 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 waves ("ground roll"). 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 ESPAC 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 dispersion properties are expressed as a change of phase velocities with 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/ESPAC records, the corresponding spectrogram analysis and resulting 1D V_S model. The ESPAC method allows deeper Vs soundings, but generally with a lower resolution for the surface portion. Its dispersion curve can then be merged with the higher frequency one from the MASW to calculate a more complete inversion.

INTERPRETATION

The main processing sequence involved data inspection and edition when required; spectral analysis ("phase shift" for MASW, and "cross-correlation" for ESPAC); picking the fundamental mode; and 1D inversion of the MASW and ESPAC 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 located on a parking lot, east of the intersection of Forest Street and Richmond Road. The geophone spacing for the main spread was 3 metres, using 24 geophones. A shorter seismic spread, with geophone spacing of 1 metre, was dedicated to the near surface materials.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and 50 µs for the seismic refraction. The records included a pre-trig portion of 10 ms. 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. The seismic records were produced with a seismograph Terraloc MK6 (from ABEM Instrument), and the geophones were 4.5 Hz. A 9 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic spreads.

RESULTS

From seismic refraction, the rock depth was calculated between 5.5 and 9 metres, with an average depth of 7.2 metres (+/- 1 metre), and its seismic shear wave velocity was calculated between 1580 and 1625 m/s for its upper portion. These results were used as initial parameters for the basic geophysical model, prior to the MASW dispersion curves modeling and inversions. The MASW calculated V_S results are illustrated at Figure 5 and they are also presented at Table 1.

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 up to 30 metres. This value represents an equivalent homogeneous single layer response.

The $\overline{V}_{S30}$ value for the actual site is 581.7 m/s (cf. Table 1), corresponding to the Site Class “C”. The planned foundation depth being 8.6 metres, the $\overline{V}_{S30}$* value would be 1610.4 m/s (cf. Table 2), allowing the use of the Site Class “A”.

CONCLUSION

Geophysical surveys were carried out on a parking lot, located east of the intersection of Forest Street and Richmond Road, in Ottawa (ON). The seismic surveys used the MASW and ESPAC analysis methods, and the seismic refraction as complement. The $\overline{V}_{S30}$ calculation, for the actual site, is presented in Table 1.

The calculated $\overline{V}_{S30}$ value of the actual site is 582 m/s corresponding to the Site Class “C” ($360 < \overline{V}_{S30} \leq 760$ m/s), as determined through the MASW and ESPAC methods, Table 4.1.8.4.A of the NBC, and the Building Code, O. Reg. 332/12. As the planned foundation depth is 8.6 metres, the corresponding $\overline{V}_{S30}*$ value would be 1610 m/s, allowing to use the Site Class “A” (Table 2).

It must be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, quick and highly sensitive clays, high moisture content etc. 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.

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

Figure 1: Regional location of the Site

(source: ***OpenStreetMap©***)

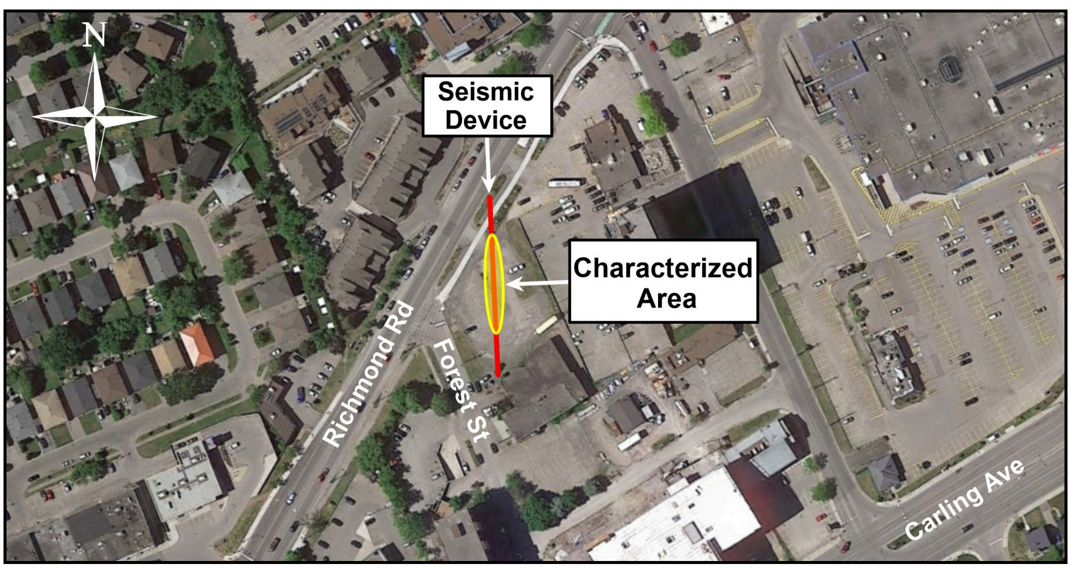


Figure 2: Location of the seismic spreads

(source: ***GoogleEarth©***)

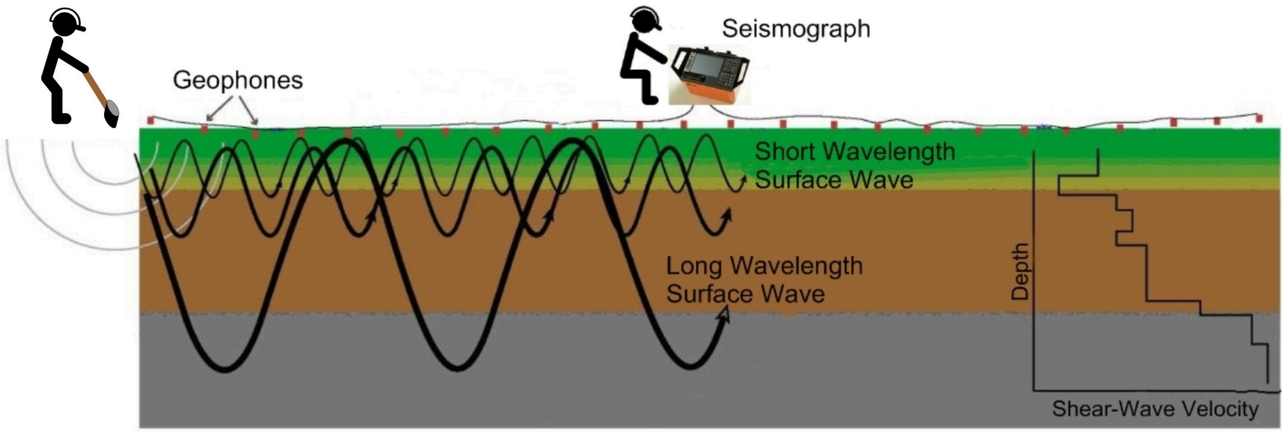
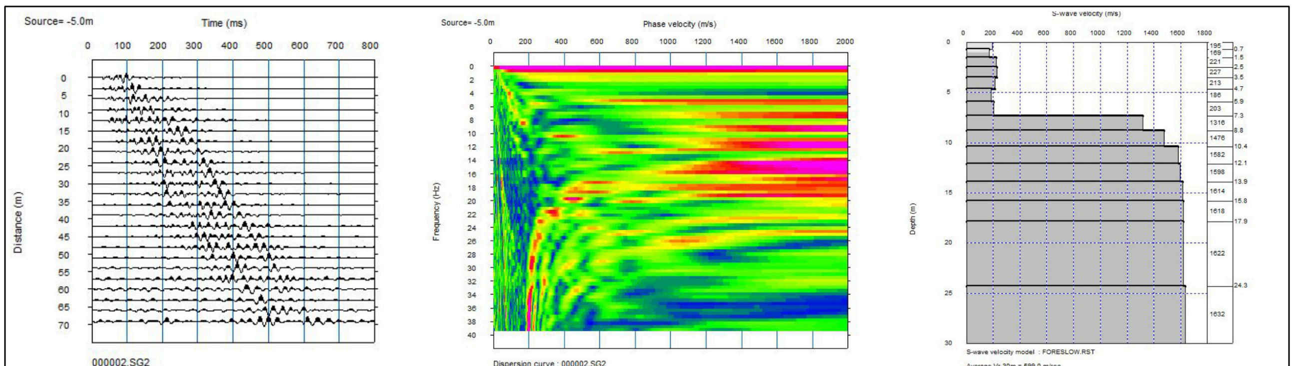


Figure 3: MASW Operating Principle



Raw Seismic Record

Phase Velocity - Frequency Spectrogram presenting Dispersion

1D Shear Wave Velocity Sounding from Inversion of Dispersion Curve

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

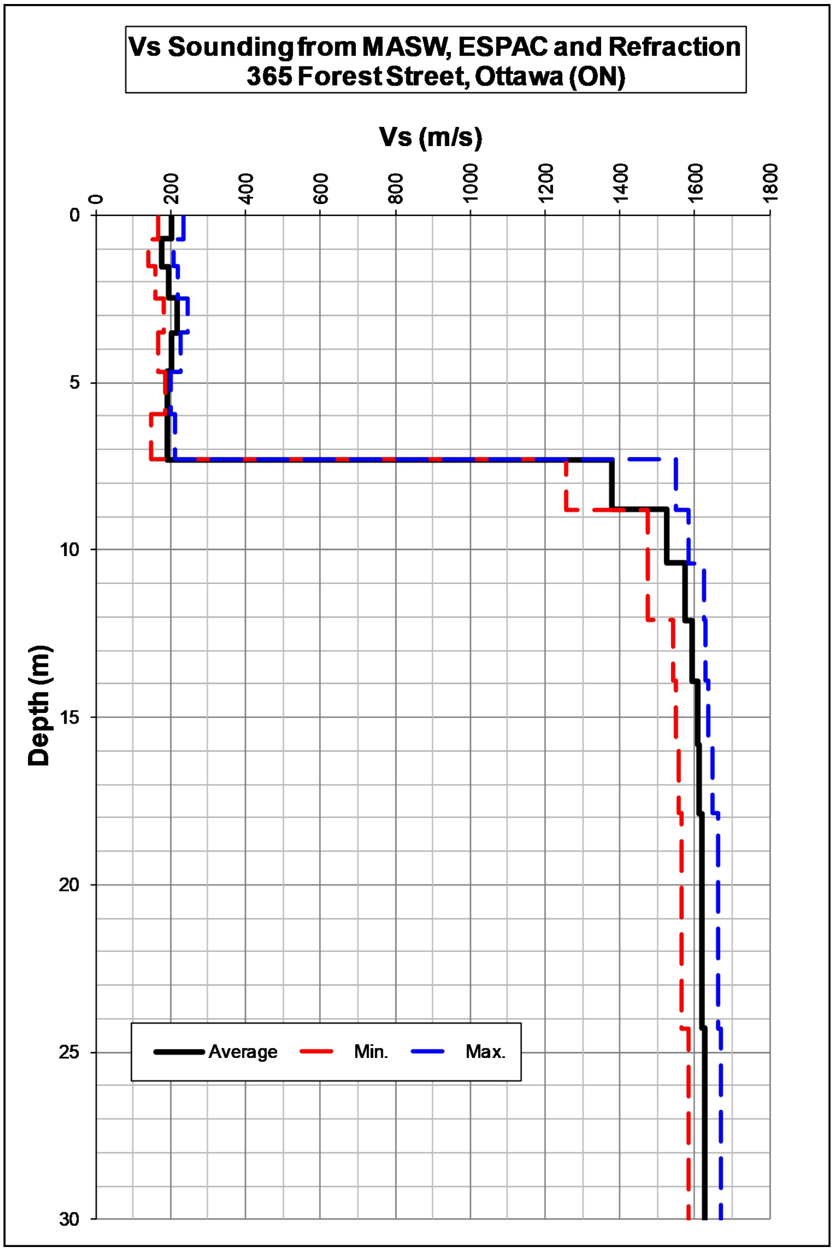


Figure 5: MASW Shear-Wave Velocities Sounding

TABLE 1
V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg Vs	Cumulative Delay	Average Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	165.3	201.0	235.9					
0.71	139.5	175.8	208.2	0.71	0.71	0.003554	0.003554	201.0
1.54	161.2	195.9	221.3	0.82	1.54	0.004688	0.008242	186.7
2.47	181.7	219.0	246.7	0.93	2.47	0.004767	0.013009	190.1
3.52	168.3	202.3	226.8	1.04	3.52	0.004766	0.017775	197.8
4.67	186.0	191.2	202.4	1.15	4.67	0.005704	0.023479	198.9
5.93	148.5	190.7	213.8	1.26	5.93	0.006610	0.030089	197.2
7.31	1258.0	1379.9	1548.7	1.37	7.31	0.007203	0.037291	196.0
8.79	1475.8	1524.5	1582.9	1.48	8.79	0.001075	0.038367	229.1
10.38	1525.2	1575.0	1624.9	1.59	10.38	0.001045	0.039412	263.5
12.09	1540.0	1593.8	1626.3	1.70	12.09	0.001081	0.040493	298.5
13.90	1550.1	1605.7	1637.2	1.81	13.90	0.001138	0.041631	333.9
15.82	1555.4	1611.5	1647.9	1.92	15.82	0.001198	0.042829	369.5
17.86	1563.9	1618.2	1662.5	2.03	17.86	0.001262	0.044090	405.0
24.29	1583.9	1627.7	1669.3	6.43	24.29	0.003973	0.048063	505.3
30				5.71	30.00	0.003511	0.051573	581.7

V_{S30} (m/s)	**581.7**
Class	**C**

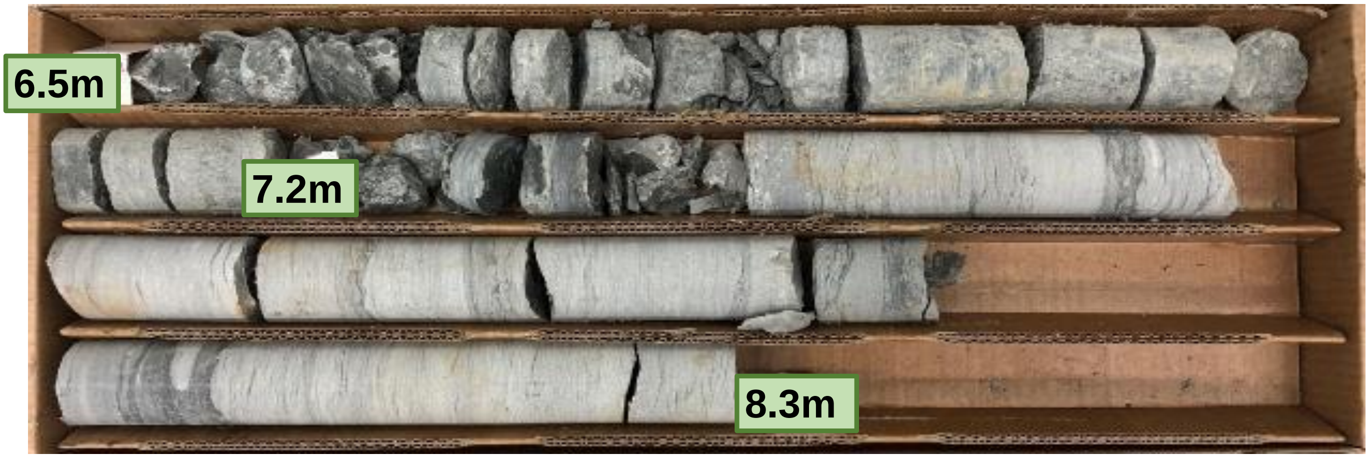
TABLE 2
V_{S30}* Calculation for the Site Class (foundation at 8.6 metres deep)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg Vs	Cumulative Delay	Average Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	165.3	201.0	235.9	**Consideration of foundation at 8.6 metres deep**				
0.71	139.5	175.8	208.2					
1.54	161.2	195.9	221.3					
2.47	181.7	219.0	246.7					
3.52	168.3	202.3	226.8					
4.67	186.0	191.2	202.4					
5.93	148.5	190.7	213.8					
7.31	1258.0	1379.9	1548.7					
8.6	1258.0	1379.9	1548.7					
8.79	1475.8	1524.5	1582.9	0.19	0.19	0.000139	0.000139	1379.9
10.38	1525.2	1575.0	1624.9	1.59	1.78	0.001045	0.001184	1507.6
12.09	1540.0	1593.8	1626.3	1.70	3.49	0.001081	0.002265	1539.7
13.90	1550.1	1605.7	1637.2	1.81	5.30	0.001138	0.003403	1557.8
15.82	1555.4	1611.5	1647.9	1.92	7.22	0.001198	0.004601	1570.3
17.86	1563.9	1618.2	1662.5	2.03	9.26	0.001262	0.005862	1579.2
24.29	1583.9	1627.7	1669.3	6.43	15.69	0.003973	0.009835	1594.9
38.6				14.31	30.00	0.008794	0.018629	1610.4

V_{S30}* (m/s)	**1610.4**
Class	**A**

Appendix C:
Bedrock Core Photographs

DRY BEDROCK CORES



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borehole no.	core runs	PROJECT	project no.
BH 19-01	Run 1: 6.5m-7.2m Run 2: 7.2m-8.3m	RESIDENTIAL DEVELOPMENT 365 FOREST AVENUE, OTTAWA, ON.	OTT-00252625-A0
date cored Apr 30, 2019		BEDROCK CORE PHOTOGRAPHS	FIG. C-1

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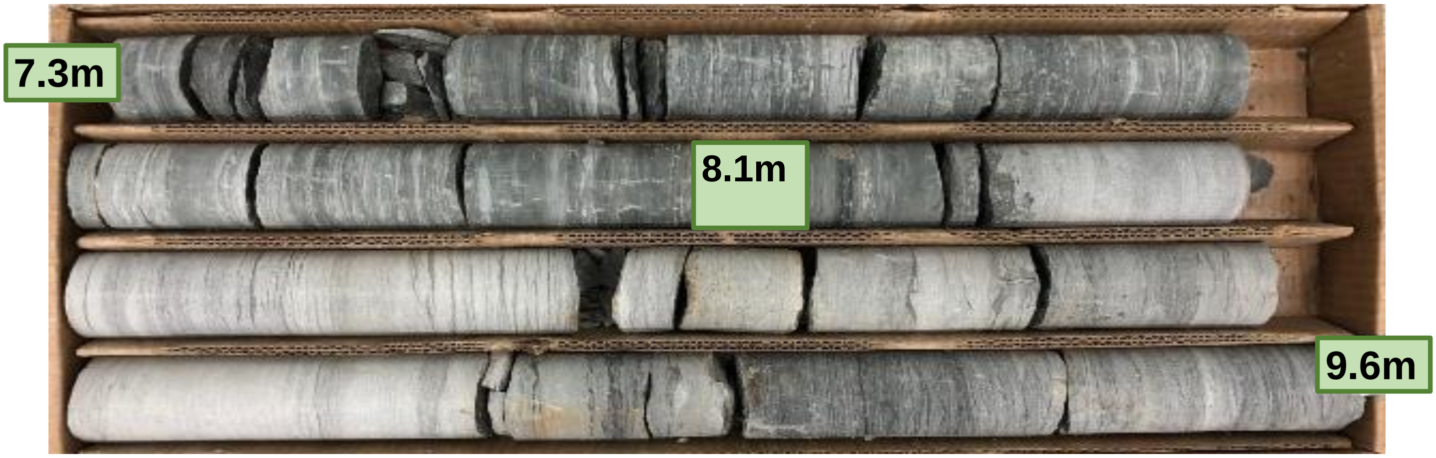
borehole no.	core runs	PROJECT	project no.
BH 19-03	Run 1: 7.0m-8.0m	RESIDENTIAL DEVELOPMENT 365 FOREST AVENUE, OTTAWA, ON.	OTT-00252625-A0
date cored Apr 30, 2019		BEDROCK CORE PHOTOGRAPHS	FIG. C-2

DRY BEDROCK CORES



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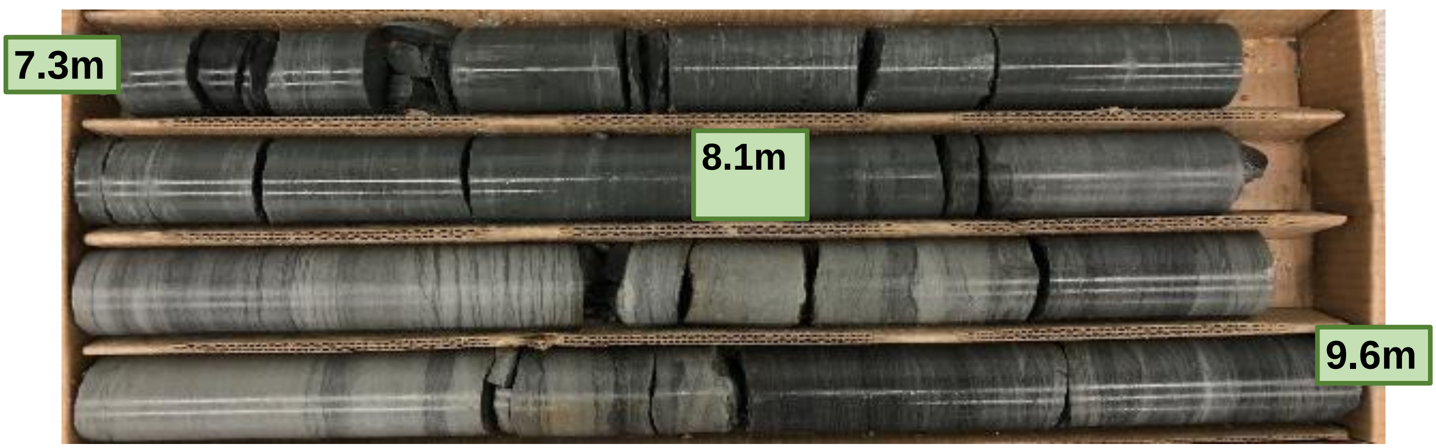

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borehole no.	core runs	PROJECT	project no.
BH 19-08	Run 1: 7.3m-8.1m Run 2: 8.1m-9.6m	RESIDENTIAL DEVELOPMENT 365 FOREST AVENUE, OTTAWA, ON.	OTT-00252625-A0
date cored Apr 25, 2019		BEDROCK CORE PHOTOGRAPHS	FIG. C-3

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borehole no.	core runs	PROJECT	project no.
BH 19-09	Run 1: 7.6m-8.7m	RESIDENTIAL DEVELOPMENT 365 FOREST AVENUE, OTTAWA, ON.	OTT-00252625-A0
date cored Apr 24, 2019		BEDROCK CORE PHOTOGRAPHS	FIG. C-4

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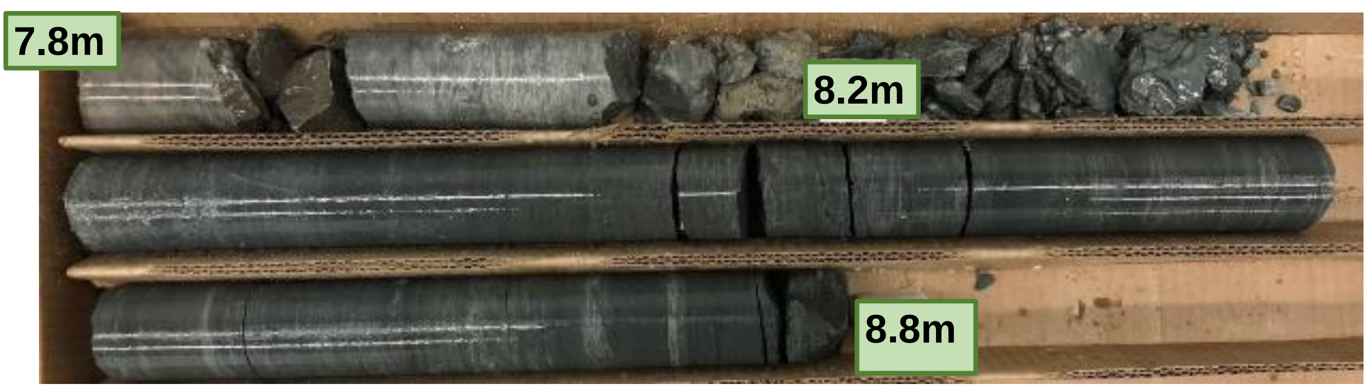

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borehole no.	core runs	PROJECT	project no.
BH 19-11	Run 1: 7.8m-8.2m Run 2: 8.2m-8.8m	RESIDENTIAL DEVELOPMENT 365 FOREST AVENUE, OTTAWA, ON.	OTT-00252625-A0
date cored Apr 29, 2019		BEDROCK CORE PHOTOGRAPHS	FIG. C-5

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