Preliminary Geotechnical Investigation

Client:
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Proposed Residential High-Rise Towers
1640 – 1660 Carling Avenue, Ottawa, Ontario

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Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the preliminary geotechnical investigation completed for the proposed residential high-rise towers to be located at 1640-1660 Carling Avenue, Ottawa, Ontario (Figure 1). Authorization to proceed with this geotechnical investigation was provided by RioCan Real Estate Investment Trust.

EXP also conducted Phase One and Two Environmental Site Assessments (ESAs) and a hydrogeological investigation of the site in conjunction with this preliminary geotechnical investigation. The Phase One and Two ESAs and the hydrogeological investigation are provided in separate reports.

The proposed development will consist of six (6), forty (40) -storey high-rise residential towers with a shared three (3) level underground parking garage. The design elevation of the lowest floor slab of the parking garage was not available at the time of this preliminary geotechnical investigation. For purposes of this preliminary geotechnical investigation, it is assumed that for a three (3) level underground parking garage, the lowest floor slab will be set at depths of 9.0 m to 10.0 m below existing grade. The design elevation of the final site grades was not available at the time of this preliminary geotechnical investigation. However, since the elevation of the current ground surface of the site is near the elevation of the adjacent roads and that the site is located in a well-established developed area of Ottawa, it is expected that the final site grades will generally match the existing grades and minimum grade raise will be required at the site as part of the proposed development.

The borehole fieldwork originally consisted of eighteen (18) boreholes (Borehole Nos. 1 to 13 and ENV-1 to ENV-5) located on site. Borehole Nos. 1, 3 and 6 could not be undertaken due to conflict with existing underground services. Therefore, a total of fifteen (15) boreholes were completed for this project. With the exception of Borehole Nos. ENV-3 and ENV-4, all of the boreholes are located outside. Borehole Nos. ENV-3 and ENV-4 are located inside the former automotive building. The boreholes were undertaken from May 4 to 12,2023. The boreholes were advanced to auger/casing refusal and termination depths of 2.2 m to 12.4 m below existing grade. Borehole Nos. 2,4, 5, 7, 9 and 11 were advanced to termination depths of 11.9 m to 12.4 m and the remaining boreholes were advanced to auger/casing refusal depths of 2.2 m to 3.8 m. The borehole fieldwork was supervised on a full-time basis by EXP. The information from the fifteen (15) boreholes was used in the preparation of this preliminary geotechnical engineering report, the Phase Two Environmental Site Assessment (ESA) report and the hydrogeological investigation report.

Based on the borehole information, the subsurface conditions at the site consist of an asphalt pavement structure (exterior boreholes), concrete floor slab (boreholes located inside the former automotive building) all underlain by fill, silty clay, glacial till and limestone bedrock. The limestone bedrock was contacted at 2.5 m to 3. 4 m depths (Elevation 76.4 m to Elevation 75.2 m). The groundwater level ranges from 2.2 m to 6.7 m (Elevation 76.4 m to Elevation 72.1 m).

Based on a review of the borehole information, the proposed buildings with three (3) level underground parking garage may be supported by strip and spread footings founded on the sound limestone bedrock. For footings founded on the sound limestone bedrock, the site classification for seismic site response is Class A, based on the 2012 Ontario Building Code as amended January 1,2020.

The borehole information indicates that compressible clays do not exist at the site. Therefore, from a geotechnical perspective, there is no restriction to raising the grades at the site.

The proposed buildings with three (3) level underground parking garage may be supported by conventional strip and spread footings founded on the sound limestone bedrock. Strip and spread footings founded on the sound limestone bedrock below 9.0 m to 10.0 m depths that is free of weathered zones, loose material, soft seams, fractures and voids may be designed for a factored geotechnical resistance at ultimate limit state (ULS) of 5.0 MPa. The factored geotechnical resistance at ULS includes a geotechnical resistance factor of 0.50. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement will be much larger than the recommended values for the factored geotechnical resistance at ULS. Therefore, for footings founded on bedrock, 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.

The lowest floor level of the parking garage for the proposed buildings is assumed to be at 9.0 m to 10.0 m depths below existing grade. Based on the borehole information, the lowest floor slab of the buildings will be founded on the limestone bedrock and may be constructed as a concrete slab-on-grade or as a paved surface as indicated in the attached preliminary geotechnical report.

The lowest floor level for the proposed three (3) level below grade parking garage is anticipated to be located below the groundwater level. Therefore, underfloor and perimeter drainage systems will be required for the proposed below grade parking garage.

The subsurface basement walls will need to be designed to resist lateral earth pressure (force) for the static and seismic conditions. The subsurface basement walls should have a perimeter drainage system.

Excavations within the soils may be undertaken using heavy equipment capable of removing possible debris, cobbles, boulders and large slabs of rock. The upper depths of the weathered/highly fractured zones of the limestone bedrock may be excavated using a hoe ram for removal of small quantities of the bedrock; however, this process is expected to be very slow. The excavation of the sound limestone bedrock is anticipated to extend deep below the bedrock surface and may be undertaken by line drilling and blasting method. Should blasting not be permitted, the excavation of the limestone bedrock would have to be undertaken by line drilling. Specialized contractors bidding on this project should decide on their own the most preferred rock removal method; hoe ramming or line drilling and blasting.

All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. It is expected that due to the anticipated significant depth of the excavation for the proposed buildings and the proximity of the excavation to existing buildings and infrastructure (roadways and underground municipal services), the excavations within the soils will likely have to be undertaken within the confines of a shoring system. The excavations are also anticipated to extend deep into the limestone bedrock. The excavation side slopes in the upper depths of the weathered/highly fractured zones of the limestone bedrock may be cut back at a 1H:1V gradient from the surface of the sound limestone bedrock. The excavation side slopes in the sound limestone bedrock may be undertaken with near vertical sides subject to examination by a geotechnical engineer. Zones of the weathered and fractured rock faces for the portion of the excavation within the bedrock may require support in the form of rock bolts to maintain the integrity of the rock face in conjunction with a wire mesh system and/or shotcrete. Excavation of the bedrock 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.

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

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

1. Introduction

EXP Services Inc. (EXP) is pleased to present the results of the preliminary geotechnical investigation completed for the proposed residential high-rise towers to be located at 1640-1660 Carling Avenue, Ottawa, Ontario (Figure 1). Authorization to proceed with this preliminary geotechnical investigation was provided by RioCan Real Estate Investment Trust.

EXP also conducted Phase One and Two Environmental Site Assessments (ESAs) and a hydrogeological investigation of the site in conjunction with the geotechnical investigation. The Phase One and Two ESAs and the hydrogeological investigation are provided in separate reports.

The proposed development will consist of six (6), forty (40)-storey high-rise residential towers with a shared three (3) level underground parking garage. The design elevation of the lowest floor slab of the parking garage was not available at the time of this preliminary geotechnical investigation. For purposes of this preliminary geotechnical investigation, it is assumed that for a three (3) level underground parking garage, the lowest floor slab will be set at depths of 9.0 m to 10.0 m below existing grade. The design elevation of the final site grades was not available at the time of this preliminary geotechnical investigation. However, since the elevation of the current ground surface of the site is near the elevation of the adjacent roads and that the site is located in a well-established developed area of Ottawa, it is expected that the final site grades will generally match the existing grades and minimum grade raise will be required at the site as part of the proposed development.

This preliminary geotechnical investigation was undertaken to:

a) Establish the subsurface soil, bedrock and groundwater conditions at the eighteen (18) boreholes located on the site,

b) Provide classification of the site for seismic site response in accordance with the requirements of the 2012 Ontario Building Code (as amended January 1,2020) 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) Provide lateral earth pressure (force) and soil parameters for subsurface (basement) walls for the static and seismic (dynamic) conditions,

f) Comment on slab on grade construction,

g) Discuss anticipated excavation conditions and de-watering requirements during construction,

h) Comment on backfilling requirements and geotechnical assessment of the suitability of on-site soils for backfilling purposes,

i) Provide pavement structures for outdoor access roads and parking lots,

j) Comment on subsurface concrete and steel requirements; and,

k) Discuss tree planting restrictions.

The comments and recommendations given in this report are preliminary in nature and assume 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 site is located at the northeast corner of the Carling Avenue and Clyde Avenue North intersection in Ottawa and is identified by the addresses, 1640 and 1660 Carling Avenue. The portion of the site with address, 1660 Carling Avenue, is currently occupied by a two-storey commercial-type building, which includes both retail space, an automotive garage, and surface parking. The portion of the site with address, 1640 Carling Avenue, comprises of a single-storey restaurant currently operating as a Boston Pizza with surface parking. The entire site has a total area of approximately 2.3 hectares.

The topography of the site is relatively flat with the elevation of the ground surface at the boreholes located on the site ranging from Elevation 79.74 m to Elevation 78.62 m.

3. Geology of the 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 glacial till.

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 limestone bedrock (with some shaly partings) of the Ottawa formation.

4. Procedure

4.1 Fieldwork

The borehole fieldwork originally consisted of eighteen (18) boreholes (Borehole Nos. 1 to 13 and ENV-1 to ENV-5) located on site. Borehole Nos. 1, 3 and 6 could not be undertaken due to conflict with existing underground services. Therefore, a total of fifteen (15) boreholes were completed for this project. With the exception of Borehole Nos. ENV-3 and ENV-4, all of the boreholes are located outside. Borehole Nos. ENV-3 and ENV-4 are located inside the former automotive building. The boreholes were undertaken from May 4 to 12,2023. The boreholes were advanced to auger/casing refusal and termination depths of 2.2 m to 12.4 m below existing grade. Borehole Nos. 2,4, 5, 7, 9 and 11 were advanced to termination depths of 11.9 m to 12.4 m and the remaining boreholes were advanced to auger/casing refusal depths of 2.2 m to 3.8 m. The borehole fieldwork was supervised on a full-time basis by EXP. The information from the fifteen (15) boreholes was used in the preparation of this preliminary geotechnical engineering report, the Phase Two Environmental Site Assessment (ESA) report and the hydrogeological investigation report.

The locations and geodetic elevations of the boreholes were established by a survey crew from EXP and are shown on the borehole location plan, Figure 2. The locations and ground surface elevations of Borehole Nos. ENV-3, ENV-4 and Borehole No. 5 shown in Figure 2 should be considered approximate.

Prior to the fieldwork, the locations of the boreholes were cleared of any public and private underground services. The boreholes were drilled using a CME-55 track-mounted drill rig equipped with continuous flight hollow-stem auger equipment and bedrock coring capabilities. Borehole Nos. ENV-3 and ENV-4 were undertaken using portable manual drilling equipment. Standard penetration tests (SPTs) were performed in the boreholes on a continuous basis to 0.75 m depth interval, the SPT N-values were recorded (with the exception of Borehole Nos. ENV-3 and ENV-4) and the soil samples were retrieved by the split-spoon sampler. The presence of the bedrock was proven in six (6) boreholes, Borehole Nos. 2,4,5,7,9 and 11, by conventional coring techniques using the NQ and HQ size core barrel. A field record of wash water return, colour of wash water and any sudden drops of the core barrel were kept during coring operations.

Monitoring wells (32 mm or 50 mm diameters) were installed in seven (7) boreholes, Borehole Nos. 2, 4, 9, 11, ENV-1, ENV-2 and ENV-4, for long-term monitoring of the groundwater level and for the sampling of the groundwater as part of the Phase Two ESA and the hydrogeological investigation. The monitoring wells were installed in accordance with EXP standard practice, and the installation configuration is documented on the respective borehole log. The boreholes were backfilled upon completion of the field work and the installation of the monitoring wells.

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

4.2 Laboratory Testing Program

The soil samples were visually examined in the laboratory by a geotechnical engineer. The rock cores were visually examined and logged in the laboratory in accordance with Section 3.2 of the 2006 Canadian Foundation Engineering Manual (Fourth Edition, CFEM).Soil classification consisted of classifying the main constituents of the soils in accordance with the Unified Soil Classification System (USCS) using the soil group name and symbol and by the modified Burmister Soil Classification System (2006 Fourth Edition of the Canadian Foundation Engineering Manual (CFEM)) to classify the minor constituents of the soil using modifiers and adjectives (such as trace and some). The rock cores were visually examined, logged, and classified by strength and rock quality designation value (RQD) in accordance with Section 3.2 of the 2006 Canadian Foundation Engineering Manual (Fourth Edition, CFEM).

A summary of the soil and bedrock geotechnical laboratory testing program is shown in Table I.

Table I: Summary of Laboratory Testing Program

Type of Test	Number of Tests Completed
Soil Samples	
Moisture Content Determination	56
Unit Weight Determination	7
Grain Size Analysis	6
Atterberg Limit Determination	1
Chemical Tests for Corrosion Potential (pH, sulphate, chloride and resistivity)	1
Bedrock Cores	
Unit Weight Determination	21
Unconfined Compressive Strength Test	21
Chemical Tests for Corrosion Potential (pH, sulphate, chloride and resistivity)	3

4.3 Seismic Shear Wave Velocity Sounding Survey

A seismic shear wave velocity sounding survey of the site was conducted on site on June 14,2023 by Geophysics GPR International Inc.

5. Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface conditions and groundwater levels from this preliminary geotechnical investigation are given on the attached Borehole Logs, Figures 3 to 17 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.

Reference is made to the Phase One and Two ESAs and the hydrogeological investigation reports prepared concurrently with this preliminary geotechnical investigation regarding the environmental aspects of the subsurface soils and groundwater.

It should be noted that the soil and bedrock 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 “Notes on Sample Descriptions” preceding the borehole logs form an integral part of this report and should be read in conjunction with this preliminary geotechnical report.

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

5.1 Pavement Structure

With the exception of the interior boreholes, Borehole Nos. ENV-3 and ENV-4, the remaining exterior boreholes are located in a paved area. The pavement structure consists of 65 mm to 180 mm thick asphaltic concrete underlain by 200 mm to 660 mm granular fill base. The pavement structure extends to depths of 0.3 m to 0.8 m (Elevation 78.6 m to Elevation 77.9 m). In Borehole Nos. 5 and ENV-5, the granular base is 1125 mm and 1300 mm thick and the pavement structure extends to 1.2 m and 1.4 m depths (Elevation 77.5 m and Elevation 77.4 m). The granular fill base consists of a mixture of silty sand and crushed gravel. The granular fill base in Borehole Nos. 4 and 8 contains concrete pieces. Based on the standard penetration test (SPT) values of 6 to 64, the granular fill base is in a loose to very dense state. The moisture content of the granular base fill is 2 percent to 10 percent. The results from the grain size analysis conducted on one (1) sample of the granular fill is summarized in Table II. The grain size distribution curve is shown in Figure 18.

Table II: Summary of Results from Grain-Size Analysis – Granular Fill Base Sample

Borehole No. (BH) – Sample No. (SS)	Depth (m)	Grain-Size Analysis (%)			Soil Classification
		Gravel	Sand	Fines (Silt and Clay)	
BH 11 – SS1	0.1 – 0.6	40	47	13	Sand and Gravel (SM), Some Silt

Based on a review of the results from the grain size analysis of one (1) sample, the fill may be classified as sand and gravel (SM) with some silt.

5.2 Concrete Floor Slab

Borehole Nos. ENV-3 and ENV-4 are located inside the former automotive building and the thickness of the concrete floor slab is 230 mm and 255 mm. The concrete floor slab in Borehole No. ENV-3 is underlain by a 230 mm thick silty sand and crushed gravel fill layer that is further underlain by a 255 mm thick buried concrete layer.

5.3 Fill

The exterior pavement structure and the interior concrete floor slab are underlain by fill in all of the boreholes that extends to depths of 1.4 m to 2.6 m (Elevation 77.6 m to Elevation 76.4 m). The fill in Borehole No. 7 extends to a 3.1 m depth (Elevation 75.8 m). In Borehole No. ENV-4, the borehole met casing refusal within the fill on inferred cobbles, boulders or bedrock at a 3.8 m depth (Elevation 75.3 m) below existing grade (Elevation 75.3 m). The fill consists of silty sand with gravel. The fill contains a topsoil (organic) seam and rootlets in Borehole No. 5, organic soil pockets in Borehole No. 9 and organic silty sand in Borehole No. 12. The fill also contains brick and concrete pieces in Borehole Nos. 4, 10 and 11. The standard penetration test (SPT) N-values range from 4 to 40 indicating the fill is in a loose to dense state. In Borehole No. 11, the SPT N-value is high for a low sampler penetration suggesting that the soil sampler made contact with debris within the fill, such as a concrete piece or possibly on a cobble or boulder; the fill may contain cobbles and/or boulders. The moisture content of the fill ranges from 4 percent to 24 percent.

The results from the grain-size analysis conducted on one (1) sample of the fill is summarized in Table III. The grain-size distribution curve is shown in Figure 19.

Table III: Summary of Results from Grain-Size Analysis – Fill Sample

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)				Soil Classification
		Gravel	Sand	Silt	Clay	
BH 7 – SS3	1.5 – 2.1	6	57	28	9	Silty Sand (SM) with Trace Gravel and Clay

Based on a review of the results from the grain size analysis of one (1) sample, the fill may be classified as a silty sand (SM) with trace gravel and clay.

5.4 Silty Clay

Silty clay was contacted in Borehole No. 5 below the fill at a 2.2 m depth (Elevation 76.4 m) and extends to a 3.0 m depth (Elevation 75.6 m). Based on the SPT N-value of 6, the silty clay has a firm consistency. The natural moisture content of the silty clay is 40 percent. The unit weight of the silty clay is 19.4 kN/m^3.

The results from the grain-size analysis and Atterberg limit determination conducted on one (1) sample of the silty clay are summarized in Table IV. The grain-size distribution curve is shown in Figure 20.

Table IV: Summary of Results from Grain-Size Analysis and Atterberg Limit Determination – Silty Clay Sample

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)				Atterberg Limits (%)				Soil Classification (USCS)
		Gravel	**Sand**	**Silt**	**Clay**	**Moisture Content**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	
BH 5 – SS4	2.3 – 2.9	0	17	40	43	40	42	19	23	Silty Clay of Medium Plasticity (CL) with Some Sand

Based on a review of the results of the grain-size analysis and Atterberg limits, the soil may be classified as a silty clay of medium plasticity (CL) with some sand.

5.5 Glacial Till

With the exception of Borehole Nos. 7 and ENV-4, the fill and silty clay are underlain by glacial till contacted at 1.4 m to 3.0 m depths (Elevation 77.6 m to Elevation 75.6 m). In Borehole Nos. 2, 4, 5, 7, 9 and 11, the glacial till extends to depths ranging from 2.5 m to 3.4 m (Elevation 76.4 m to Elevation 75.2 m). In Borehole Nos. 8, 10, 12, 13 and ENV-1 to ENV-5, auger/casing refusal occurred at 2.2 m to 3.8 m depths (Elevation 76.6 m to Elevation 75.3 m) on inferred cobbles and/or boulders within the glacial till or on inferred bedrock. The glacial till consists of silty sand with gravel and possible cobbles and boulders. Based on the SPT N-values of 12 to 50, the glacial till is in a compact to very dense state. High SPT N-values for low sampler penetration were recorded at some depths within the glacial till and may be a result of the sampler making contact on a cobble, boulder or rock piece within the glacial till. Therefore, the glacial till may contain possible cobbles, boulders and/or rock slabs. The natural moisture content of the glacial till ranges from 6 percent to 19 percent. The natural unit weight of the glacial till is 21.9 kN/m^3 to 24.0 kN/m^3.

The results from the grain-size analysis conducted on three (3) samples of the glacial till are summarized in Table V. The grain-size distribution curves are shown in Figures 21 to 23.

Table V: Summary of Results from Grain-Size Analysis and Atterberg Limit Determination – Glacial Till Samples

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)				Soil Classification
		Gravel	Sand	**Silt**	**Clay**	
BH 4 – SS4	2.3 – 2.9	11	49	30	10	Silty Sand (SM), Some Gravel and Clay
BH 8 – SS3	1.5 – 2.1	6	52	34	8	Silty Sand (SM), Trace Gravel and Clay
ENV-5	2.3 – 2.9	16	45	30	9	Silty Sand (SM) , Some Gravel, Trace Clay

Based on a review of the results of the grain-size analysis of the three (3) samples, the glacial till may be classified as a silty sand with trace to some gravel and clay. As previously mentioned, the glacial till may contain possible cobbles, boulders and/or rock pieces (slabs).

5.6 Limestone Bedrock

Auger and casing refusal was met on inferred cobbles, boulders or bedrock in Borehole Nos. 8, 10,12,13 and ENV-1 to ENV -5 at 2.2 m to 3.8 m depths (Elevation 76.6 m to Elevation 75.3 m). The presence of the bedrock was proven by coring the bedrock in Borehole Nos. 2,4,5,7,9 and 11 and found to be at depths ranging from 2.5 m to 3.4 m (Elevation 76.4 m to Elevation 75.2 m). Based on a review of the bedrock cores, the bedrock is considered to be limestone with shaley partings. The bedrock is weathered in the upper 150 mm in Borehole Nos. 2 and 4 and the upper 800 mm in Borehole No. 9. The actual and inferred bedrock depths (elevations) are summarized in Table VI. Photographs of the bedrock cores are shown in Appendix A.

Table VI: Summary of Bedrock Depths (Elevations)		
Borehole No. (BH)	Ground Surface Elevation (m)	Actual and Inferred Bedrock Depth (Elevation) m
BH-2	78.80	2.5 (76.3)
BH-4	78.87	3.4 (75.5)
BH-5	78.63	3.4 (75.2)
BH-7	78.92	3.1 (75.8)
BH-8	79.10	2.5 (76.6) – Inferred Bedrock (Auger Refusal)
BH-9	78.87	2.5 (76.4)
BH-10	78.86	3.5 (75.4) – Inferred Bedrock (Auger Refusal)
BH-11	78.90	3.0 (75.9)
BH-12	78.73	2.7 (76.0) – Inferred Bedrock (Auger Refusal)
BH-13	78.96	3.0 (76.0)
ENV-1	78.62	2.2 (76.4) - Inferred Bedrock (Auger refusal)
ENV-2	78.96	2.7 (76.3) -Inferred Bedrock (Auger Refusal)
ENV-3	79.08	3.5 (75.6) – Inferred Bedrock (Auger Refusal)
ENV-4	79.08	3.8 (75.3) – Inferred Bedrock (Auger Refusal)
ENV-5	78.85	3.1 (75.8) – Inferred Bedrock (Auger Refusal)

Based on the bedrock coring results, the total core recovery (TCR) ranges from 86 percent to 100 percent. The rock quality designation (RQD) ranges from 50 percent to 100 percent with the upper 300 mm to 800 mm of the bedrock having an RQD value of 50 percent to 83 percent. Based on the overall RQD values, the bedrock is considered to be of a fair to excellent quality.

The unit weight and unconfined compressive strength of the selected sections of the rock cores sections are summarized in Table VII.

Table VII: Summary of Unconfined Compressive Strength Test Results – Bedrock Cores				
Borehole No. (BH) – Run No.	Depth (m)	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)	Classification of Rock with respect to Strength
BH-2 -Run 1	2.8 – 3.0	25.9	137.9	Very Strong (R5)
BH-2 -Run 3	5.2 – 5.4	26.3	169.1	Very Strong (R5)
BH-2 – Run 6	9.6 – 9.8	25.0	89.9	Strong (R4)
BH-2 – Run 7	11.0 – 11.2	25.5	77.2	Strong (R4)

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

Borehole No. (BH) – Run No.	Depth (m)	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)	Classification of Rock with respect to Strength
BH-4 -Run 5	9.0 – 9.2	25.8	130.8	Very Strong (R5)
BH-4 – Run 6	11.2 – 11.4	26.2	123.9	Very Strong (R5)
BH-5 – Run 2	4.9 – 5.1	26.6	111.4	Very Strong (R5)
BH-5 – Run 4	7.6 – 7.8	26.8	213.9	Very Strong (R5)
BH-5 – Run 5	8.9 – 9.1	26.1	155.7	Very Strong (R5)
BH-5 – Run 6	11.6 – 11.8	26.9	229.1	Very Strong (R5)
BH-7 – Run 5	9.9 – 10.1	26.9	123.9	Very Strong (R5)
BH-7 – Run 6	11.6 – 11.8	26.4	95.9	Strong (R4)
BH-9 – Run 1	2.9 – 3.1	25.8	120.4	Very Strong (R5)
BH-9 – Run 3	5.6 – 5.8	25.6	90.4	Strong (R4)
BH-9 – Run 5	8.2 – 8.4	26.6	150.0	Very Strong (R5)
BH-9 – Run 6	9.7 – 9.9	25.9	97.9	Strong (R4)
BH-9 -Run 7	10.9 – 11.1	26.8	139.2	Very Strong (R5)
BH-11-Run 2	4.0 – 4.2	25.5	159.1	Very Strong (R5)
BH-11-Run 5	8.8 – 9.0	26.5	138.6	Very Strong (R5)
BH-11-Run 6	10.2 – 10.4	25.6	110.4	Very Strong (R5)
BH-11-Run 7	11.2 – 11.4	26.4	166.2	Very Strong (R5)

The unit weight of the bedrock ranges from 25.0 kN/m^3 to 26.9 kN/m^3. The unconfined compressive strength of the bedrock ranges from 77.2 MPa to 229.1 MPa and the average unconfined compressive strength is 134.8 MPa. The unconfined compressive strength of the limestone bedrock below the lowest floor slab of the proposed buildings at a 9.0 m to 10.0 m depth below existing grade ranges from 77.2 MPa to 229.1 MPa with an average unconfined compressive strength of 128.3 MPa. A review of the test results in Table VII indicates the strength of the rock may be classified as strong to very strong in accordance with the Canadian Foundation Engineering Manual (CFEM), Fourth Edition, 2006.

5.7 Groundwater Level Measurements

A summary of the groundwater level measurements taken on May 31,2023 in the boreholes equipped with monitoring wells is shown in Table VIII.

Table VIII: Summary of Groundwater Level Measurements

Borehole No. (BH)	Ground Surface Elevation (m)	Elapsed Time in Days from Date of Installation	Depth Below Ground Surface (Elevation), m
BH-2	78.80	23 days	6.7 (72.1)
BH-4	78.87	26 days	6.1 (72.8)
BH-9	78.87	21 days	4.3 (74.6)
BH-11	78.90	22 days	4.5 (74.4)
BH-ENV-1	78.62	27 days	2.2 (76.4)
BH-ENV-2	78.96	27 days	2.7 (76.3)
BH-ENV-3	79.08	19 days	3.0 (76.1)
BH-ENV-4	79.08	19 days	3.1 (76.0)

The groundwater level ranges from 2.2 m to 6.7 m (Elevation 76.4 m to Elevation 72.1 m).

Water levels were determined in the monitoring wells at the times and under the conditions noted above. 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 seismic shear wave velocity sounding report prepared by GPR is shown in Appendix B. Based on a review of seismic shear wave velocity sounding survey results in conjunction with the 2012 Ontario Building Code (OBC; as amended January 1,2020), for footings that will support the proposed buildings at depths below 9.0 m to 10.0 m on the limestone bedrock (as indicated in Section 8 of this report), the site classification for seismic site response is Class A.

6.2 Liquefaction Potential of Soils

The site is not underlain by liquefiable soils.

7. Grade Raise Restrictions

The borehole information indicates that compressible clays do not exist at the site. Therefore, from a geotechnical perspective, there is no restriction to raising the grades at the site.

8. Foundation Considerations

The design elevation of the lowest floor slab of the parking garage was not available at the time of this preliminary geotechnical investigation. For purposes of this preliminary geotechnical investigation, it is assumed that for a three (3) level underground parking garage, the lowest floor slab will be set at depths of 9.0 m to 10.0 m below existing grade. Based on a review of the borehole information, for the lowest floor slab set at 9.0 m to 10.0 m below existing grade, the slab will be founded on the limestone bedrock. It is considered feasible to support the proposed buildings by strip and spread footings founded below the lowest floor slab, below the 9.0 m to 10.0 m depths, on the competent sound limestone that is free of weathered zones, loose material, soft seams, fractures and voids.

Auger and casing refusal was met on inferred cobbles, boulders or bedrock in Borehole Nos. 8, 10,12,13 and ENV-1 to ENV -5 at 2.2 m to 3.8 m depths (Elevation 76.6 m to Elevation 75.3 m). The presence of the bedrock was proven by coring the bedrock in Borehole Nos. 2,4,5,7,9 and 11 and found to be at 2.5 m to 3.4 m depths (Elevation 76.4 m to Elevation 75.2 m). Based on a review of the bedrock cores, the bedrock is considered to be limestone with shaley partings. The bedrock is weathered in the upper 150 mm of the bedrock in Borehole Nos. 2 and 4 and the upper 800 mm of the bedrock in Borehole No. 9. The inferred and actual bedrock depths (elevations) are summarized in Table IX.

Table IX: Summary of Bedrock Depths (Elevations)

Borehole No. (BH)	Ground Surface Elevation (m)	Bedrock Depth (Elevation) m
BH-2	78.80	2.5 (76.3)
BH-4	78.87	3.4 (75.5)
BH-5	78.63	3.4 (75.2)
BH-7	78.92	3.1 (75.8)
BH-8	79.10	2.5 (76.6) – Inferred Bedrock (Auger Refusal)
BH-9	78.87	2.5 (76.4)
BH-10	78.86	3.5 (75.4) – Inferred Bedrock (Auger Refusal)
BH-11	78.90	3.0 (75.9)
BH-12	78.73	2.7 (76.0) – Inferred Bedrock (Auger Refusal)
BH-13	78.96	3.0 (76.0)
ENV-1	78.62	2.2 (76.4) - Inferred Bedrock (Auger refusal)
ENV-2	78.96	2.7 (76.3) -Inferred Bedrock (Auger Refusal)
ENV-3	79.08	3.5 (75.6) – Inferred Bedrock (Auger Refusal)
ENV-4	79.08	3.8 (75.3) – Inferred Bedrock (Auger Refusal)
ENV-5	78.85	3.1 (75.8) – Inferred Bedrock (Auger Refusal)

Strip and spread footings founded on the sound limestone bedrock below 9.0 m to 10.0 m depths that is free of weathered zones, loose material, soft seams, fractures and voids may be designed for a factored geotechnical resistance at ultimate limit state (ULS) of 5.0 MPa. The factored geotechnical resistance at ULS includes a geotechnical resistance factor of 0.50. The Serviceability Limit State

(SLS) bearing pressure of the bedrock, required to produce 25 mm settlement will be much larger than the recommended values for the factored geotechnical resistance at ULS. Therefore, for footings founded on bedrock, 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.

Footings at different elevations in sound bedrock should be located such that the higher footing is located 6V:1H from the limit of the footing excavation in the sound bedrock.

All footing beds should be examined by a geotechnician working under the direction of a geotechnical engineer to ensure that the founding limestone bedrock is capable of supporting the factored geotechnical resistance at ULS and that the footings have been properly prepared. Where weathered zones, loose material, soft seams, fractures or voids are present in the limestone bedrock subgrade for the footing, sub-excavation will be required to the underlying more competent bedrock with the footings stepped down to the more competent bedrock.

The footings for the proposed buildings will be protected from frost action since they are anticipated to be located at a depth greater than the required 1.5 m depth for frost protection for heated buildings.

The recommended factored geotechnical resistances at ULS for the foundations have been calculated by EXP from the borehole information for the design stage only. The preliminary geotechnical 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 preliminary geotechnical 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 and Drainage Requirements

The lowest floor level of the parking garage for the proposed buildings is assumed to be at 9.0 m to 10.0 m depths below existing grade. Based on the borehole information, the lowest floor slab of the buildings will be founded on the limestone bedrock and may be constructed as a concrete slab-on-grade or as a paved surface. The concrete and asphalt pavement structures indicated below are for light duty traffic only (cars). EXP can provide concrete and asphalt pavement structures for heavy duty traffic (cars and trucks), if required.

The lowest floor level for the parking garage is anticipated to be located below the groundwater level. Therefore, underfloor and perimeter drainage systems will be required for the proposed below grade 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 thick bed of 19 mm sized clear stone covered on top and sides with 150 mm thick clear stone that is fully wrapped with 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 thick clear stone fully wrapped with a geotextile membrane. The perimeter and underfloor drains should be connected to separate sumps equipped with backup pumps and generators in case of mechanical failure and/or power outage, so that at least one system would be operational should the other fail.

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

9.1 Lowest Floor Level as a Concrete Surface

The subgrade is anticipated to consist of limestone bedrock. The limestone bedrock should be examined by EXP and any loose/soft zones of the bedrock should be excavated and removed.

Following approval of the bedrock subgrade, the concrete slab for light duty traffic (cars only) may be constructed as follows:

- 150 mm thick concrete with 32 MPa compressive strength and air content of 5 percent to 8 percent, over
- 150 mm thick layer of Ontario Provincial Standard Specification (OPSS) 1010 Granular A compacted to 100 percent standard Proctor maximum dry density (SPMDD); over
- 300 mm minimum thick layer of OPSS 1010 Granular B Type II compacted to 100 percent SMPDD.

The concrete slab should be reinforced, and adequate saw cuts should be provided in the floor slab to control cracking. Additional recommendations can be provided once the final design of the lowest floor level is available.

9.2 Lowest Floor Level as a Paved Surface

The subgrade is anticipated to consist of limestone bedrock. The limestone bedrock should be examined by EXP and any loose/soft zones of the bedrock should be excavated and removed.

Following approval of the bedrock subgrade, the asphalt pavement structure for light duty traffic (cars only) may be constructed on the bedrock subgrade as follow:

- 65 mm thick layer of asphaltic concrete consisting of HL3/SP12.5 – The asphaltic concrete should be placed and compacted as per OPSS 310 and 313 and should be designed in accordance with OPSS 1150/1151, over
- 150 mm thick layer of OPSS Granular A compacted to 100 percent SPMDD; over
- 450 mm thick layer of OPSS Granular B Type II compacted to 100 percent SPMDD.

10. Lateral Earth Pressures Against Subsurface Walls

The subsurface basement walls should be backfilled with free draining material, such as OPSS Granular B Type II compacted to 95 percent SPMDD and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the walls. The walls will be subjected to lateral static and dynamic (seismic) earth forces. The expressions below assume free draining backfill material, a perimeter drainage system, level backfill surface behind the wall and vertical face on the back side of the wall.

Equation (i) will be applicable for the portion of the subsurface wall in the overburden (soil). Equation (ii) will be applicable for 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 applied at the ground surface should be considered as surcharge when computing lateral pressure using equation (ii).

Lateral static earth pressure, p, for subsurface basement wall in overburden soil:

$$p = k\,(\gamma h + q) \text{ ------------- (i) - where}$$

k = lateral earth pressure coefficient for 'at rest' condition = 0.50
γ = unit weight of backfill = 22 kN/m^3
h = depth of interest below ground surface (m)
q = any surcharge acting at ground surface (kPa)

Lateral static earth pressure, σ_n at depth, z, below the top of the wall, for subsurface basement wall in bedrock due to narrow earth backfill between subsurface wall and bedrock face:

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

γ = unit weight of backfill = 22 kN/m
B = backfill width (m)
z = depth from top of wall (m)
δ = friction angle between the backfill and wall and backfill 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 traffic, equipment/materials at ground surface behind the wall (kPa)

The lateral dynamic earth force (dynamic thrust) due to seismic loading may be computed from the equation given below:

$$\Delta_{Pe} = \gamma h^2 \frac{a_h}{g} F_b \text{ ------------------------------ (iii)}$$

where Δ_{Pe} = dynamic thrust in kN/m of wall
h = height of basement wall against soil above the bedrock surface (m)
γ = unit weight of soil = 22 kN/m^3
$\frac{a_h}{g}$ = seismic coefficient = 0.281
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.

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. Full coverage of the exterior face of the basement walls 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 non-woven geotextile separation membrane 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. The solid pipe inside the building should be connected to a separate sump from the sumps used for the perimeter and underfloor drains, so that this system would be operational should one of the other drainage systems fail.

All subsurface walls should be waterproofed.

11. Excavations and De-Watering Requirements

11.1 Excess Soil Management

Ontario Regulation 406/19 specifies protocols that are required for the management and disposal of excess soils. As set forth in the regulation, specific analytical testing protocols need to be implemented and followed based on the volume of soil to be managed and the requirements of the receiving site. The testing protocols are specific as to whether the soils are stockpiled or in situ. In either scenario, the testing protocols are far more onerous than have been historically carried out as part of standard industry practices. These decisions should be factored in and accounted for prior to the initiation of the project-defined scope of work. EXP would be pleased to assist with the implementation of a soil management and testing program that would satisfy the requirements of Ontario Regulation 406/19.

11.2 Excavations

11.2.1 Overburden Soil Excavation

Excavations for the construction of the proposed residential high-rise towers is expected to extend below a 9.0 m to 10.0 m depths below the existing ground surface. These excavations will extend through the fill, silty clay and glacial till and deep into the limestone bedrock. The excavations are anticipated to be below the groundwater level.

Excavations within the soils may be undertaken using heavy equipment capable of removing debris, cobbles, boulders and possible large slabs of rock.

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 3 and as such must be cut back at 1H:1V from the bottom of the excavation above the groundwater level. Within zones of persistent seepage and below the groundwater level, the excavation side slopes are expected to slough and eventually stabilize at a slope of 2H:1V to 3H:1V from the bottom of the excavation.

If side slopes cannot be achieved due to space restrictions on site such as the proximity of open cut excavations to the property limits, existing infrastructure or to foundations of adjacent existing building(s), the new building construction would have to be undertaken within the confines of an engineered support system (shoring system). The installation of the municipal underground services may be undertaken within the confines of a prefabricated support system (trench box) designed and installed in accordance with OHSA.

The need for a shoring system, the most appropriate type of shoring system and the design and installation of the shoring system should be determined by the contractors bidding on this project. The design of the shoring system should be undertaken by a professional engineer experienced in shoring design and the installation of the shoring system should be undertaken by a contractor experienced in the installation of shoring systems. The shoring system should be designed and installed in accordance with latest edition of Ontario Regulation 213/91 under the OHSA and the 2006 Fourth Edition of the Canadian Foundation Engineering Manual (CFEM).

A conventional steel H soldier pile and timber lagging shoring system may be considered for this project. The shoring system must be designed to support the lateral earth pressure given by the expression below:

$$P = k(\gamma h + q)$$

where P = lateral earth pressure, at any depth, h, below the ground surface

k = appropriate earth pressure coefficient:

- Coefficient of static ‘active’ lateral earth pressure coefficient, $K_A = 0.25$, if movement can be tolerated.
- Coefficient of static ‘at rest’ lateral earth pressure coefficient, $K_0 = 0.50$, if no movement can be tolerated.

γ = estimated unit weight of soil to be retained = 22 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 (kPa)

The resultant force from the lateral earth pressure will act one-third from the bottom of the shored wall and the resultant from adjacent surcharge at ground surface will act at one-half from bottom of the wall. 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 underground services and building structures. The traffic loads in the adjacent parking lots, access roads and roadways 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 0.7 m of rock ledge around the perimeter, 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.

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.

The shoring system may require lateral restraint by tiebacks in the form of grouted rock anchors. The shoring system should be tied back by rock anchors grouted into the sound limestone bedrock. The factored ULS grout to rock bond stress of 1200 kPa may be used for design of the anchors. The factored ULS bond stress value includes a geotechnical resistance factor of 0.4. This value assumes a grout with a minimum strength of 30 MPa is used and that the sides of the drilled holes are cleaned prior to the grouting operation. In weathered zones of the bedrock and in areas of horizontal and near vertical seams and fractures, some grout loss should be anticipated when grouting anchors in the bedrock. The grout loss is expected to be higher in the fractured bedrock and lower in the sound bedrock. Difficulties may be encountered during the installation of the rock anchors due to the presence of debris and boulders/cobbles within the subsurface soils.

If the rock anchors extend into adjacent properties, permission will be required from the adjacent property owners for the installation of the tiebacks. If permission is not granted, the shoring system may be braced by cross bracing or the use of rakers on the inside of the shored excavation.

Design anchors should be load tested to two times the design capacity. All anchors should be proof tested to 1.33 times the working load. The anchor should be locked off at working load plus an allowance for relaxation (usually 10 percent). When installing tie backs, casing would be required to advance through the soils (fill and the native silty clay and glacial till). The deflection of the shoring system should be carefully monitored during construction.

A pre-construction condition survey of buildings and infrastructure within the influence zone of the construction should be undertaken prior to start of construction activities.

It is recommended that vibration monitoring be conducted at the site and at adjacent existing buildings and infrastructure during the installation of the shoring system and during construction of the new buildings to ensure the existing structures and infrastructure are not damaged as a result of the construction activities.

11.2.2 Rock Excavation

The excavations will extend deep below the surface of the limestone bedrock. The excavation side slopes in the upper depths of the weathered/highly fractured zones of the limestone bedrock may be cut back at a 1H:1V gradient from the surface of the sound limestone bedrock. The excavation side slopes in the sound limestone bedrock may be undertaken with near vertical sides subject to examination by a geotechnical engineer.

The upper depths of the weathered/highly fractured zones of the limestone bedrock may be excavated using a hoe ram for removal of small quantities of the bedrock; however, this process is expected to be very slow.

The excavation of the sound limestone bedrock to the proposed extensive depths below the bedrock surface may be undertaken by line drilling and blasting method. Should blasting not be permitted, the excavation of the limestone bedrock would have to be undertaken by line drilling. Specialized contractors bidding on this project should decide on their own the most preferred rock removal method; hoe ramming or line drilling and blasting.

Zones of the weathered and fractured rock faces for the portion of the excavation within the bedrock may require support in the form of rock bolts to maintain the integrity of the rock face in conjunction with a wire mesh system and/or shotcrete. Excavations into the bedrock 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 vibration limits for blasting should be in accordance with City of Ottawa Special Provisions (SP No. 1201).

As previously indicated, it is recommended that a pre-construction condition survey of adjacent building(s) and infrastructure (roadways, sidewalks, municipal services) 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.

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.

11.3 De-Watering Requirements

Seepage of the surface and subsurface water into the excavations (including shored excavation) is anticipated and it should be possible to collect water entering the excavations at low points and to remove it by conventional pumping techniques. In areas of high infiltration, a higher seepage rate should be anticipated and may require high-capacity pumps to keep the excavation dry.

For construction dewatering, an Environmental Activity and Sector Registry (EASR) approval may be obtained for water takings greater than 50 m^3 and less than 400 m^3. If more than 400 m^3 per day of groundwater are generated per day for dewatering purposes, then a Permit to Take Water (PTTW) must be obtained from the MECP. Reference is made to the EXP hydrogeological investigation report for this site regarding additional information for EASR and PTTW.

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

The soils to be excavated from the site will comprise of fill, silty clay, glacial till and limestone bedrock. From a geotechnical perspective, these soils and the limestone bedrock are not considered suitable for reuse as backfill material in the interior or exterior of the buildings and should be discarded. It may be possible to use portions of the fill, silty clay 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 times 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 for underfloor fill including backfilling in service trenches inside the building - OPSS Granular B Type II (50 mm minus) placed in 300 mm thick lifts with each lift compacted to 100 percent SPMDD beneath the floor slab,
- Backfill against exterior subsurface walls - OPSS 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 design subgrade level for pavement - OPSS 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.

13. Pavement Structures for Outdoor Access Roads and Parking Lots

The subgrade for the pavement structures of the outdoor access roads and parking lots is anticipated to consist of OPSS Granular B Type II material and/or OPSS Select Subgrade Material (SSM). Pavement structure thicknesses required for the access roads and parking lots set on the anticipated approved subgrade materials were computed and are shown in Table X. The pavement structures assume a functional design life of 15 to 20 years. The proposed functional design life represents the number of years to the first rehabilitation, assuming regular maintenance is carried out.

Table X: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Computed Pavement Structure	
		Light Duty Traffic (Cars Only)	Heavy Duty Traffic (Buses and Trucks)
Asphaltic Concrete	92 percent-97 percent MRD	65 mm HL3/SP12.5 mm/ Cat. B (PG 58-34)	50 mm HL3/SP12.5 Cat. B (PG 58-34) 60 mm HL8/SP 19 Cat. B (PG 58-34)
OPSS 1010 Granular A Base (crushed limestone)	100% percent SPMDD	150 mm	150 mm
OPSS 1010 Granular B Type II Sub-base	100% percent SPMDD	300 mm	450 mm

Notes:

1. SPMDD denotes standard Proctor maximum dry density, ASTM, D-698-12e2.
2. MRD denotes Maximum Relative Density, ASTM D2041.
3. The upper 300 mm of the subgrade fill must be compacted to 98 percent SPMDD.
4. The approved subgrade should be covered with a woven geotextile prior to placement of granular sub-base of the pavement structure.

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 may be required in addition to the woven geotextile indicated in Table X.

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

1. As part of the subgrade preparation, the proposed parking area and access roads should be stripped of any obviously unsuitable material. The subgrade should be properly shaped, crowned, then proofrolled with a heavy vibratory roller in the full-time presence of a representative of this office. Any soft or spongy subgrade areas detected should be sub excavated and properly replaced with suitable approved backfill compacted to 95 percent SPMDD (ASTM D698-12e2).
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. Subdrains should be installed on both sides of the access road(s). Subdrains must be installed in the proposed parking area at low points and should be continuous between catchbasins 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 subdrains 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/manholes to facilitate drainage of any water that may accumulate in the granular fill.
4. The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted lanes, half-loads during paving, temporary construction roadways, etc., may be required, especially if construction is carried out during unfavorable weather.
5. 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.
6. Relatively weaker subgrade may develop over service trenches at subgrade level. These areas may require the use of thicker/coarser sub-base material and the use of a geotextile at the subgrade level. If this is the case, it is recommended that additional 150 mm thick granular sub-base, OPSS Granular B Type II, should be provided in these areas, in addition to the use of a geotextile at the subgrade level.
7. The granular materials used for pavement construction should conform to Ontario Provincial Standard Specifications (OPSS 1010) for Granular A and Granular B Type II and should be compacted to 100 percent of the SPMDD.

The asphaltic concrete used, and its placement should meet OPSS 1150 or 1151 requirements. It should be compacted from 92 percent to 97 percent of the MRD (ASTM D2041). Asphalt placement should be in accordance with OPSS 310 and OPSS 313.

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

14. Corrosion Potential

Chemical tests limited to pH, sulphate, chloride and resistivity were undertaken on one (1) soil sample and three (3) bedrock core sections. A summary of the results is shown in Table XI. The laboratory certificate of analysis is shown in Appendix C.

Table XI: Chemical Test Results

Borehole No. (BH) – Sample (SS)/Run No.	Depth (m)	Soil/Bedrock Type	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
BH-4 - SS3	1.5 – 2.1	Fill	8.02	0.0152	0.0523	833
BH-2 – Run 6	9.9 – 10.0	Limestone Bedrock	9.11	0.0436	0.0034	1180
BH 5 – Run 5	9.4 – 9.5	Limestone Bedrock	9.37	0.0262	0.0060	1430
BH 11 – Run 6	10.0 – 10.1	Limestone Bedrock	9.49	0.0281	0.0031	1480

The results indicate the glacial till has a negligible sulphate attack on subsurface concrete. The concrete should be in accordance with CSA A.23.1-19.

The results of the resistivity tests indicate the soil sample and rock core sections are corrosive to moderately 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.

15. Tree Planting Restrictions

Since the site is not underlain by sensitive marine clay soil, there are no restrictions for tree planting at the site from a geotechnical perspective.

16. General Comments

The comments and recommendations given in this preliminary geotechnical report are preliminary in nature and are based on the assumption that the above-described design concepts 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.

The information contained in this report is not intended to reflect on environmental aspects of the subsurface soils and groundwater. Reference is made to the Phase One and Two Environmental Site Assessments (ESAs) reports and the hydrogeological investigation report prepared by EXP regarding the environmental aspects of the subsurface soils and groundwater at the site.

We trust that the information contained in this preliminary report is satisfactory for your purposes. Should you have any questions, please contact this office.

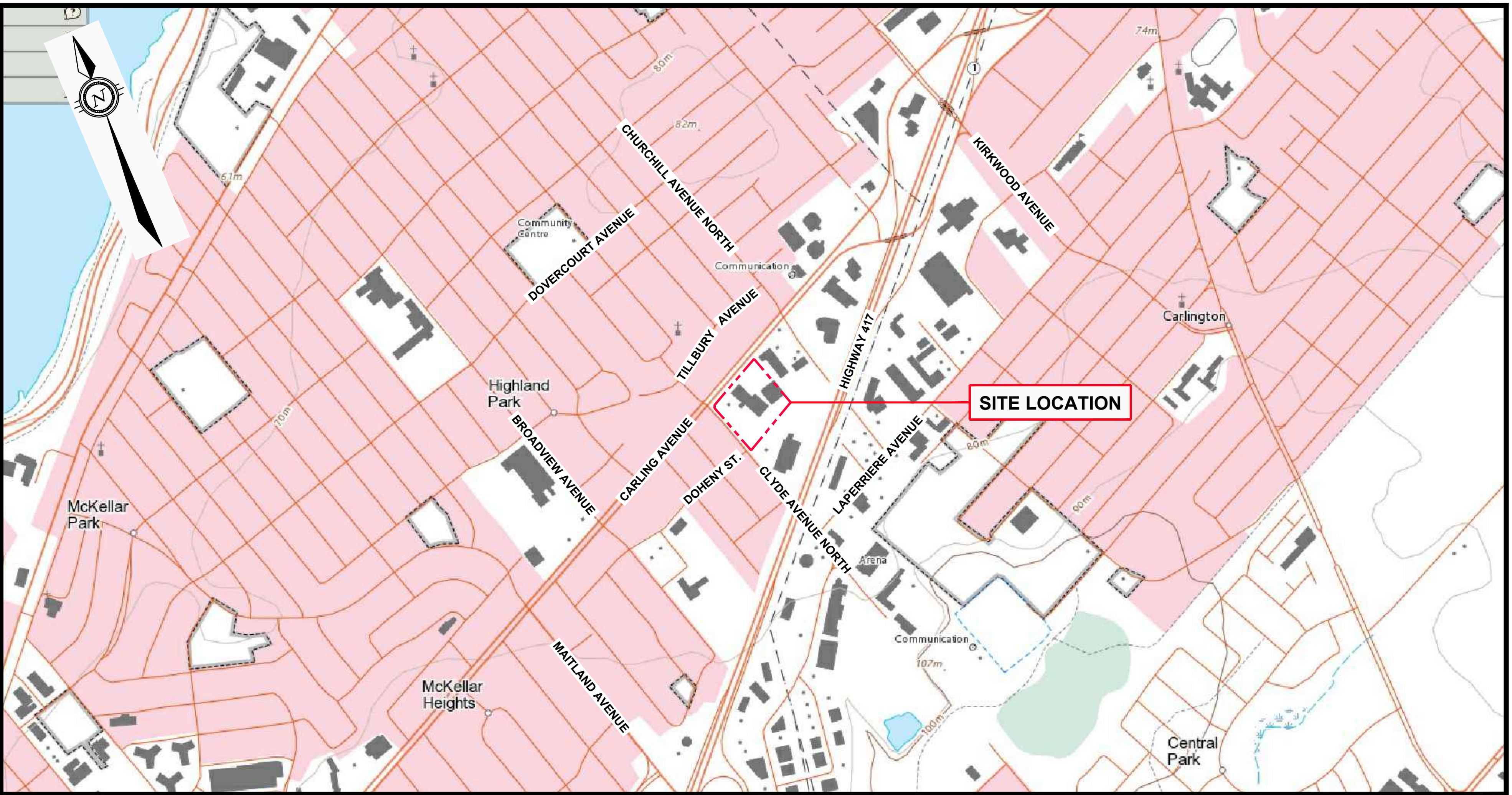
Sincerely,

Susan M. Potyondy, P.Eng.
Senior Geotechnical Engineer
Earth and Environment

LICENSED PROFESSIONAL ENGINEER
2023-08-01
S.M. POTYONDY
37232568
PROVINCE OF ONTARIO

Ismail M. Taki, M.Eng., P.Eng.
Senior Manager, Eastern Region
Earth and Environment

Figures



LEGEND

PROPERTY BOUNDARY

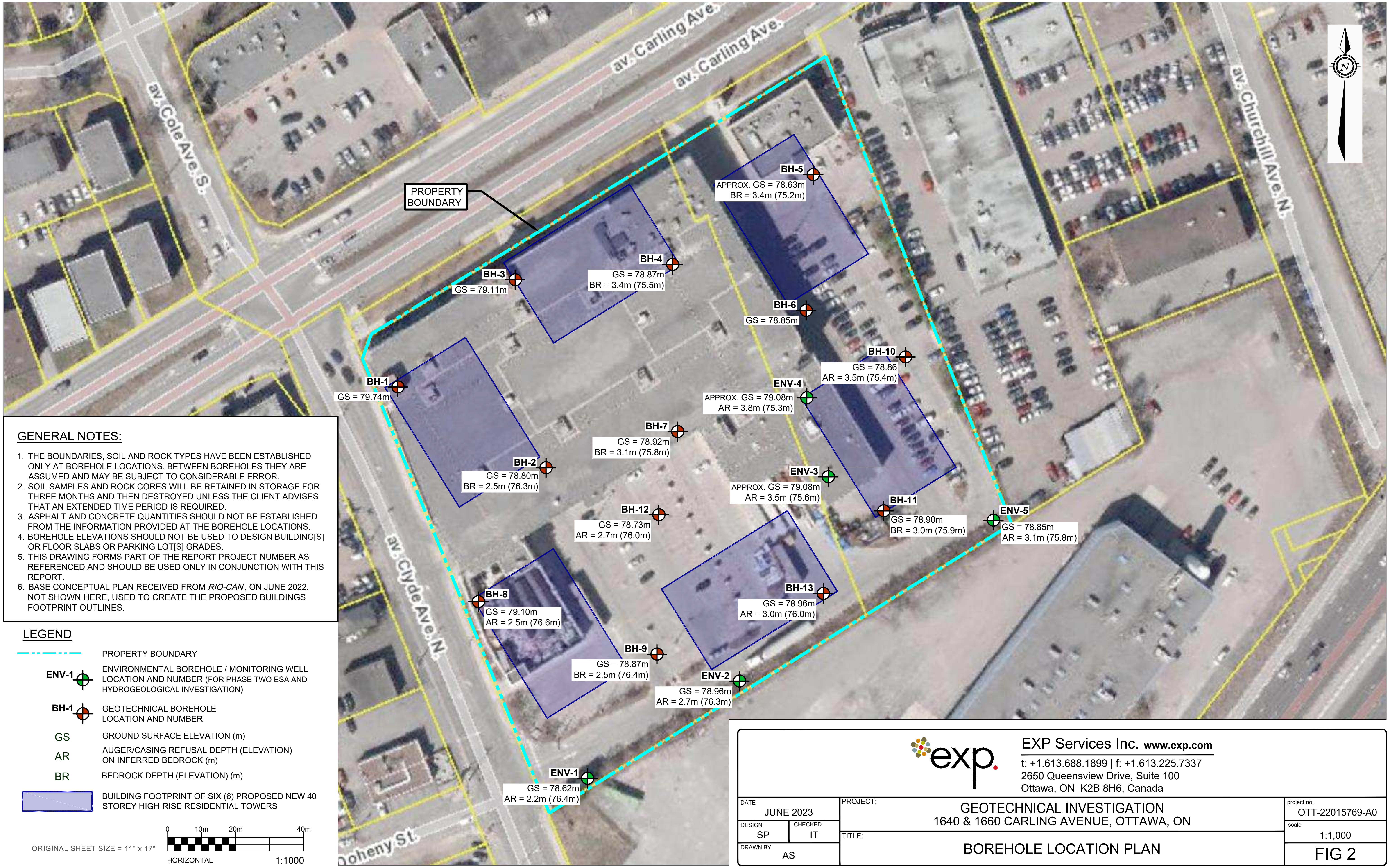
0 150m 300m 600m

HORIZONTAL 1:15,000

EXP Services Inc. www.exp.com
t: +1.613.688.1899 | f: +1.613.225.7337
2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

DATE MAY 2023		PROJECT: GEOTECHNICAL INVESTIGATION 1640 & 1660 CARLING AVENUE, OTTAWA, ON	project no. OTT-22015769-A0
DESIGN SP	CHECKED IT	TITLE: SITE LOCATION PLAN	scale 1:15,000
DRAWN BY AS			FIG 1

Filename: E:\OTT\OTT-22015769-A0\60 Execution\65 Drawings\Geotechnical\22015769-A0_Geo02.dwg
Last Saved: May 29, 2023 3:26 PM Last Plotted: May 29, 2023 3:27 PM Plotted by: SeverA



GENERAL NOTES:

1. THE BOUNDARIES, SOIL AND ROCK 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 AND CONCRETE 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 CONCEPTUAL PLAN RECEIVED FROM *RIO-CAN*, ON JUNE 2022. NOT SHOWN HERE, USED TO CREATE THE PROPOSED BUILDINGS FOOTPRINT OUTLINES.

LEGEND

‐ · ‐ · ‐	PROPERTY BOUNDARY
ENV-1 ⊕	ENVIRONMENTAL BOREHOLE / MONITORING WELL LOCATION AND NUMBER (FOR PHASE TWO ESA AND HYDROGEOLOGICAL INVESTIGATION)
BH-1 ⊕	GEOTECHNICAL BOREHOLE LOCATION AND NUMBER
GS	GROUND SURFACE ELEVATION (m)
AR	AUGER/CASING REFUSAL DEPTH (ELEVATION) ON INFERRED BEDROCK (m)
BR	BEDROCK DEPTH (ELEVATION) (m)
▨	BUILDING FOOTPRINT OF SIX (6) PROPOSED NEW 40 STOREY HIGH-RISE RESIDENTIAL TOWERS

0 10m 20m 40m

HORIZONTAL 1:1000

ORIGINAL SHEET SIZE = 11" x 17"

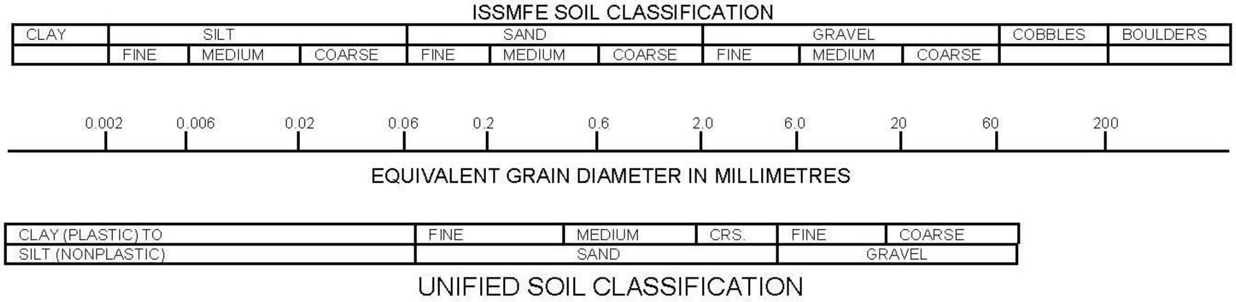
EXP Services Inc. www.exp.com
t: +1.613.688.1899 | f: +1.613.225.7337
2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

DATE	JUNE 2023
DESIGN	SP
CHECKED	IT
DRAWN BY	AS
PROJECT:	GEOTECHNICAL INVESTIGATION 1640 & 1660 CARLING AVENUE, OTTAWA, ON
TITLE:	BOREHOLE LOCATION PLAN
project no.	OTT-22015769-A0
scale	1:1,000
	FIG 2

Filename: E:\OTT\OTT-22015769-A0\60 Execution\65 Drawings\Geotechnical\22015769-A0_Geo02.dwg
Last Saved: Jun 5, 2023 10:55 AM Last Plotted: Jun 5, 2023 10:56 AM Plotted by: SeverA

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 Society 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.



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-02

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 3

Page. 1 of 2

Date Drilled: 'May 8, 2023

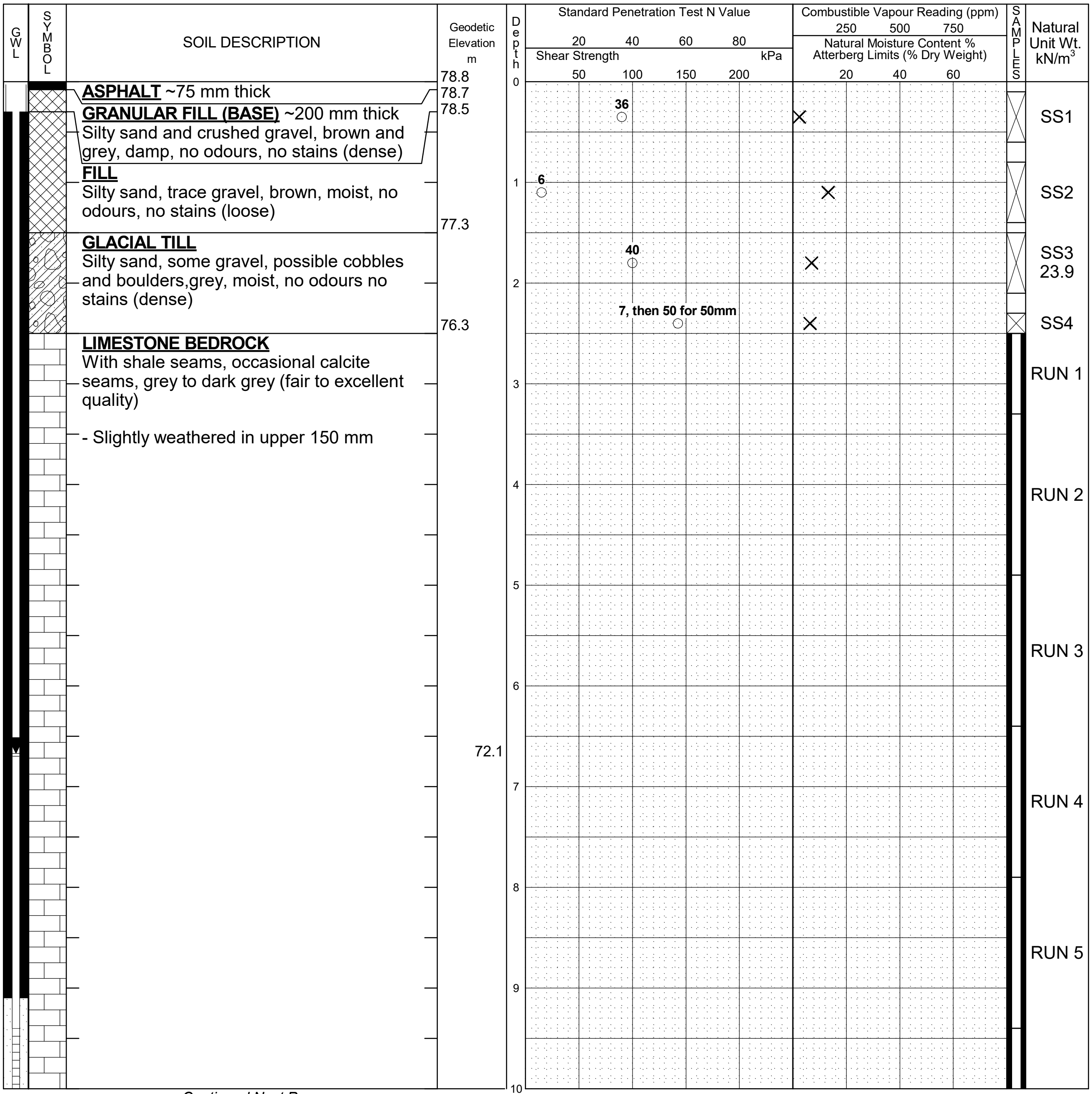
Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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		

Geodetic Elevation m	SOIL DESCRIPTION	Samples	Natural Unit Wt. kN/m³
78.8			
78.7	**ASPHALT** ~75 mm thick		
78.5	**GRANULAR FILL (BASE)** ~200 mm thick Silty sand and crushed gravel, brown and grey, damp, no odours, no stains (dense)	SS1	
	FILL Silty sand, trace gravel, brown, moist, no odours, no stains (loose)	SS2	
77.3	**GLACIAL TILL** Silty sand, some gravel, possible cobbles and boulders,grey, moist, no odours no stains (dense)	SS3 SS4	23.9
76.3	**LIMESTONE BEDROCK** With shale seams, occasional calcite seams, grey to dark grey (fair to excellent quality) - Slightly weathered in upper 150 mm	RUN 1 RUN 2 RUN 3 RUN 4 RUN 5	
72.1			



Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	6.7	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	2.5 - 3.3	100	83
2	3.3 - 4.9	100	98
3	4.9 - 6.4	100	100
4	6.4 - 7.9	100	66
5	7.9 - 9.4	100	100
6	9.4 - 10.9	100	100
7	10.9 - 12.4	100	93

Log of Borehole BH-02

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Figure No. 3

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation 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	SAMPLES	Natural Unit Wt. kN/m^3
		LIMESTONE BEDROCK With shale seams, occasional calcite seams, grey to dark grey (fair to excellent quality) - Slightly weathered in upper 150 mm *(continued)*	68.8	10				RUN 6
				11				RUN 7
			66.4	12				
		Borehole Terminated at 12.4 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	6.7	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	2.5 - 3.3	100	83
2	3.3 - 4.9	100	98
3	4.9 - 6.4	100	100
4	6.4 - 7.9	100	66
5	7.9 - 9.4	100	100
6	9.4 - 10.9	100	100
7	10.9 - 12.4	100	93

Log of Borehole BH-04

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 4

Page. 1 of 2

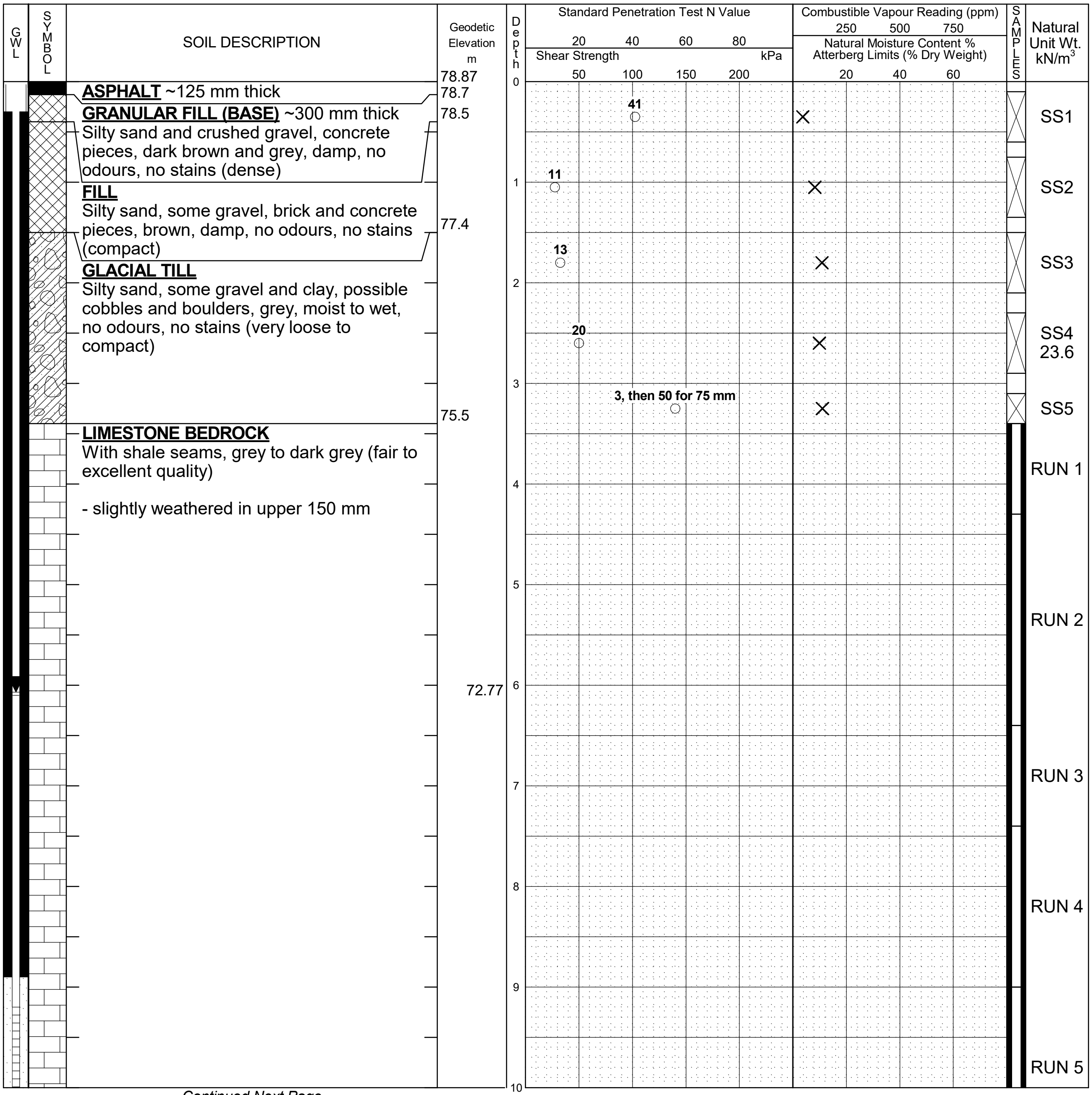
Date Drilled: 'May 5, 2023

Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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		



ASPHALT ~125 mm thick

GRANULAR FILL (BASE) ~300 mm thick
Silty sand and crushed gravel, concrete pieces, dark brown and grey, damp, no odours, no stains (dense)

FILL
Silty sand, some gravel, brick and concrete pieces, brown, damp, no odours, no stains (compact)

GLACIAL TILL
Silty sand, some gravel and clay, possible cobbles and boulders, grey, moist to wet, no odours, no stains (very loose to compact)

LIMESTONE BEDROCK
With shale seams, grey to dark grey (fair to excellent quality)

- slightly weathered in upper 150 mm

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	6.1	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.4 - 4.3	97	82
2	4.3 - 6.4	96	71
3	6.4 - 7.4	100	95
4	7.4 - 9	100	100
5	9 - 10.6	100	100
6	10.6 - 12.2	100	90

Log of Borehole BH-04

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Figure No. 4

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation 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	SAMPLES	Natural Unit Wt. kN/m^3
			68.87	10				
		LIMESTONE BEDROCK With shale seams, grey to dark grey (fair to excellent quality) - slightly weathered in upper 150 mm *(continued)*		11				RUN 6
			66.7	12				
		Borehole Terminated at 12.2 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	6.1	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.4 - 4.3	97	82
2	4.3 - 6.4	96	71
3	6.4 - 7.4	100	95
4	7.4 - 9	100	100
5	9 - 10.6	100	100
6	10.6 - 12.2	100	90

Log of Borehole BH-05

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 5

Page. 1 of 2

Date Drilled: 'May 9, 2023

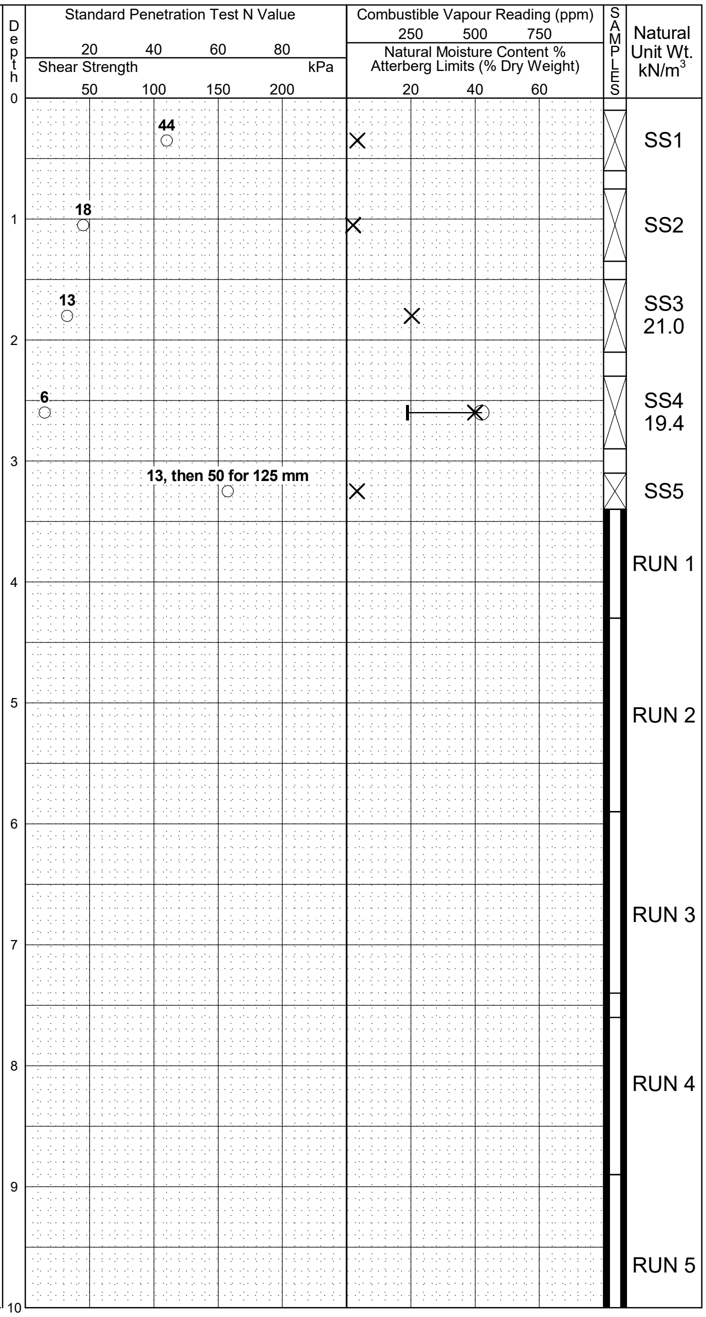
Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE
Checked by: SP

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
	78.63
ASPHALT ~75 mm thick	78.6
GRANULAR FILL (BASE) ~1125 mm thick Silty sand and crushed gravel, concrete pieces, dark brown and grey, damp, no odours, no stains (dense)	77.4
FILL Silty sand, some gravel, with 25 mm thick topsoil seam, rootlets, brown to grey, moist, no odours, no stains (compact)	76.4
SILTY CLAY Medium plasticity, some sand, grey, moist, no odours, no stains (firm)	75.6
GLACIAL TILL Silty sand, some gravel, possible cobbles and boulders, grey, moist, no odours, no stains (compact)	75.2
LIMESTONE BEDROCK With shale seams, occasional calcite seams, grey to dark grey (fair to excellent quality)	



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NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. The borehole was backfilled upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.4 - 4.3	86	71
2	4.3 - 5.9	100	90
3	5.9 - 7.4	100	90
4	7.4 - 8.9	98	92
5	8.9 - 10.4	100	98
6	10.4 - 11.9	100	97

Log of Borehole BH-05

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Figure No. 5

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation 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	SAMPLES	Natural Unit Wt. kN/m³
		LIMESTONE BEDROCK With shale seams, occasional calcite seams, grey to dark grey (fair to excellent quality) *(continued)*	68.63 66.7	10 11				RUN 6
		Borehole Terminated at 11.9 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. The borehole was backfilled upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.4 - 4.3	86	71
2	4.3 - 5.9	100	90
3	5.9 - 7.4	100	90
4	7.4 - 8.9	98	92
5	8.9 - 10.4	100	98
6	10.4 - 11.9	100	97

Log of Borehole BH-07

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 6

Page. 1 of 2

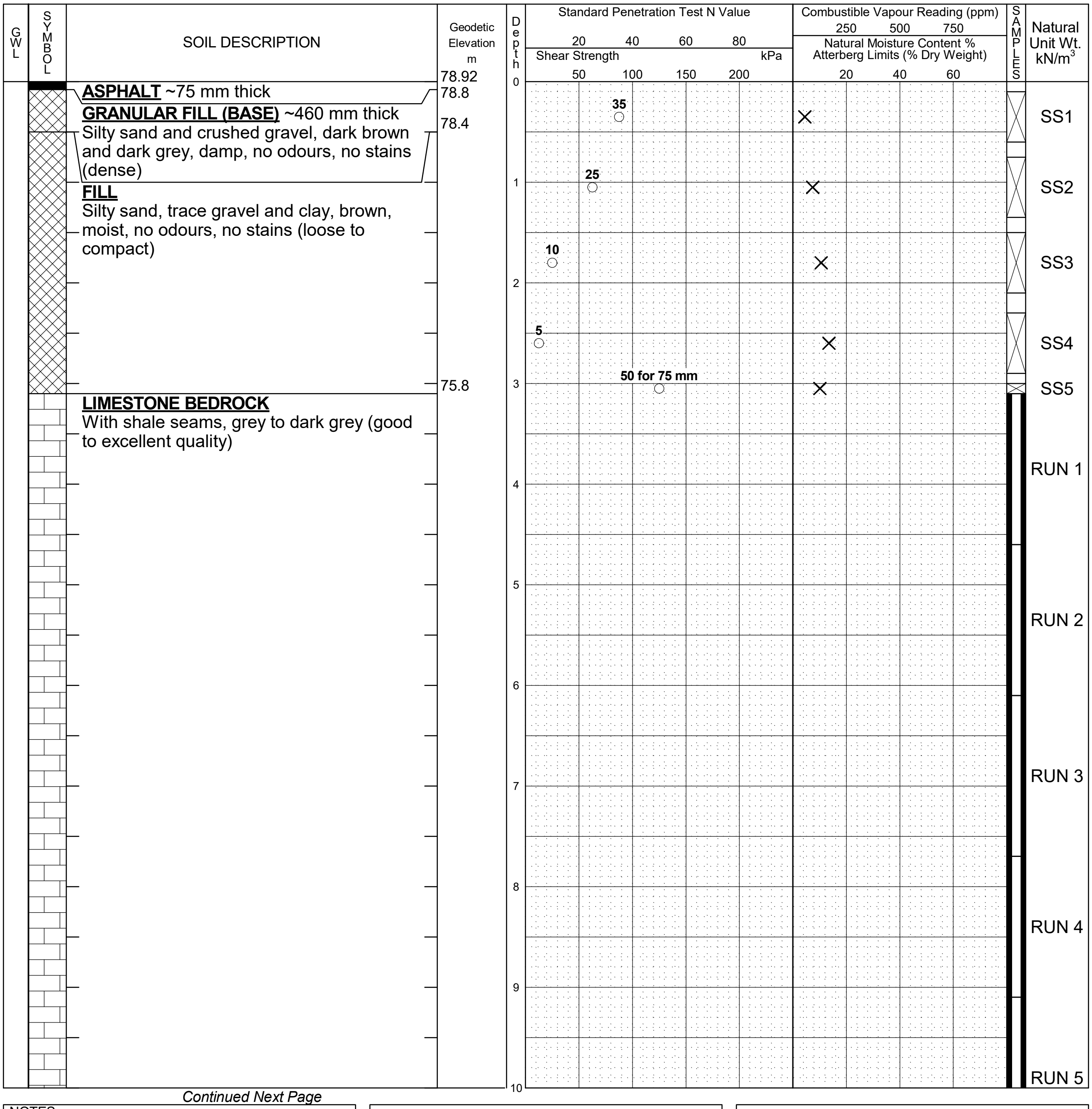
Date Drilled: 'May 8, 2023

Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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		



ASPHALT ~75 mm thick

GRANULAR FILL (BASE) ~460 mm thick
Silty sand and crushed gravel, dark brown and dark grey, damp, no odours, no stains (dense)

FILL
Silty sand, trace gravel and clay, brown, moist, no odours, no stains (loose to compact)

LIMESTONE BEDROCK
With shale seams, grey to dark grey (good to excellent quality)

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. The borehole was backfilled upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.1 - 4.6	92	95
2	4.6 - 6.1	100	90
3	6.1 - 7.7	100	80
4	7.7 - 9.1	100	98
5	9.1 - 10.7	100	95
6	10.7 - 12.2	100	94

Log of Borehole BH-07

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Figure No. 6

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation 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	SAMPLES	Natural Unit Wt. kN/m³
		LIMESTONE BEDROCK With shale seams, grey to dark grey (good to excellent quality) *(continued)*	68.92	10 11 12				RUN 6
			66.7					
		Borehole Terminated at 12.2 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. The borehole was backfilled upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.1 - 4.6	92	95
2	4.6 - 6.1	100	90
3	6.1 - 7.7	100	80
4	7.7 - 9.1	100	98
5	9.1 - 10.7	100	95
6	10.7 - 12.2	100	94

Log of Borehole BH-08

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 7

Page. 1 of 1

Date Drilled: 'May 5, 2023

Drill Type: CME-55 Track Mounted Drill Rig

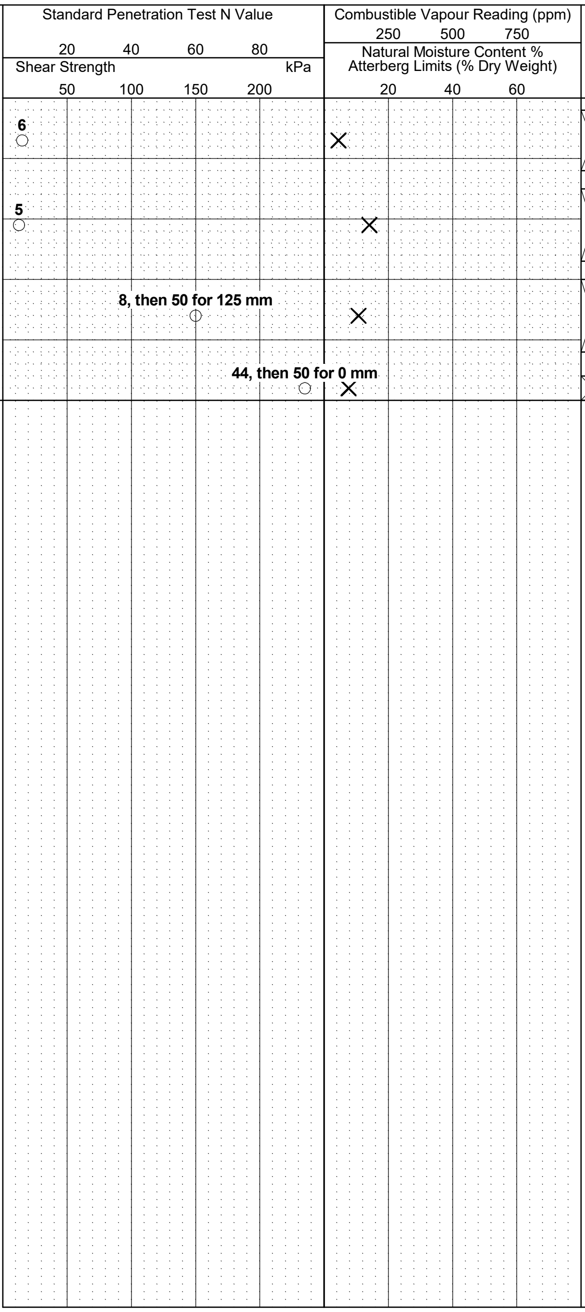
Datum: Geodetic Elevation

Logged by: JE

Checked by: SP

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	SOIL DESCRIPTION	Geodetic Elevation m	Depth	SAMPLES	Natural Unit Wt. kN/m³
		79.1	0		
	ASPHALT ~75 mm thick	79.0			
	GRANULAR FILL (BASE) ~460 mm thick Silty sand and crushed gravel, concrete pieces, dark brown and dark grey, damp, no odours, no stains (loose)	78.6		SS1	
	FILL Silty sand, some gravel, brown, damp to moist, no odours, no stains (loose)	77.6	1	SS2	
	GLACIAL TILL Silty sand, trace gravel and clay, possible cobbles and boulders, grey, moist, no odours, no stains (loose to dense)	76.6	2	SS3 SS4	
	Auger Refusal at 2.5 m Depth				



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. The borehole was backfilled upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

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

Log of Borehole BH-09

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 8

Page. 1 of 2

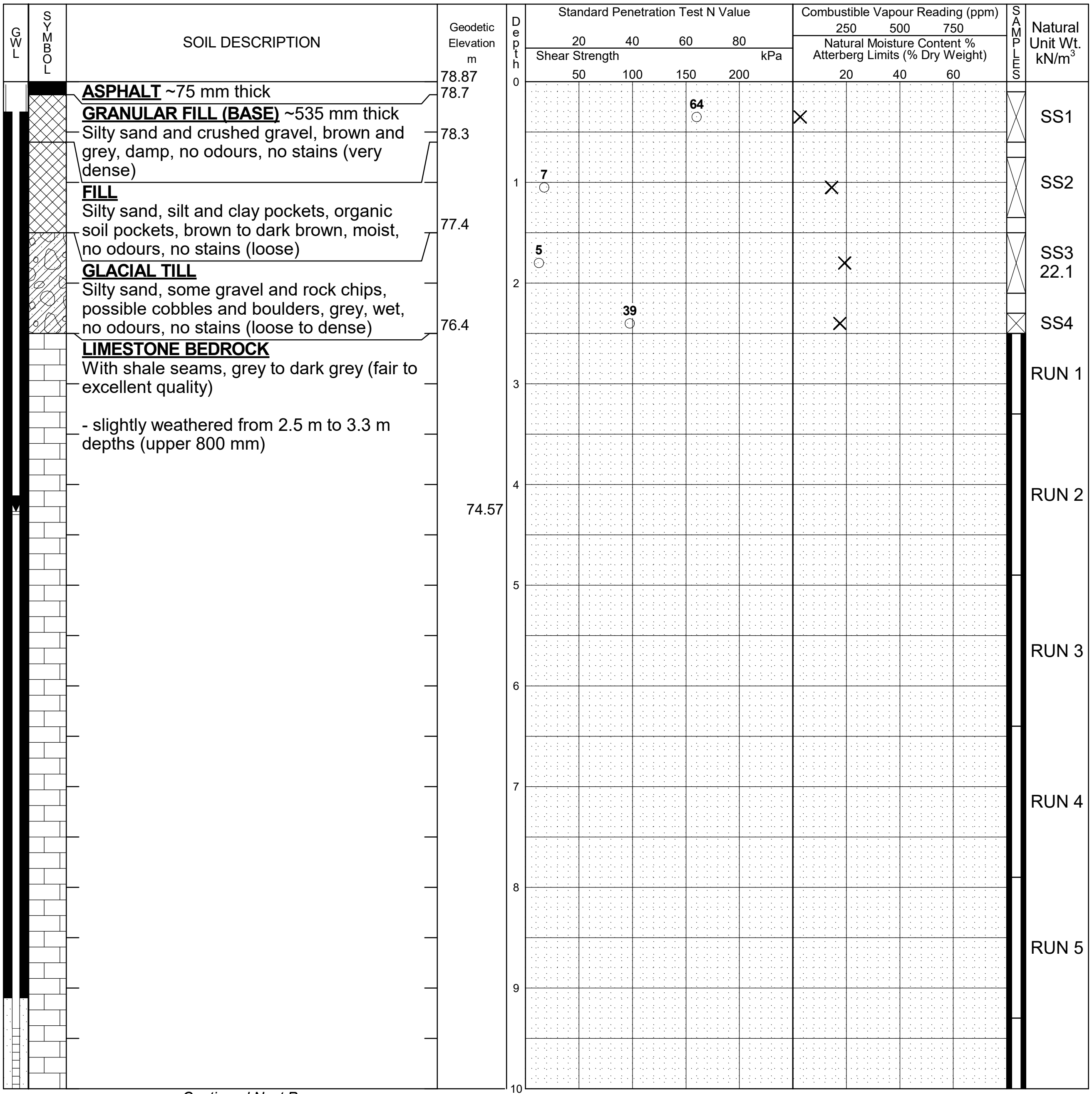
Date Drilled: 'May 10, 2023

Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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	▲



ASPHALT ~75 mm thick

GRANULAR FILL (BASE) ~535 mm thick
Silty sand and crushed gravel, brown and grey, damp, no odours, no stains (very dense)

FILL
Silty sand, silt and clay pockets, organic soil pockets, brown to dark brown, moist, no odours, no stains (loose)

GLACIAL TILL
Silty sand, some gravel and rock chips, possible cobbles and boulders, grey, wet, no odours, no stains (loose to dense)

LIMESTONE BEDROCK
With shale seams, grey to dark grey (fair to excellent quality)

- slightly weathered from 2.5 m to 3.3 m depths (upper 800 mm)

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	4.3	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	2.5 - 3.3	100	61
2	3.3 - 4.9	98	89
3	4.9 - 6.4	100	90
4	6.4 - 7.9	100	90
5	7.9 - 9.3	100	98
6	9.3 - 11	100	100
7	11 - 12.4	100	95

Log of Borehole BH-09

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Figure No. 8

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation 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)	SAMPLES	Natural Unit Wt. kN/m^3
		LIMESTONE BEDROCK With shale seams, grey to dark grey (fair to excellent quality) - slightly weathered from 2.5 m to 3.3 m depths (upper 800 mm) *(continued)*	68.87	10				RUN 6
				11				
				12				RUN 7
			66.5					
		Borehole Terminated at 12.4 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	4.3	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	2.5 - 3.3	100	61
2	3.3 - 4.9	98	89
3	4.9 - 6.4	100	90
4	6.4 - 7.9	100	90
5	7.9 - 9.3	100	98
6	9.3 - 11	100	100
7	11 - 12.4	100	95

Log of Borehole BH-10

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 9

Page. 1 of 1

Date Drilled: 'May 4, 2023

Drill Type: CME-55 Track Mounted Drill Rig

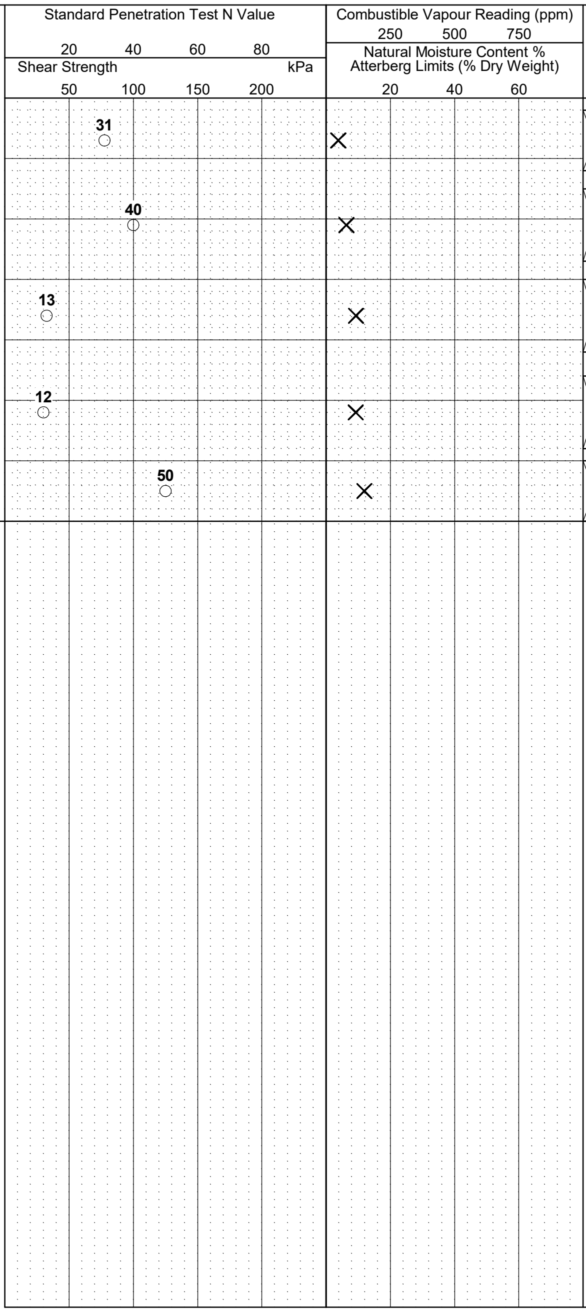
Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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	SAMPLES	Natural Unit Wt. kN/m³
			78.86	0		
		ASPHALT ~ 65 mm thick	78.8			
		GRANULAR FILL (BASE) ~595 mm thick Silty sand and crushed gravel, brown and grey, damp, no odours, no stains (dense)	78.2		☒	SS1
		FILL Silty sand, some gravel, concrete pieces, brown, damp to moist, no odours, no stains (compact to dense)	76.7	1 2	☒ ☒	SS2 SS3
		GLACIAL TILL Silty sand, some gravel, possible cobbles and boulders, grey, wet, no odours, no stains (compact to very dense)	75.4	3	☒ ☒	SS4 23.5 SS5

Auger Refusal at 3.5 m Depth



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. The borehole was backfilled upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

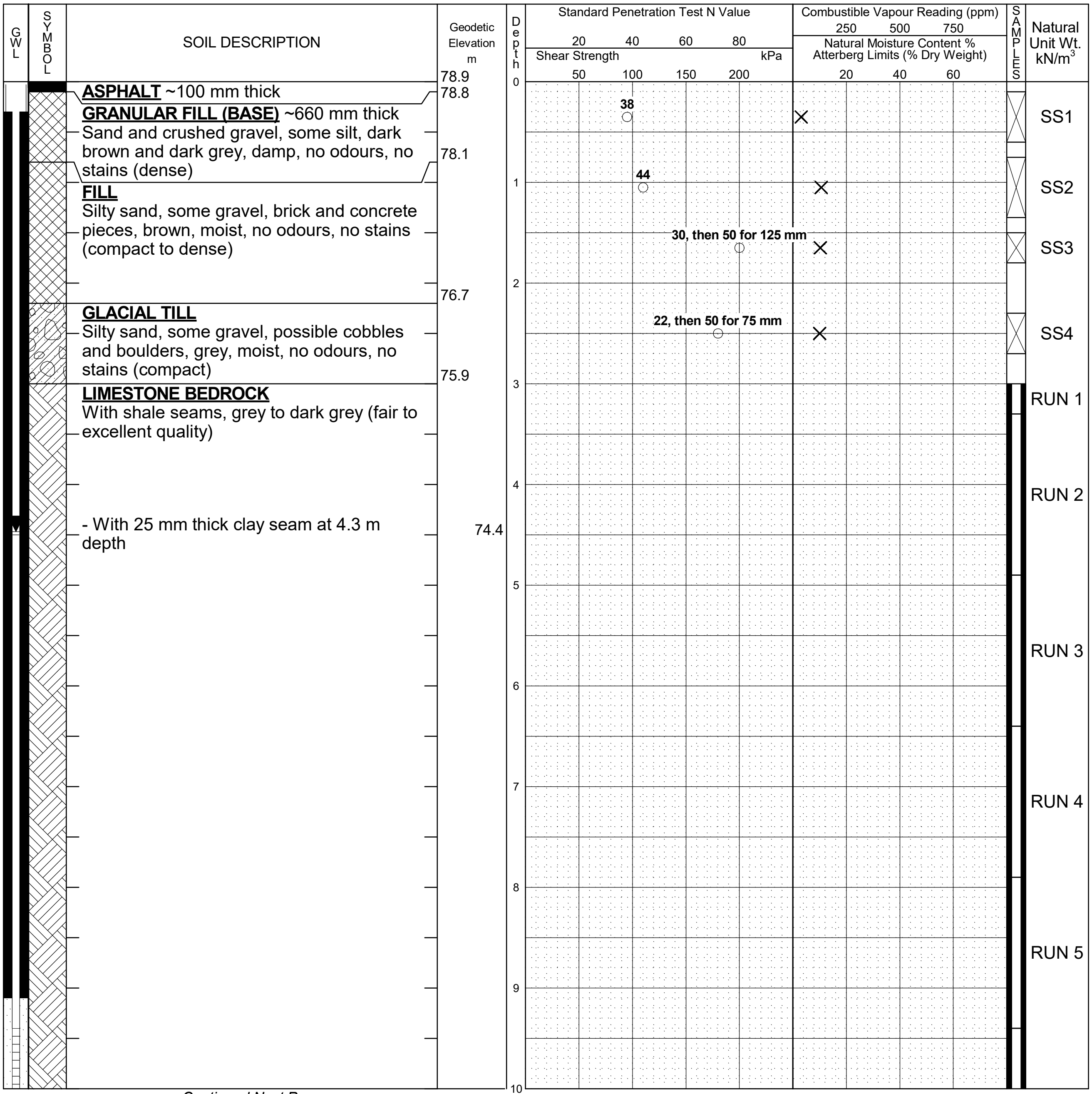
Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-11

Project No: OTT-22015769-A0
Project: Proposed Residential High-Rise Towers
Location: 1640 - 1660 Carling Avenue, Ottawa, ON
Date Drilled: 'May 9, 2023
Drill Type: CME-55 Track Mounted Drill Rig
Datum: Geodetic Elevation
Logged by: JE Checked by: SP

Figure No. 10
Page. 1 of 2

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
	78.9
ASPHALT ~100 mm thick	78.8
GRANULAR FILL (BASE) ~660 mm thick Sand and crushed gravel, some silt, dark brown and dark grey, damp, no odours, no stains (dense)	78.1
FILL Silty sand, some gravel, brick and concrete pieces, brown, moist, no odours, no stains (compact to dense)	76.7
GLACIAL TILL Silty sand, some gravel, possible cobbles and boulders, grey, moist, no odours, no stains (compact)	75.9
LIMESTONE BEDROCK With shale seams, grey to dark grey (fair to excellent quality)	
- With 25 mm thick clay seam at 4.3 m depth	74.4

Continued Next Page

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	4.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3 - 3.3	100	50
2	3.3 - 4.9	100	77
3	4.9 - 6.4	100	86
4	6.4 - 7.9	100	95
5	7.9 - 9.4	100	93
6	9.4 - 11	100	100
7	11 - 12.4	100	96

Log of Borehole BH-11

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Figure No. 10

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation 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)	SAMPLES	Natural Unit Wt. kN/m^3
		LIMESTONE BEDROCK With shale seams, grey to dark grey (fair to excellent quality) *(continued)*	68.9	10				RUN 6
				11				
								RUN 7
				12				
			66.5					
		Borehole Terminated at 12.4 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	4.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3 - 3.3	100	50
2	3.3 - 4.9	100	77
3	4.9 - 6.4	100	86
4	6.4 - 7.9	100	95
5	7.9 - 9.4	100	93
6	9.4 - 11	100	100
7	11 - 12.4	100	96

Log of Borehole BH-12

Project No: OTT-22015769-A0

Figure No. 11

Page. 1 of 1

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Date Drilled: 'May 4, 2023

Drill Type: CME-55 Track Mounted Drill Rig

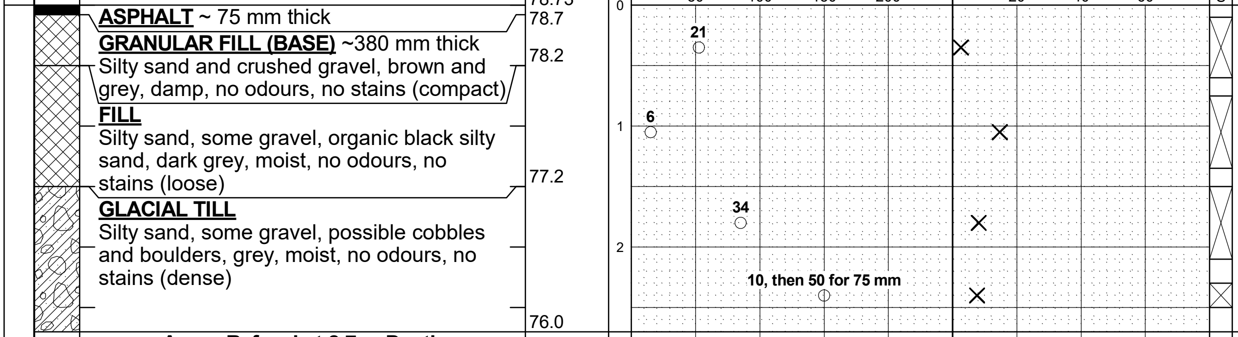
Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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	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)	SAMPLES	Natural Unit Wt. kN/m^3
			78.73	0				
		ASPHALT ~ 75 mm thick	78.7					
		GRANULAR FILL (BASE) ~380 mm thick Silty sand and crushed gravel, brown and grey, damp, no odours, no stains (compact)	78.2		21	×	☒	SS1
		FILL Silty sand, some gravel, organic black silty sand, dark grey, moist, no odours, no stains (loose)	77.2	1	6	×	☒	SS2
		GLACIAL TILL Silty sand, some gravel, possible cobbles and boulders, grey, moist, no odours, no stains (dense)		2	34	×	☒	SS3
			76.0		10, then 50 for 75 mm	×	☒	SS4

Auger Refusal at 2.7 m Depth



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

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

Log of Borehole BH-13

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 12

Page. 1 of 1

Date Drilled: 'May 4, 2023

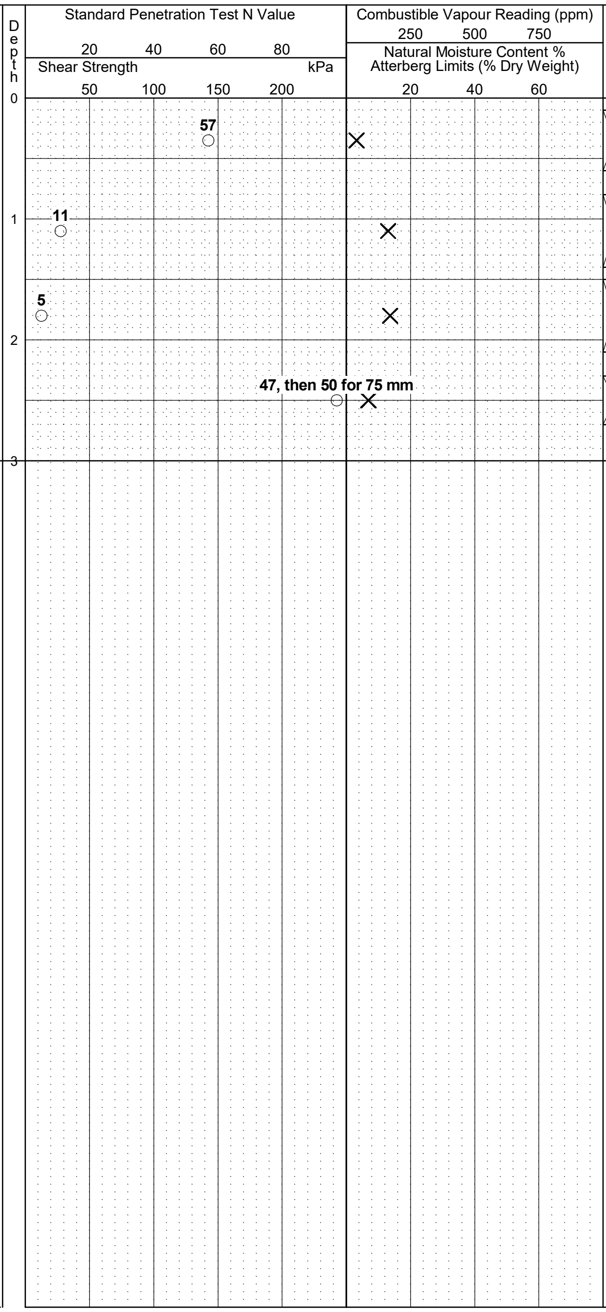
Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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
			78.96	0
		ASPHALT ~ 125 mm thick	78.8	
		GRANULAR FILL (BASE) ~535 mm thick Silty sand and crushed gravel, brown and grey, damp, no odours, no stains (very dense)	78.3	
		FILL Silty sand, some gravel, brown, moist, no odours, no stains (loose to compact)	76.8	1 2
		GLACIAL TILL Silty sand, some gravel, possible cobbles and boulders, grey, moist, no odours, no stains (dense)	76.0	3
		Auger Refusal at 3.0 m Depth		



SAMPLES	Natural Unit Wt. kN/m³
☒	SS1
☒	SS2
☒	SS3
☒	SS4 21.9

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. The borehole was backfilled upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

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

Log of Borehole ENV-1

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 13

Page. 1 of 1

Date Drilled: 'May 4, 2023

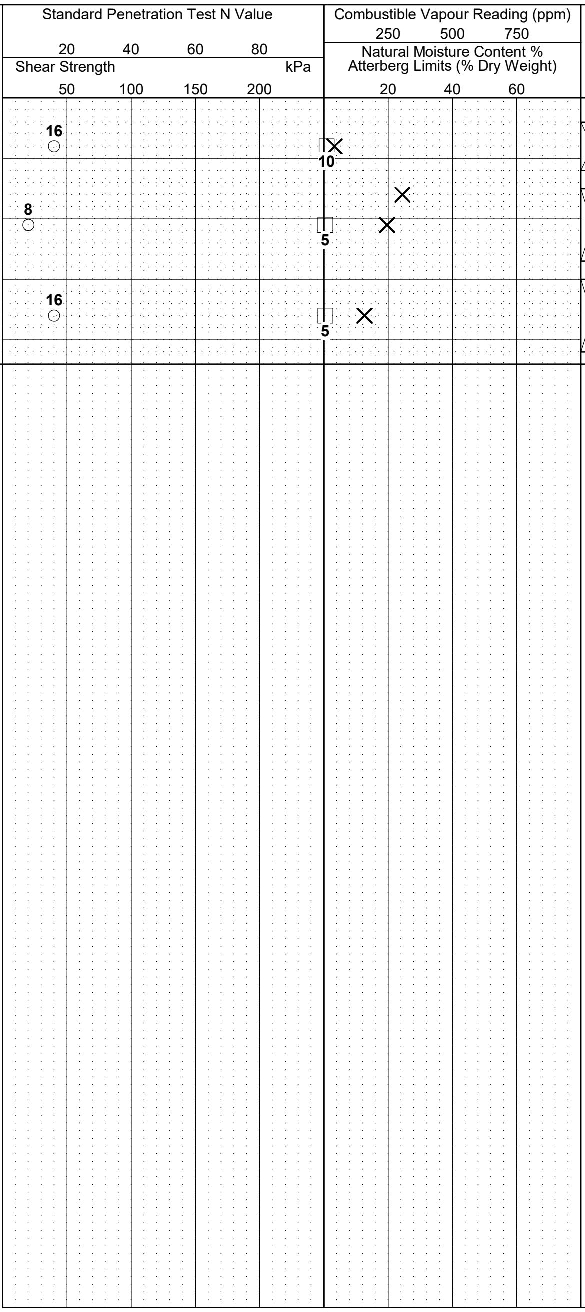
Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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	SAMPLES	Natural Unit Wt. kN/m³
			78.62	0		
		ASPHALT ~180 mm thick	78.4			
		GRANULAR FILL (BASE) ~505 mm thick Silty sand and crushed gravel, brown and grey, damp, no odours, no stains (compact)	77.9		SS1	
		FILL Silty sand, some gravel, brown, wet, no odours, no stains (loose)	77.2	1	SS2	
▼		**GLACIAL TILL** Silty sand, some gravel and clay, possible cobbles and boulders, grey, wet, no odours, no stains (compact)	76.4 76.42	2	SS3	
		Auger Refusal at 2.2 m Depth				



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	2.2	

CORE DRILLING RECORD

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

Log of Borehole ENV-2

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 14

Page. 1 of 1

Date Drilled: 'May 4, 2023

Drill Type: CME-55 Track Mounted Drill Rig

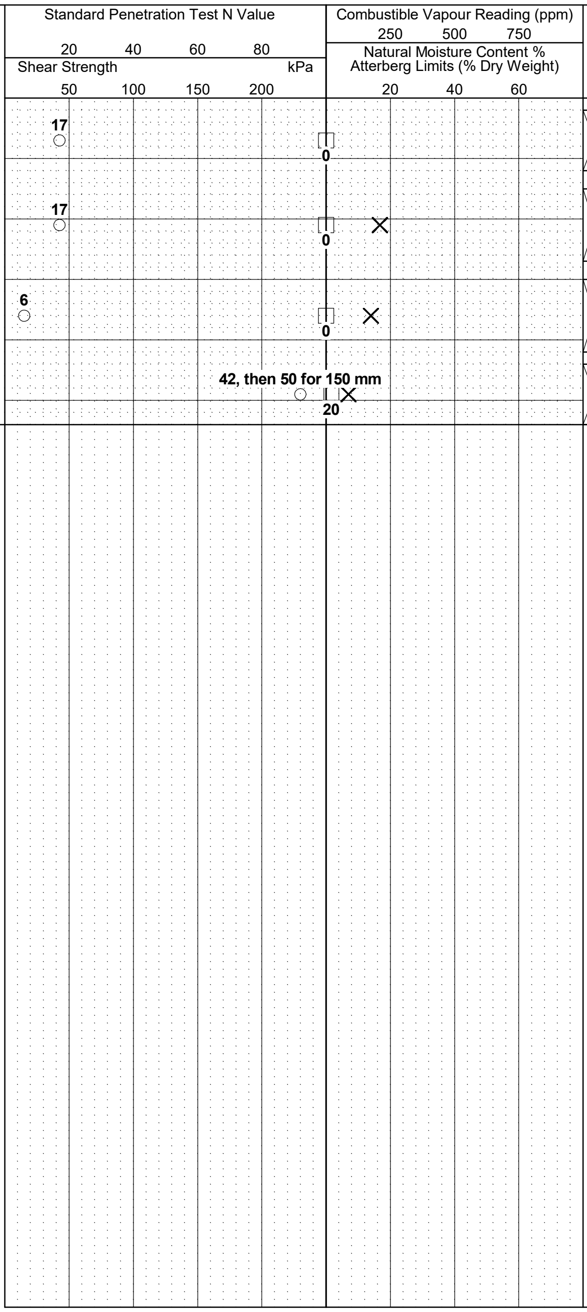
Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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	SAMPLES	Natural Unit Wt. kN/m³
	78.96	0		
ASPHALT ~125 mm thick	78.8			
GRANULAR FILL (BASE) ~575 mm thick Silty sand and crushed gravel, brown and grey, damp, no odours, no stains (compact)	78.3		SS1	
FILL Silty sand, some gravel and clay, brown, moist to wet, no odours, no stains (loose to compact)	76.8	1 2	SS2 SS3	
GLACIAL TILL Silty sand, some gravel, possible cobbles and boulders, grey, wet, no odours, no stains (dense)	76.3 76.26		SS4	

Auger Refusal at 2.7 m Depth



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	2.7	

CORE DRILLING RECORD

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

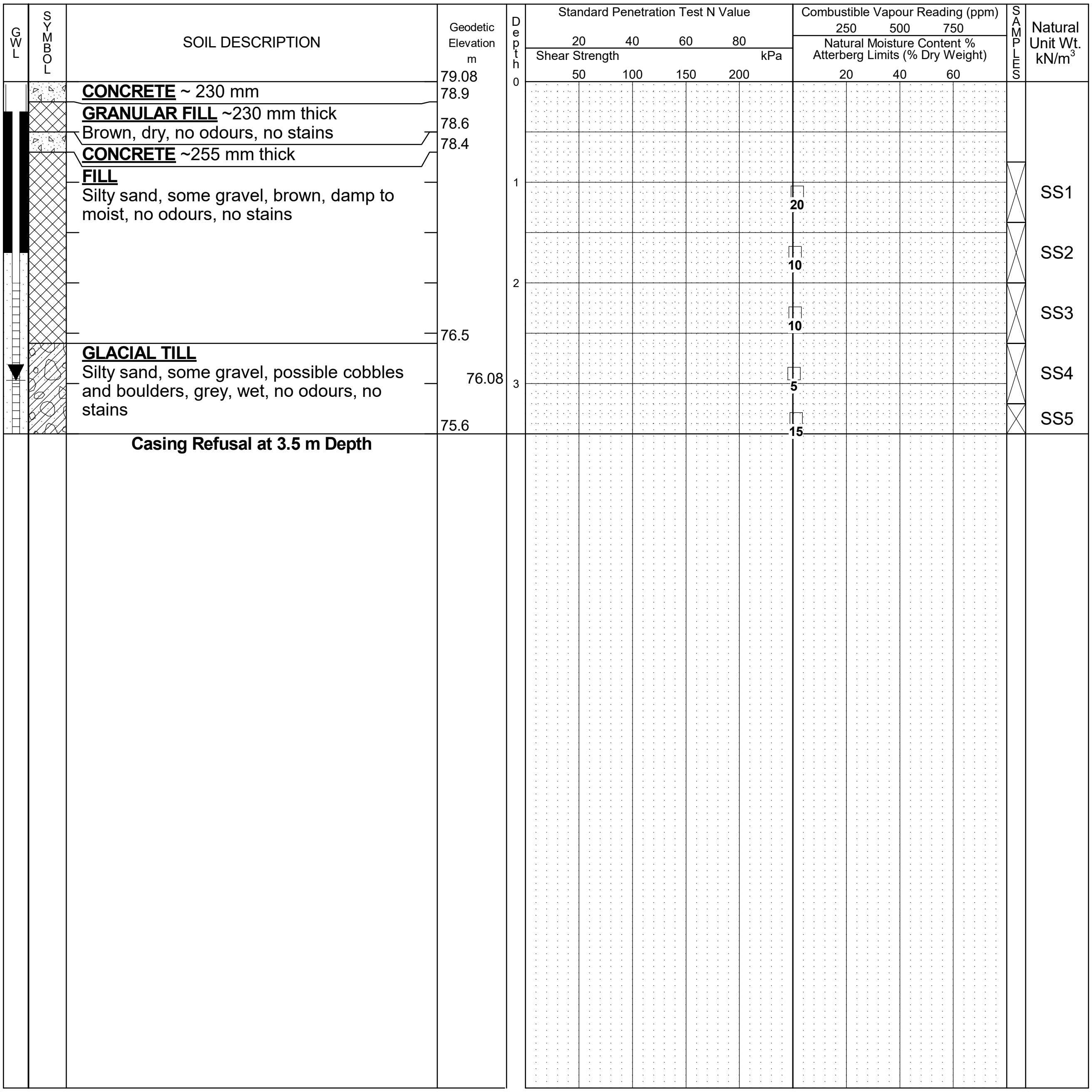
Log of Borehole ENV-3

Project No: OTT-22015769-A0
Project: Proposed Residential High-Rise Towers
Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 15
Page. 1 of 1

Date Drilled: 'May 12, 2023
Drill Type: Portable Drilling Equipment
Datum: Geodetic Elevation
Logged by: JE Checked by: SP

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	Combustible Vapour Reading (ppm)	SAMPLES	Natural Unit Wt. kN/m^3
	79.08	0			
CONCRETE ~ 230 mm	78.9				
GRANULAR FILL ~230 mm thick Brown, dry, no odours, no stains	78.6				
CONCRETE ~255 mm thick	78.4				
FILL Silty sand, some gravel, brown, damp to moist, no odours, no stains		1	20	SS1	
			10	SS2	
		2	10	SS3	
GLACIAL TILL Silty sand, some gravel, possible cobbles and boulders, grey, wet, no odours, no stains	76.5				
	76.08	3	5	SS4	
	75.6		15	SS5	
Casing Refusal at 3.5 m Depth					

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	3.0	

CORE DRILLING RECORD

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

Log of Borehole ENV-4

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 16

Page. 1 of 1

Date Drilled: 'May 12, 2023

Drill Type: Portable Drilling Equipment

Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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	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)	SAMPLES	Natural Unit Wt. kN/m³
	79.08	0				
CONCRETE ~255 mm thick	78.8					
FILL Silty sand and gravel, brown, damp to wet, no odours, no stains				10	☒	SS1
		1		15	☒	SS2
				10	☒	SS3
		2		10	☒	SS4
	75.98	3		5	☒	SS5
	75.3			25	☒	SS6
Casing Refusal at 3.8 m Depth						

LOG OF BOREHOLE BH LOGS 1660 CARLING AVE.GPJ TROW OTTAWA.GDT 6/20/23

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 50mm PVC monitoring well was installed upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
May 31,2023	3.1	

CORE DRILLING RECORD

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

Log of Borehole ENV-5

Project No: OTT-22015769-A0

Project: Proposed Residential High-Rise Towers

Location: 1640 - 1660 Carling Avenue, Ottawa, ON

Figure No. 17

Page. 1 of 1

Date Drilled: 'May 4, 2023

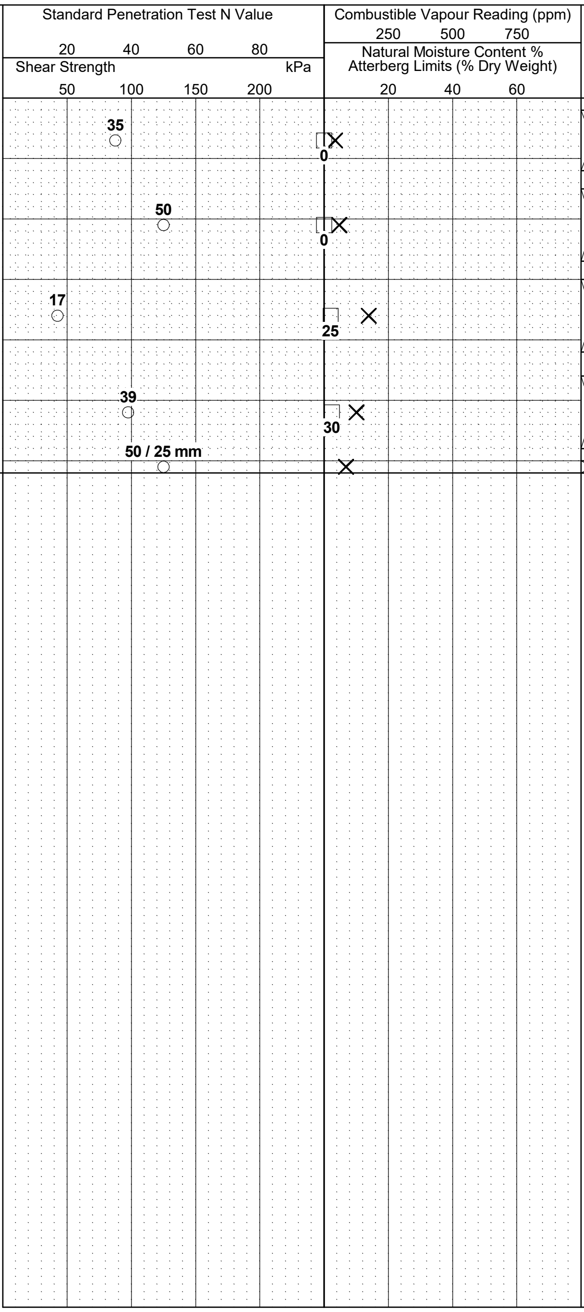
Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE Checked by: SP

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
	78.85	0
ASPHALT ~125 mm thick	78.7	
GRANULAR FILL (BASE) ~1300 mm thick Silty sand and gravel, brown and grey, damp, no odours, no stains (dense to very dense).	77.5	1
FILL Silty sand, some gravel, brown, moist, no odours, no stains (compact)	76.7	2
GLACIAL TILL Silty sand, some gravel, trace clay, possible cobbles and boulders, rock chips, grey, damp, no odours, no stains (dense)	75.8	3
Auger Refusal at 3.1 m Depth		



SAMPLES	Standard Penetration Test N Value	Combustible Vapour Reading (ppm)	Natural Unit Wt. kN/m^3
SS1	35	0	
SS2	50	0	
SS3	17	25	
SS4	39	30	23.7
SS5	50 / 25 mm		

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. The borehole was backfilled upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22015769-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

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

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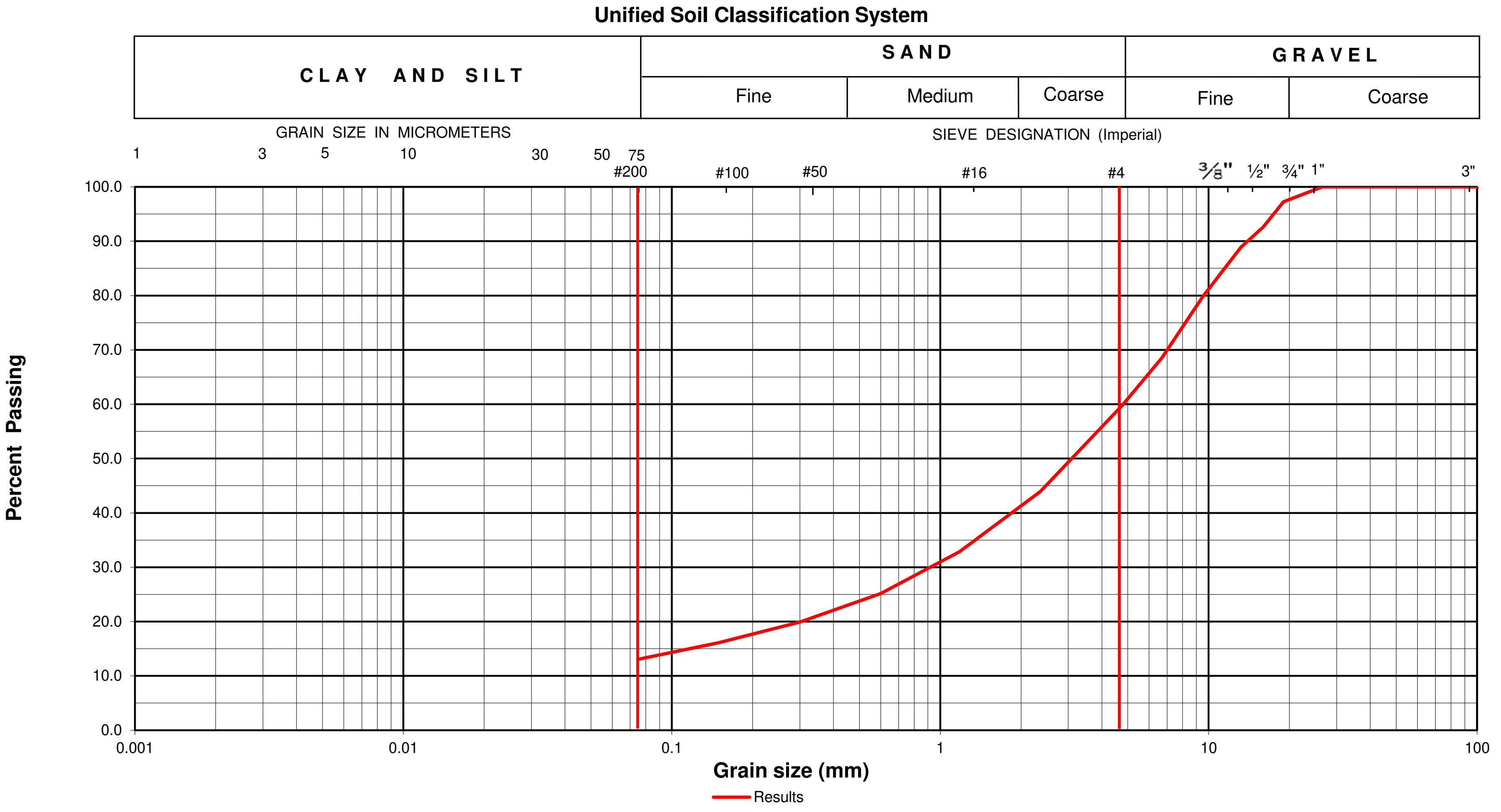
Grain-Size Distribution Curve

Method of Test for Sieve Analysis of Aggregate

ASTM C-136 (LS-602)



Exp Project No.:	**OTT-22015769-A0**	**Project Name :**	**Proposed Residential High-Rise Towers**					
Client :	**RioCan Real Estate Investment Trust**	**Project Location :**	**1640 - 1660 Carling Avenue, Ottawa, ON**					
Date Sampled :	**May 9, 2023**	**Borehole:**	**BH11**	**Sample:**	**SS1**		**Depth (m) :**	**0.1 - 0.6**
Sample Description :		**% Silt & Clay**	**13**	**% Sand**	**47**	**% Gravel**	**40**	**Figure : 18**
Sample Description :	**GRANULAR FILL: Sand and Gravel (SM) - Some Silt**							

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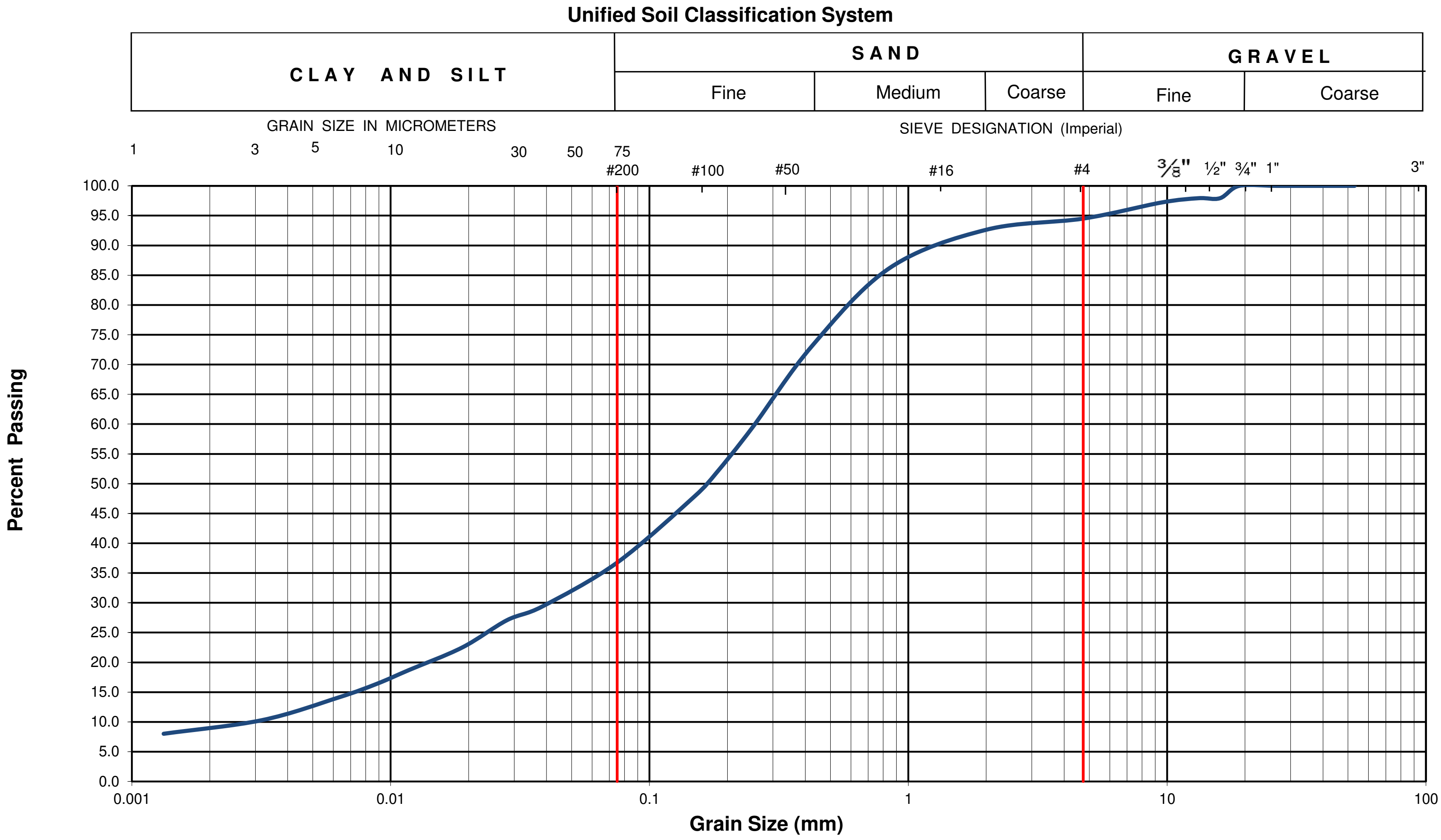
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-22015769-A0**	**Project Name :**	**Proposed Residential High-Rise Towers**						
Client :	**RioCan Real Estate Investment Trust**	**Project Location :**	**1640-1660 Carling Avenue, Ottawa, ON**						
Date Sampled :	**May 8, 2023**	**Borehole No:**	**BH7**	**Sample No.:**	**SS3**			**Depth (m) :**	**1.5-2.1**
Sample Description :		**% Silt and Clay**	**37**	**% Sand**	**57**	**% Gravel**	**6**	**Figure :**	**19**
Sample Description :	**FILL: Silty Sand (SM) - Trace Gravel and Clay**								

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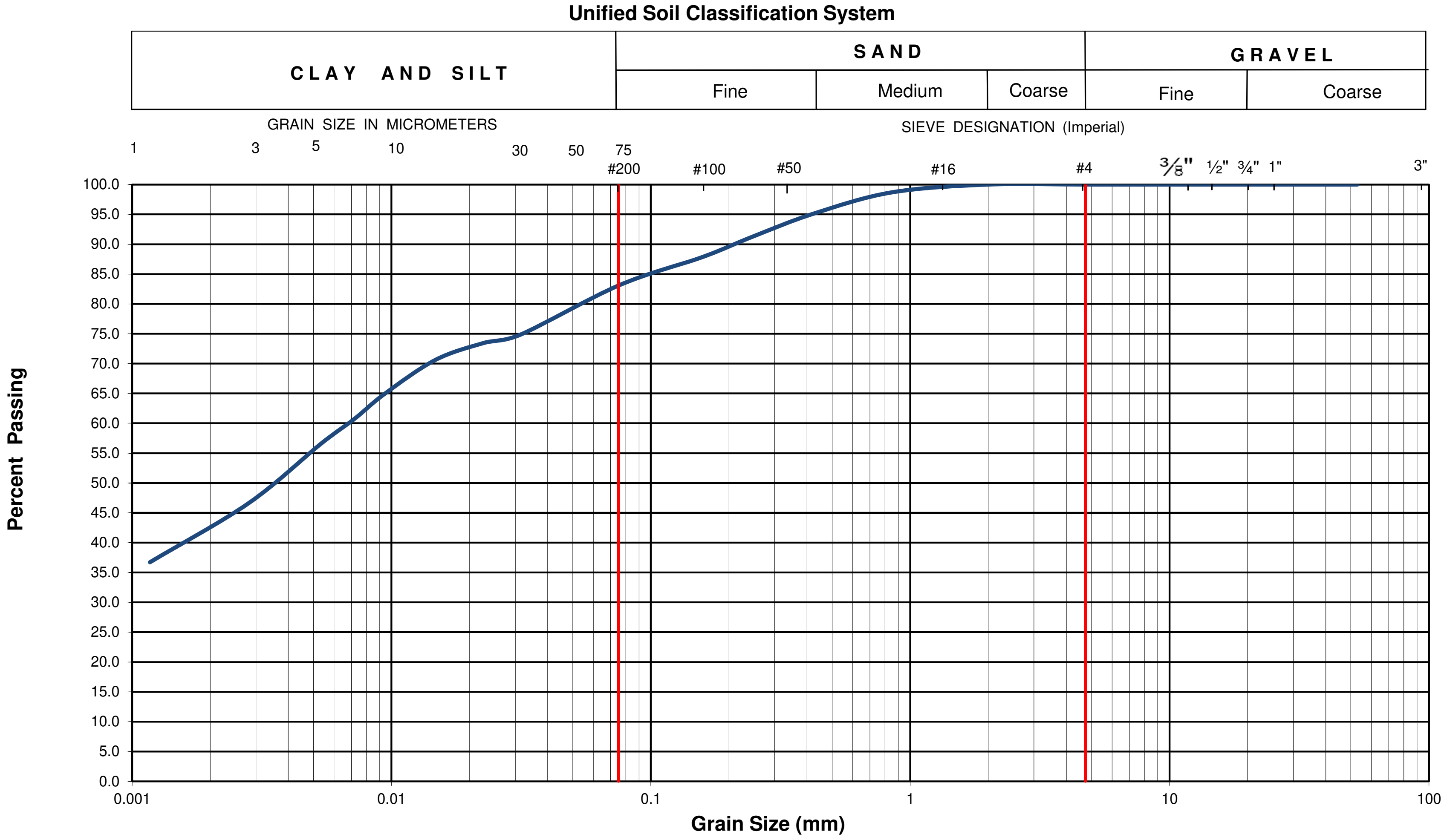
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.: OTT-22015769-A0		**Project Name :** Proposed Residential High-Rise Towers					
Client : RioCan Real Estate Investment Trust		**Project Location :** 1640-1660 Carling Avenue, Ottawa, ON					
Date Sampled : May 9, 2023		**Borehole No:** BH5		**Sample No.:** SS4		**Depth (m) :**	2.3-2.9
Sample Description :		**% Silt and Clay**	83	**% Sand** 17	**% Gravel** 0	**Figure :**	20
Sample Description :	SILTY CLAY of Medium Plasticity (CL) - Some Sand						

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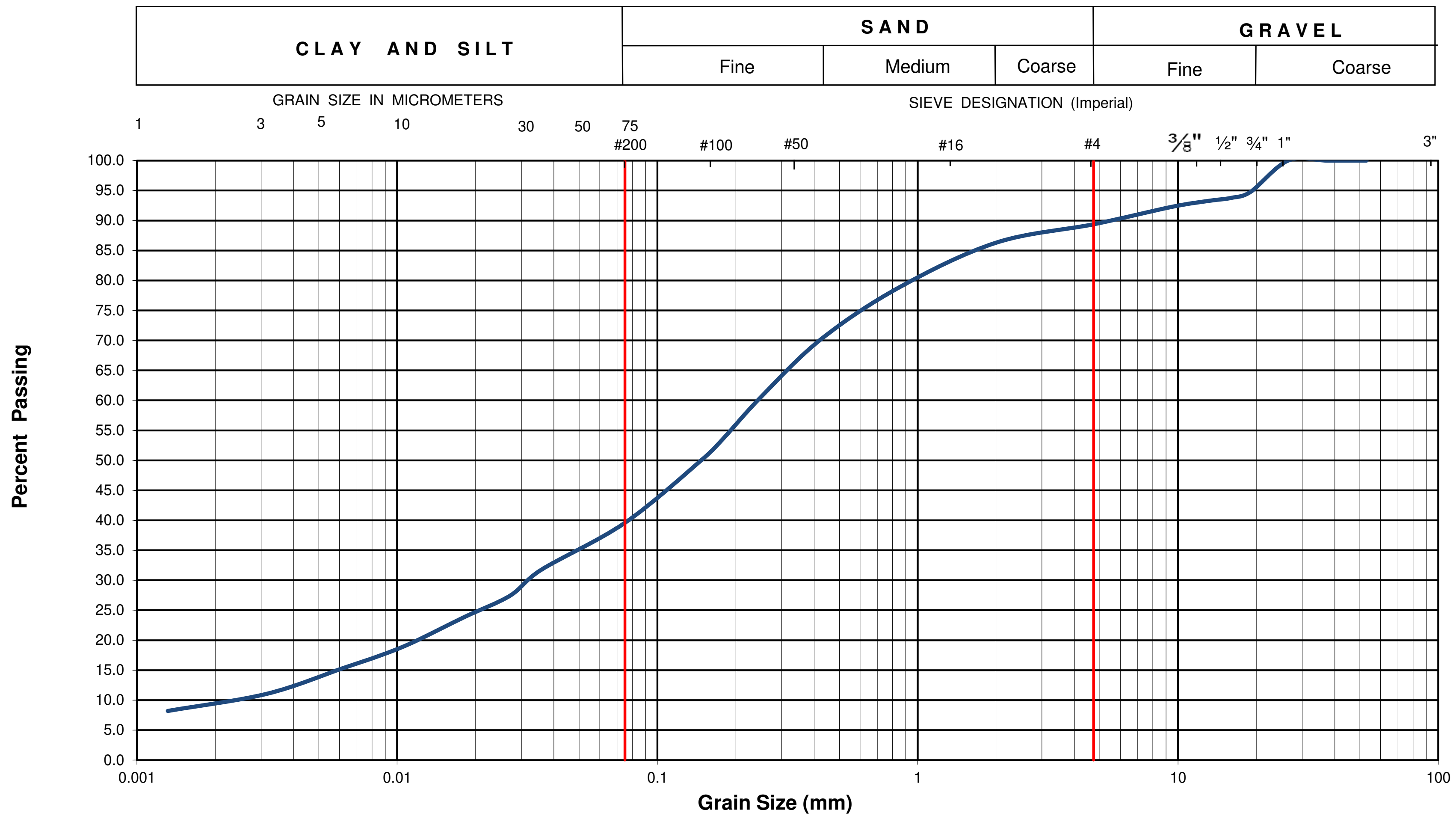
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422

Unified Soil Classification System



EXP Project No.:	**OTT-22015769-A0**	**Project Name :**	**Proposed Residential High-Rise Towers**						
Client :	**RioCan Real Estate Investment Trust**	**Project Location :**	**1640-1660 Carling Avenue, Ottawa, ON**						
Date Sampled :	**May 5, 2023**	**Borehole No:**	**BH4**	**Sample No.:**	**SS4**			**Depth (m) :**	**2.3-2.9**
Sample Description :		**% Silt and Clay**	**40**	**% Sand**	**49**	**% Gravel**	**11**	**Figure :**	**21**
Sample Description :	**GLACIAL TILL: Silty Sand (SM) - Some Gravel and Clay**								

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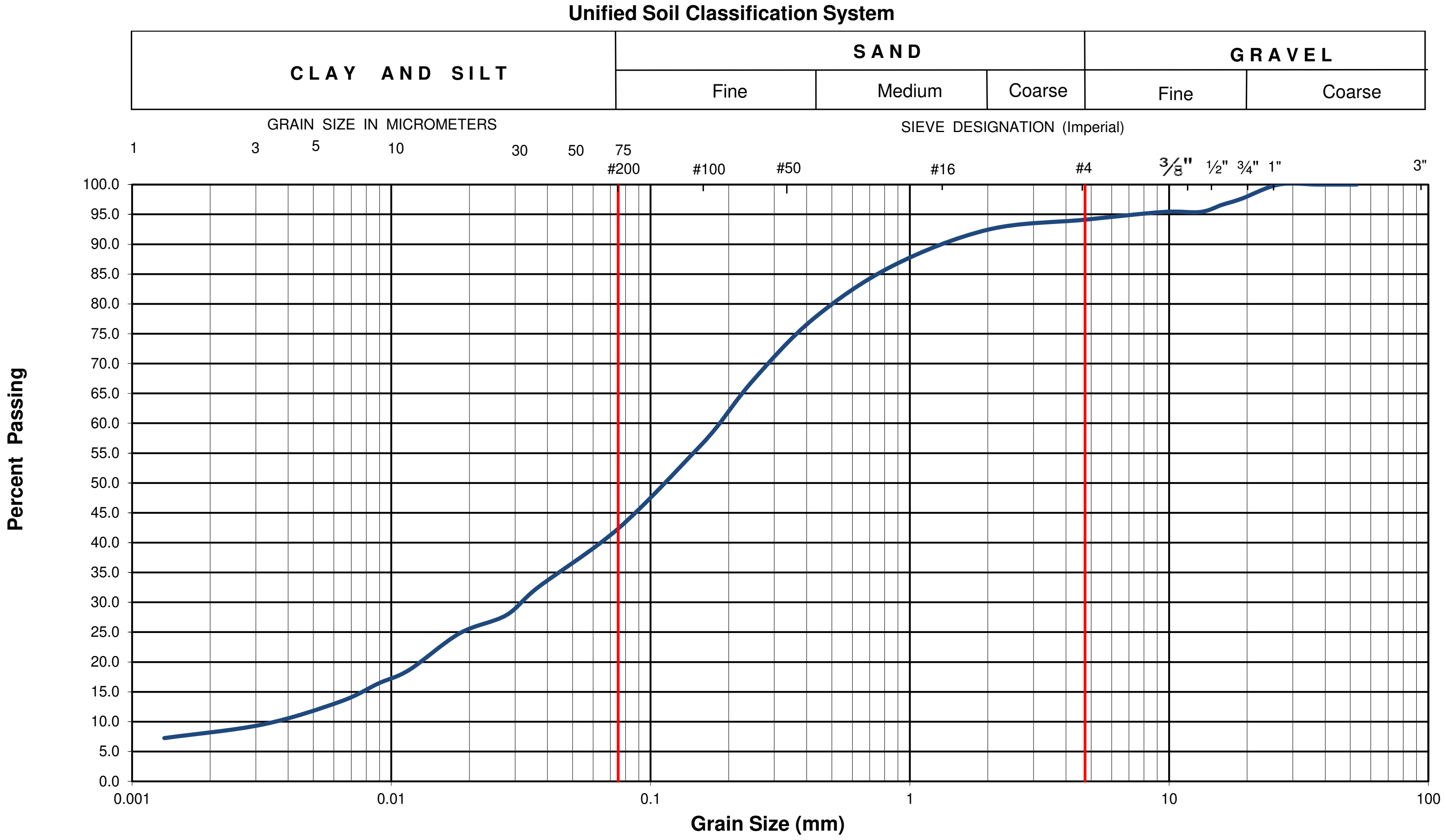
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-22015769-A0**	**Project Name :**	**Proposed Residential High-Rise Towers**						
Client :	**RioCan Real Estate Investment Trust**	**Project Location :**	**1640-1660 Carling Avenue, Ottawa, ON**						
Date Sampled :	**May 5, 2023**	**Borehole No:**		**BH8**	**Sample No.:**		**SS3**	**Depth (m) :**	**1.5-2.1**
Sample Description :		**% Silt and Clay**	**42**	**% Sand**	**52**	**% Gravel**	**6**	**Figure :**	**22**
Sample Description :	**GLACIAL TILL: Silty Sand (SM) - Trace Gravel and Clay**								

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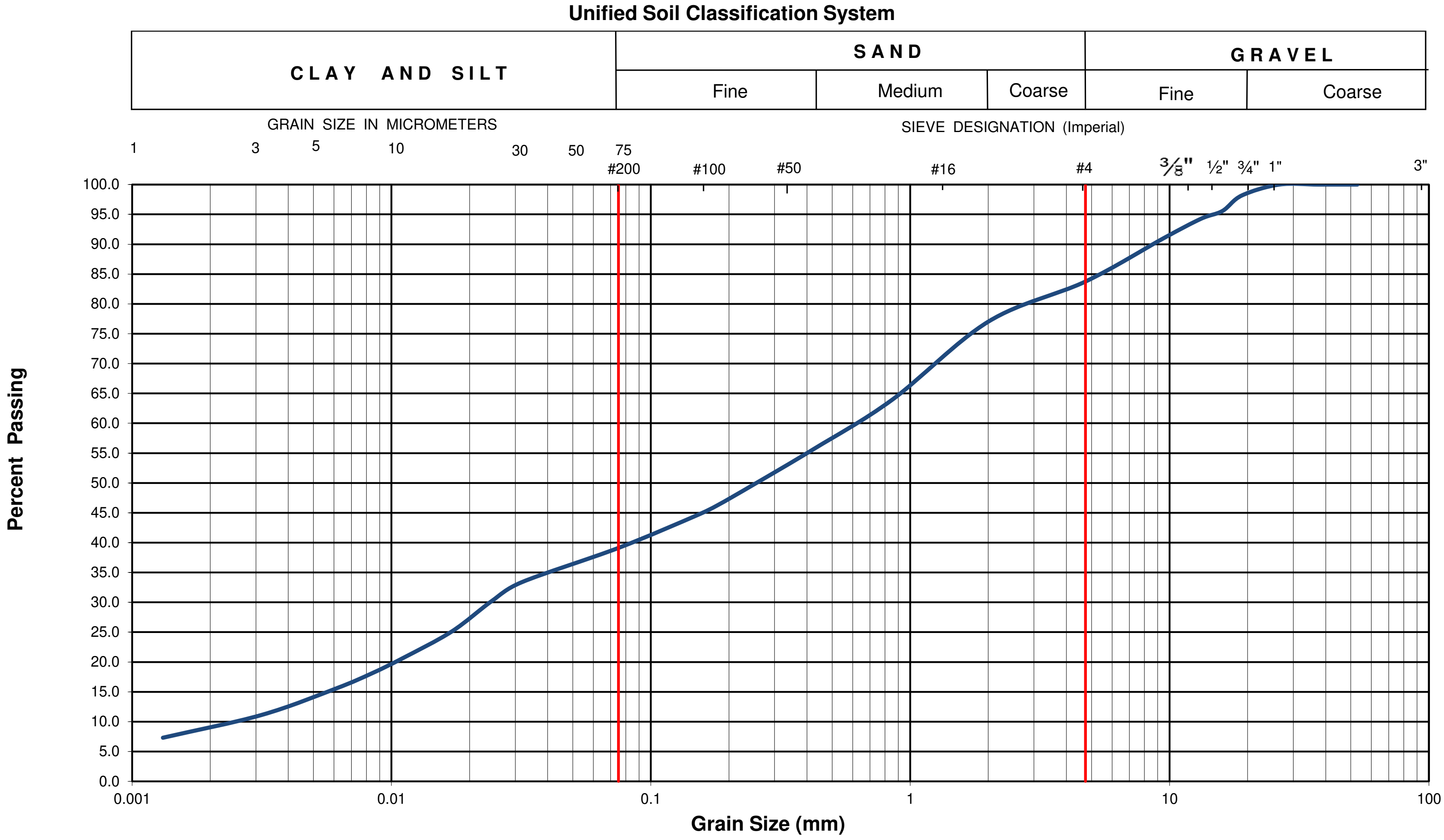
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.: OTT-22015769-A0	**Project Name :** Proposed Residential High-Rise Towers						
Client : RioCan Real Estate Investment Trust	**Project Location :** 1640-1660 Carling Avenue, Ottawa, ON						
Date Sampled : May 4, 2023	**Borehole No:**	ENV-5		**Sample No.:** SS4			**Depth (m) :** 2.3-2.9
Sample Description :	**% Silt and Clay**	39	**% Sand**	45	**% Gravel**	16	**Figure :** 23
Sample Description : GLACIAL TILL: Silty Sand (SM) - Some Gravel and Trace Clay							

Appendix A – Bedrock Core Rock Photographs

DRY BEDROCK CORES

WET BEDROCK CORES

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Ottawa, ON K2B 8H6
Canada
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borehole no. **BH-2**	core runs Run 1: 2.5 to 3.3 Run 2: 3.3 to 4.9 Run 3: 4.9 to 6.4 Run 4: 6.4 to 7.9 Run 5: 7.9 to 9.4 Run 6: 9.4 to 10.9 Run 7: 10.9 to 12.4	PROJECT Geotechnical Investigation - 1640 to 1660 Carling Avenue, Ottawa	project no. OTT-22015769-A0
date cored May 08, 2023		Rock Core Photographs	Figure: A-1

DRY BEDROCK CORES

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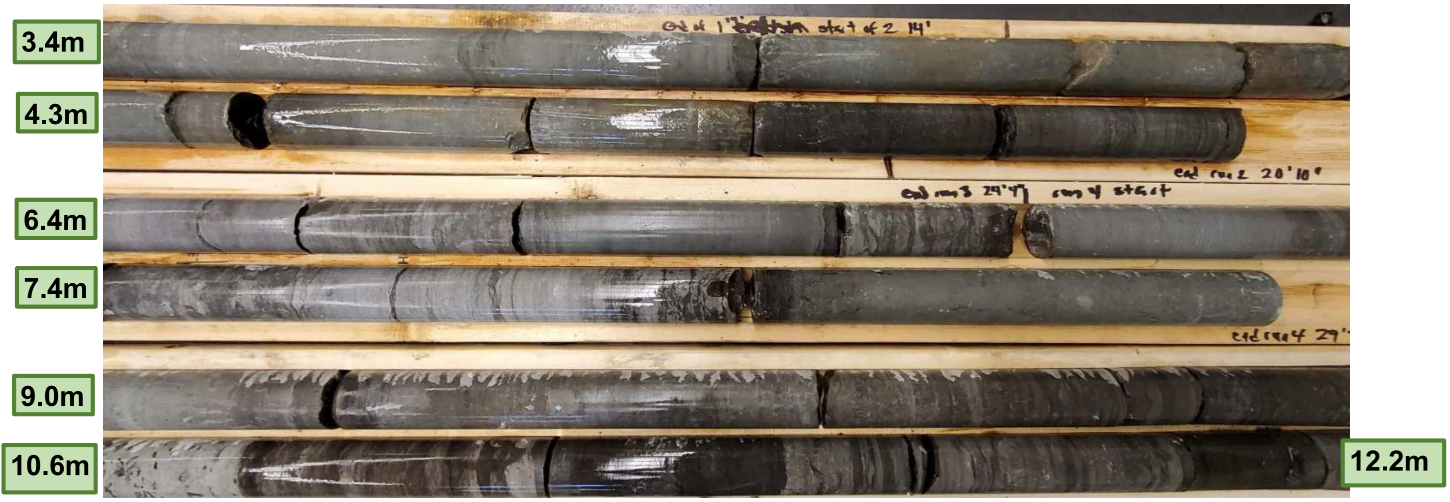

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borehole no. **BH-4**	core runs Run 1: 3.4 to 4.3 Run 2: 4.3 to 6.4 Run 3: 6.4 to 7.4 Run 4: 7.4 to 9.0 Run 5: 9.0 to 10.6 Run 6: 10.6 to 12.2	PROJECT Geotechnical Investigation - 1640 to 1660 Carling Avenue, Ottawa	project no. OTT-22015769-A0
date cored May 05, 2023		Rock Core Photographs	Figure: A-2

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borehole no. **BH-5**	core runs Run 1: 3.4 to 4.3 Run 2: 4.3 to 5.9 Run 3: 5.9 to 7.4 Run 4: 7.4 to 8.9 Run 5: 8.9 to 10.4 Run 6: 10.4 to 11.9	PROJECT Geotechnical Investigation - 1640 to 1660 Carling Avenue, Ottawa	project no. OTT-22015769-A0
date cored May 09, 2023		Rock Core Photographs	Figure: A-3

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borehole no. **BH-7**	core runs Run 1: 3.1 to 4.6 Run 2: 4.6 to 6.1 Run 3: 6.1 to 7.7 Run 4: 7.7 to 9.1 Run 5: 9.1 to 10.7 Run 6: 10.7 to 12.2	PROJECT Geotechnical Investigation - 1640 to 1660 Carling Avenue, Ottawa	project no. OTT-22015769-A0
date cored May 08, 2023		Rock Core Photographs	Figure A-4

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borehole no. **BH-9**	core runs Run 1: 2.5 to 3.3 Run 2: 3.3 to 4.9 Run 3: 4.9 to 6.4 Run 4: 6.4 to 7.9 Run 5: 7.9 to 9.3 Run 6: 9.3 to 11.0 Run 7: 11.0 to 12.4	PROJECT Geotechnical Investigation - 1640 to 1660 Carling Avenue, Ottawa	project no. OTT-22015769-A0
date cored May 10, 2023		Rock Core Photographs	Figure: A-5

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borehole no. **BH-11**	core runs Run 1: 3.0 to 3.3 Run 2: 3.3 to 4.9 Run 3: 4.9 to 6.4 Run 4: 6.4 to 7.9 Run 5: 7.9 to 9.4 Run 6: 9.4 to 11.0 Run 7: 11.0 to 12.4	PROJECT Geotechnical Investigation - 1640 to 1660 Carling Avenue, Ottawa	project no. OTT-22015769-A0
date cored May 09, 2023		Rock Core Photographs	Figure A-6

Appendix B – Seismic Shear Wave Velocity Sounding Report by GPR

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

June 23rd, 2023

Transmitted by email: ismail.taki@exp.com
Our Ref.: GPR23-04556-a

Mr. Ismail Taki, M.Eng., P.Eng.
Senior Manager, Earth & Environment, Eastern Region
exp Services inc.
100 – 2650 Queensview Drive
Ottawa ON K2B 8H6

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
1640 – 1660 Carling Avenue, Ottawa (ON)

[Project: OTT-22015769-A0]

Dear Sir,

Geophysics GPR International inc. has been mandated by **exp** Services inc. to carry out seismic surveys at 1640 – 1660 Carling Avenue, in Ottawa (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Spatial AutoCorrelation (SPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocity values were calculated for the soil and the rock, to determine the Site Class.

The surveys were carried out on June 14th, 2023, by Mr. Mario Nucciarone, B.Sc. geophysics and Mrs. Anne-Catherine Cyr, trainee. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

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

MASW PRINCIPLE

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

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

INTERPRETATION

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

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_S) is around 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 spreads were installed on a parking lot (Figure 2). The geophone spacing was 3.0 metres for the main spread, using 24 geophones. Two shorter seismic spreads, with geophone spacing of 0.5 and 1.0 metre, were dedicated to the near surface materials. The seismic records were produced with a seismograph Terraloc Pro 2 (from ABEM Instrument), and the geophones were 4.5 Hz.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and at 40 µs for the seismic refraction. The records included a pre-trigged portion of 10 ms. An 8 kg sledgehammer was used as the energy source, with impacts being recorded off both ends of the seismic spreads. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

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

RESULTS

The MASW calculated V_S results are illustrated at Figure 5.

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

$$\bar{V}_{S30} = \frac{\sum_{i=1}^{N} \mathrm{H}_i}{\sum_{i=1}^{N} {H_i}/{V_i}} \ \mid \ \sum_{i=1}^{N} \mathrm{H}_i \ = \ 30 \text{ m}$$

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

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

The calculated $\overline{V}_{S30}$ value of the actual site is 1176.8 m/s (Table 1), corresponding to the Site Class "B". However, the Site Classes A and B are not to be used if there is 3 metres or more of soils between the rock and the bottom of the spread footing, pile cap or mat foundation. In the case the bottom of the foundation would be 1.2 metres or less from the rock, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" (Table 2).

CONCLUSION

Geophysical surveys were carried out to identify the Site Class at 1640 – 1660 Carling Avenue, in Ottawa (ON). The seismic surveys used the MASW and the SPAC analysis, and the seismic refraction to calculate the $\overline{V}_{S30}$ value. Its calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 1177 m/s, corresponding to the Site Class "B" ($760 < \overline{V}_{S30} \leq 1500$ m/s), as determined through the MASW and SPAC methods, Table 4.1.8.4.-A of the NBC (2015), and the Building Code, O. Reg. 332/12. It must be noted that the Site Classes A and B are not to be used if there is 3 metres or more of soils between the rock and the bottom of the spread footing, pile cap or mat foundation.

In the case the bottom of the foundation would be 1.2 metres or less from the rock surface, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" ($\overline{V}_{S30} > 1500$ m/s).

It must also be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, very soft clays, high moisture content etc. (cf. Table 4.1.8.4.-A of the NBC 2015) can supersede the Site classification provided in this report based on the $\overline{V}_{S30}$ value.

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

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

P. Eng.

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

J.-L. ARSENAULT
100148122
2023-06-23

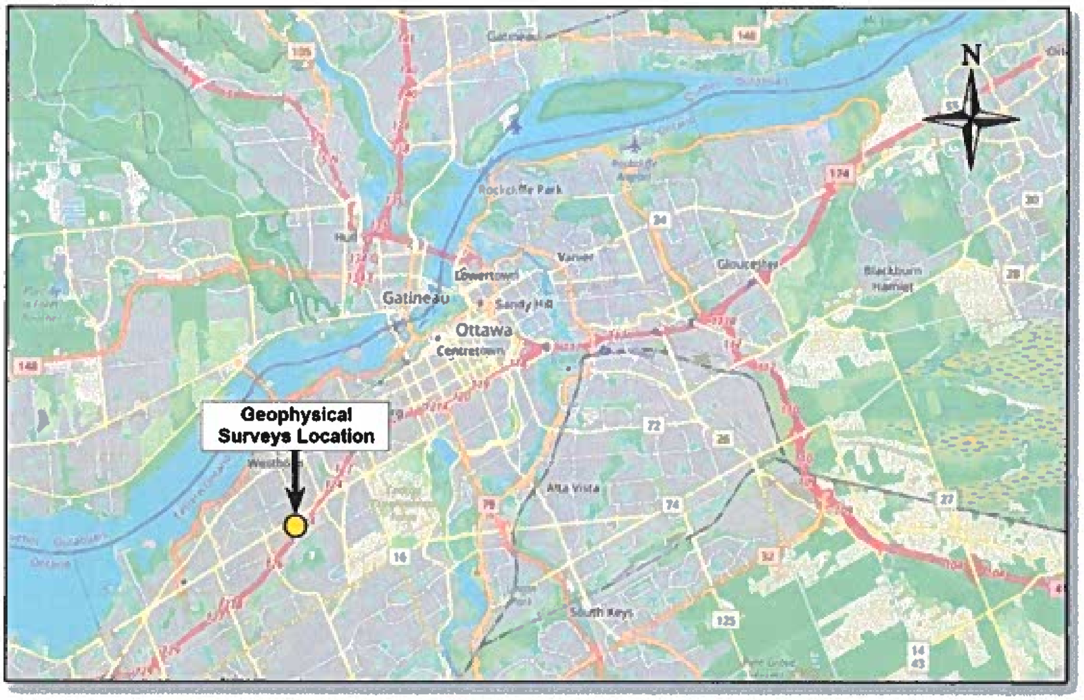


Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

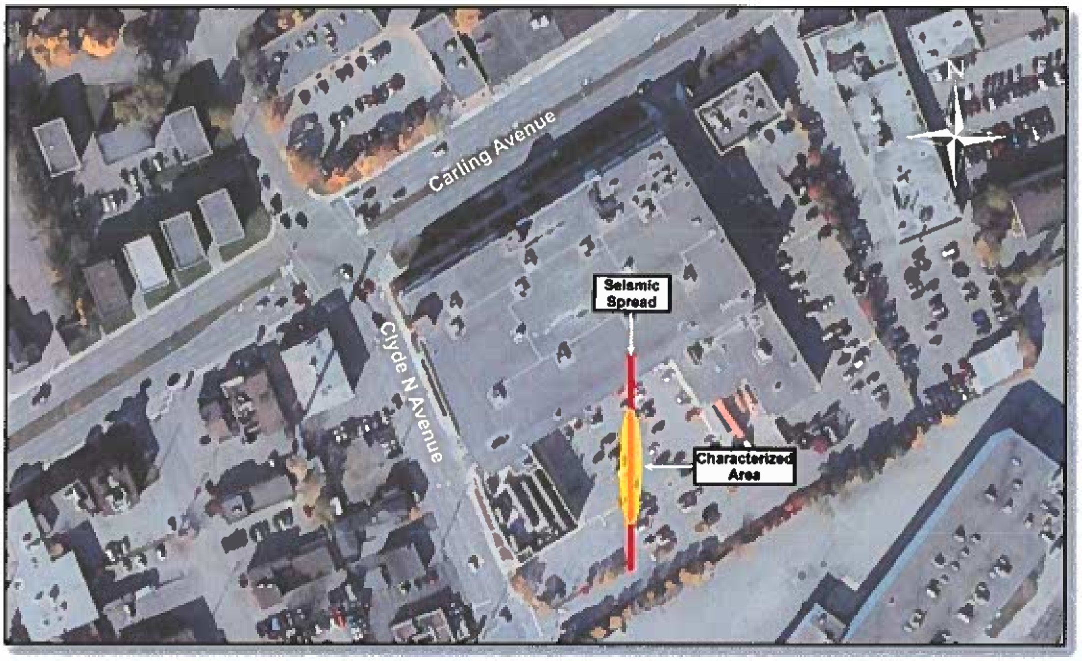


Figure 2: Location of the seismic spreads
(source: *geoOttawa*)

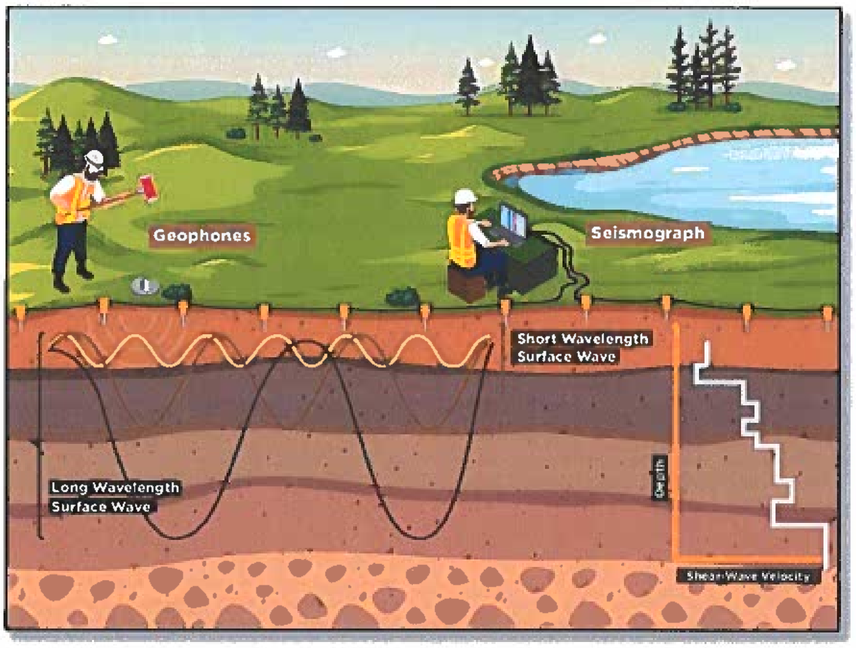
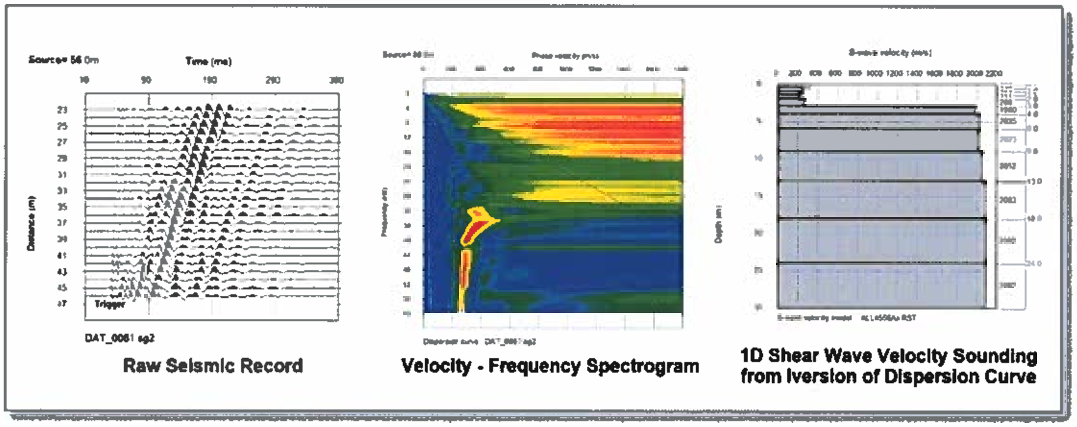


Figure 3: MASW Operating Principle



Raw Seismic Record

Velocity - Frequency Spectrogram

1D Shear Wave Velocity Sounding from Iversion of Dispersion Curve

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

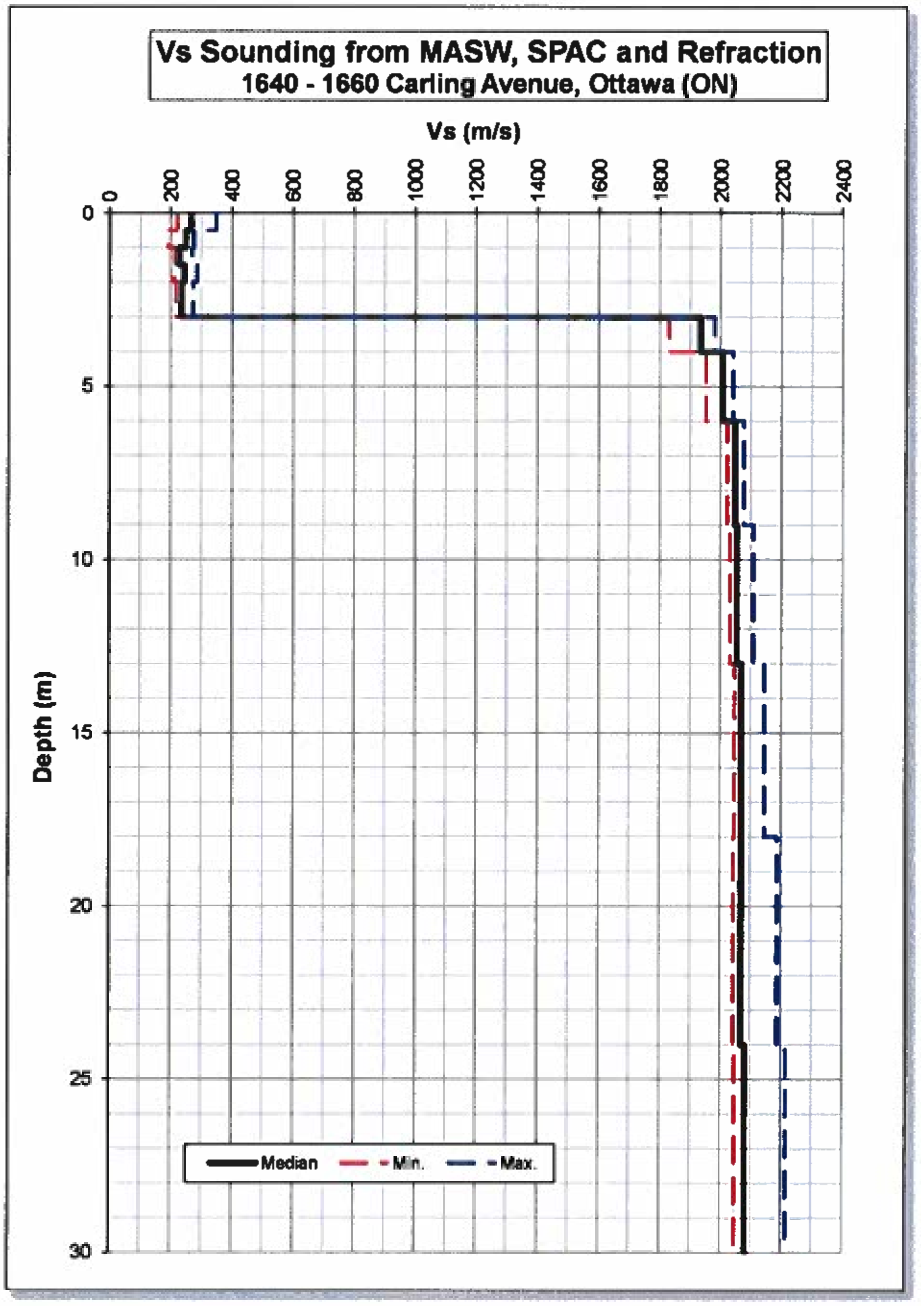


Figure 5: MASW Shear-Wave Velocity Sounding

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

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	220.9	267.0	347.5	**Grade Level (June 14, 2023)**				
0.5	192.1	248.8	274.6	0.50	0.50	0.001873	0.001873	267.0
1.0	214.6	227.5	268.9	0.50	1.00	0.002010	0.003883	257.6
1.5	211.7	243.6	285.6	0.50	1.50	0.002198	0.006080	246.7
2.0	217.3	235.9	272.6	0.50	2.00	0.002053	0.008133	245.9
3.0	1831.0	1933.6	1980.1	1.00	3.00	0.004240	0.012373	242.5
4.0	1951.3	2007.9	2041.9	1.00	4.00	0.000517	0.012890	310.3
6.0	2023.3	2048.1	2078.3	2.00	6.00	0.000996	0.013886	432.1
9.0	2032.9	2054.7	2109.5	3.00	9.00	0.001465	0.015351	586.3
13.0	2048.2	2070.0	2145.2	4.00	13.00	0.001947	0.017297	751.6
18.0	2045.1	2070.3	2186.9	5.00	18.00	0.002415	0.019713	913.1
24.0	2048.2	2082.3	2215.4	6.00	24.00	0.002898	0.022611	1061.4
30				6.00	30.00	0.002881	0.025492	1176.8

V_{S30} (m/s)	**1176.8**
Class	**B (1)**

(1) The Site Classes A and B are not to be used if there is 3 metres or more of soils between the rock and the bottom of the spread footing, pile cap or mat foundation.

TABLE 2
Limit for the Site Class A

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	220.9	267.0	347.5	**Limit for the Site Class A (1.4 metres of soil)**				
0.5	192.1	248.8	274.6					
1.0	214.6	227.5	268.9					
1.5	211.7	243.6	285.6					
1.6	211.7	243.6	285.6					
2.0	217.3	235.9	272.6	0.40	0.40	0.001642	0.001642	243.6
3.0	1831.0	1933.6	1980.1	1.00	1.40	0.004240	0.005882	238.0
4.0	1951.3	2007.9	2041.9	1.00	2.40	0.000517	0.006399	375.1
6.0	2023.3	2048.1	2078.3	2.00	4.40	0.000996	0.007395	595.0
9.0	2032.9	2054.7	2109.5	3.00	7.40	0.001465	0.008860	835.2
13.0	2048.2	2070.0	2145.2	4.00	11.40	0.001947	0.010806	1054.9
18.0	2045.1	2070.3	2186.9	5.00	16.40	0.002415	0.013222	1240.4
24.0	2048.2	2082.3	2215.4	6.00	22.40	0.002898	0.016120	1389.6
31.6				7.60	30.00	0.003650	0.019770	1517.5

V_{S30}*	**1517.5**
Class	**A**

Appendix C – Laboratory Certificate of Analysis

5835 COOPERS AVENUE
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FAX (905)712-5122
http://www.agatlabs.com

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2650 QUEENSVIEW DRIVE, UNIT 100
OTTAWA, ON K2B8H6
(613) 688-1899
ATTENTION TO: **Susan Potyondy**
PROJECT: **OTT-22015769-A0**
AGAT WORK ORDER: **23Z025305**
SOIL ANALYSIS REVIEWED BY: **Nivine Basily, Inorganics Report Writer**
DATE REPORTED: **May 25, 2023**
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

Disclaimer:

- *All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.*
- *All samples will be disposed of within 30 days after receipt unless a Long Term Storage Agreement is signed and returned. Some specialty analysis may be exempt, please contact your Client Project Manager for details.*
- *AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.*
- *This Certificate shall not be reproduced except in full, without the written approval of the laboratory.*
- *The test results reported herewith relate only to the samples as received by the laboratory.*
- *Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to, warranties of merchantability, fitness for a particular purpose, or non-infringement. AGAT assumes no responsibility for any errors or omissions in the guidelines contained in this document.*
- *All reportable information as specified by ISO/IEC 17025:2017 is available from AGAT Laboratories upon request.*
- *For environmental samples in the Province of Quebec: The analysis is performed on and results apply to samples as received. A temperature above 6°C upon receipt, as indicated in the Sample Reception Notification (SRN), could indicate the integrity of the samples has been compromised if the delay between sampling and submission to the laboratory could not be minimized.*

Member of: Association of Professional Engineers and Geoscientists of Alberta (APEGA)
Western Enviro-Agricultural Laboratory Association (WEALA)
Environmental Services Association of Alberta (ESAA)

AGAT Laboratories is accredited to ISO/IEC 17025 by the Canadian Association for Laboratory Accreditation Inc. (CALA) and/or Standards Council of Canada (SCC) for specific tests listed on the scope of accreditation. AGAT Laboratories (Mississauga) is also accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA) for specific drinking water tests. Accreditations are location and parameter specific. A complete listing of parameters for each location is available from www.cala.ca and/or www.scc.ca. The tests in this report may not necessarily be included in the scope of accreditation. Measurement Uncertainty is not taken into consideration when stating conformity with a specified requirement.

Certificate of Analysis

AGAT WORK ORDER: 23Z025305

PROJECT: OTT-22015769-A0

CLIENT NAME: EXP SERVICES INC

ATTENTION TO: Susan Potyondy

SAMPLING SITE:1640-1660 Carling Ave., Ottawa

SAMPLED BY:EXP

Inorganic Chemistry (Soil)

DATE RECEIVED: 2023-05-16

DATE REPORTED: 2023-05-25

			SAMPLE DESCRIPTION:	BH4 SS3 5'-7'	BH2 Run 6 30'9'-35'10"	BH5 Run 5 29'4"-34'2"	BH11 Run 6 31'-36'
			SAMPLE TYPE:	Soil	Rock	Rock	Rock
			DATE SAMPLED:	2023-05-05	2023-05-08	2023-05-09	2023-05-09
Parameter	**Unit**	**G / S**	**RDL**	**4997456**	**4997457**	**4997458**	**4997459**
pH (2:1)	pH Units		NA	8.02	9.11	9.37	9.49
Chloride (2:1)	µg/g		2	523	34	60	31
Sulphate (2:1)	µg/g		2	152	436	262	281
Resistivity (2:1) (Calculated)	ohm.cm		1	833	1180	1430	1480

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

4997456-4997459 pH, Chloride and 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.

Analysis performed at AGAT Toronto (unless marked by *)

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-22015769-A0
SAMPLING SITE:1640-1660 Carling Ave., Ottawa

AGAT WORK ORDER: 23Z025305
ATTENTION TO: Susan Potyondy
SAMPLED BY:EXP

Soil Analysis

RPT Date: May 25, 2023			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Inorganic Chemistry (Soil)															
pH (2:1)	4997456	4997456	8.02	8.12	1.2%	NA	91%	80%	120%						
Chloride (2:1)	4990686		852	919	7.6%	< 2	94%	70%	130%	94%	80%	120%	NA	70%	130%
Sulphate (2:1)	4990686		86	89	3.4%	< 2	99%	70%	130%	96%	80%	120%	93%	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.

Matrix spike NA: Spike level < native concentration. Matrix spike acceptance limits do not apply and are not calculated.

Certified By:

NIVINE BASILY

Method Summary

CLIENT NAME: EXP SERVICES INC
AGAT WORK ORDER: 23Z025305
PROJECT: OTT-22015769-A0
ATTENTION TO: Susan Potyondy
SAMPLING SITE:1640-1660 Carling Ave., Ottawa
SAMPLED BY:EXP

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
pH (2:1)	INOR 93-6031	modified from EPA 9045D and MCKEAGUE 3.11	PH METER
Chloride (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Resistivity (2:1) (Calculated)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION

AGAT Laboratories

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 #: 23Z025305

Cooler Quantity: one-no ice packs

Arrival Temperatures: 23.0 | 23.1 | 23.1
4.7 | 4.9 | 5.2

Custody Seal Intact: ☐ Yes ☐ No ☑ N/A

Notes: bagged ice

Report Information:

Company: EXP Services Inc.
Contact: Susan Potyondy
Address: 2650 Queensview dr. Unit 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)

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

☐ Excess Soils R406
Table ___ *Indicate One*
☐ Regulation 558
☐ CCME

☐ Sewer Use
☐ Sanitary ☐ Storm
___ *Region*
☐ 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 (Most Analysis) ☑ 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-22015769-A0
Site Location: 1640-1660 Carling ave., Ottawa
Sampled By: EXP
AGAT ID #: 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, DOC Y/N	O. Reg 153 Metals & Inorganics	O. Reg 153 Metals - ☐ CrVI, ☐ Hg, ☐ HWSB	O. Reg 153 BTEX, F1-F4 PHCs Analyze F4G if required ☐ Yes ☐ No	PAHs	Total PCBs ☐ Aroclor	VOC	O. Reg 558 Landfill Disposal Characterization TCLP: TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	O. Reg 406 Excess Soils SPLP Rainwater Leach SPLP: ☐ Metals ☐ VOCs ☐ SVOCs	O. Reg 406 Excess Soils Characterization Package pH, ICPMS Metals, BTEX, F1-F4	Salt - EC/SAR	pH	sulphate	chloride	resistivity				Potentially Hazardous or High Concentration (Y/N)
BH 4 SS3 5'-7'	May 5	— AM PM	1	S													✓	✓	✓	✓				
BH 2 run 6 30'9"-35'10"	May 8	— AM PM	1	rock													✓	✓	✓	✓				
BH 5 run 5 29'4"-34'2"	May 9	— AM PM	1	|													✓	✓	✓	✓				
BH 11 run 6 31'-36'	May 9	— AM PM	1	|													✓	✓	✓	✓				
		AM PM																						
		AM PM																						
		AM PM																						
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		AM PM																						
		AM PM																						
		AM PM																						

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Jeff MacMillan Jeff M	May 16/23	3:40PM	C. Griffith	MAY 16 2023	15h45
CG to Duro	MAY 17 2023	16h00	T. K	May 18	8:39AM

Page 1 of 1

Nº: T 122906

Pink Copy - Client I Yellow Copy - AGAT I White Copy- AGAT

Legal Notification

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