Geotechnical Investigation

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Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed residential development to be located at 1740-1760 St. Laurent Boulevard, Ottawa, Ontario. Written authorization to proceed with the geotechnical investigation was provided by Mr. Carmine Zayoun on behalf of 11421247 Canada Inc on June 23, 2020.

The proposed residential development will be located on a 1.8-hectare parcel of land registered by the street address 1740-1760 St. Laurent Boulevard, Ottawa, Ontario. The site is currently occupied by four (4) structures (St. Hubert Restaurant, 168 Sushi and Petro-Canada service station (two buildings)) which will be demolished as part of the proposed development. The development will comprise of the construction of four (4) towers that will include two 12 storey towers and two 15 storey towers. A three-level underground parking garage will be located beneath the four (4) towers with the lowest slab at approximately 9.0 m depth below the ground floor level. The elevation of the ground floor of the proposed buildings is not known but is assumed to be near the elevation of the existing ground surface of the site.

Phase One and Two Environmental Site Assessments (ESAs) were conducted by EXP concurrently with this geotechnical investigation and the results of these assessments are reported in separate documents.

A Phase Two Environmental Site Assessment (ESA) was undertaken in 2019 by EXP at 1760 St. Laurent Boulevard and consists of four (4) boreholes equipped with monitoring wells (MWs 1 to 4). The locations and the borehole logs of these monitoring wells are included in this report. The monitoring wells have been relabelled to MWs 19-01 to 19-04 to reflect the year the boreholes and monitoring well installations were completed.

The fieldwork for this geotechnical investigation was undertaken between August 20 and 25, 2020 and consists of ten (10) boreholes (BH Nos. 20-05 to 20-14) advanced to auger refusal and termination depths ranging from 5.9 m to 17.7 m below existing grade. The presence of the bedrock was proven in three (3) boreholes by conventional coring techniques. Standpipes (19 mm diameter) and monitoring wells (50 mm diameter) with slotted/screened sections were installed in selected boreholes for long-term monitoring of the groundwater levels and sampling of the groundwater. All boreholes were backfilled upon completion of drilling and sampling operations.

The borehole information indicates that the subsurface conditions consist of a surficial pavement structure and fill underlain by sand, silt and clay layers, silty clay, glacial till and shale bedrock contacted at 6.0 m to 7.7 m (Elevation 64.7 m to 62.8 m) below existing grade. The groundwater level ranges from 1.2 m to 3.2 m depths (Elevation 70.3 m to Elevation 67.9 m).

The overburden at the site is underlain by shale bedrock of the Carlsbad formation which is a type of shale that is prone to deterioration when exposed to the elements. It also heaves due to a complex mechanism caused in part from the bio-oxidation of the sulphides in the rock, which then react with calcite seams to form expanding gypsum. This occurs when oxygen is permitted to enter the rock, usually by lowering of the water table and is accelerated by the presence of heat. Therefore, special procedures will be required for the construction of footings and floor slabs and for the excavation in the shale bedrock as presented in detail in the main body of the attached report.

A shear wave velocity sounding survey (seismic shear wav survey) of the site was conducted by Geophysics GPR International Inc. acting as subcontractor to EXP and the survey results are presented in Appendix C. A review of Table 4.1.8.4.A of the 2012 Ontario Building Code (as amended May 2, 2019) and the seismic shear wave survey results, indicate that the site classification for seismic site response for this site is **Site Class A**, provided the footings of the proposed buildings are set on sound bedrock as recommended in the report.

It is anticipated that all subsurface soils on site including the fill and native soils will be excavated down to the bedrock and removed from site for the construction of the proposed new buildings. Since all subsurface soils will be excavated and removed from the site, liquefaction of the soils on site during a seismic event is not a concern.

From a geotechnical perspective there are no restrictions to raising the grades at the site since it is anticipated that all subsurface soils will be excavated down to the bedrock, removed from the site and replaced with imported granular fill compacted to the specified degree of compaction indicated in this report.

The geotechnical investigation revealed that it would be feasible to support the proposed buildings on spread and strip footings founded on the founded on the competent, sound bedrock (free of weathered zones, loose material, clay seams, fractures and voids0 and may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) 2.0 MPa. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement of the footing will be much larger than the recommended value for factored geotechnical resistance at ULS. Therefore, the factored geotechnical resistance at ULS will govern the design. Settlements of footing designed for the above recommended factored geotechnical resistance at ULS and properly constructed are expected to be less than 10 mm.

At Borehole No. 20-13, the shale bedrock from its surface at a 7.0 m depth (Elevation 64.1 m) to a 14.7 m depth (Elevation 56.4 m) is of a very poor quality and is not considered capable of supporting footings designed for the factored ULS value of 2.0 MPa. It is not known whether there is a fault in this area or this is a localized condition. Therefore, all broken bedrock in the vicinity of this borehole will have to be sub-excavated below the anticipated design footing founding depth of approximately 9.5 m to competent bedrock expected at 14.7 m depth (Elevation 56.4 m) and the excavation backfilled using concrete (with a compressive strength of 25 MPa) to the design founding elevation of the footing and the footing is cast on the concrete backfill. The lateral extent of the area requiring this treatment will be best established in the field during construction and should be allowed for as part of the construction budget.

As previously mentioned, special procedures will be required for the construction of the footings founded on the shale bedrock as discussed in the main body of the attached report.

The lowest slab of the underground parking garages of the proposed buildings may be designed as slabs-on-grade with perimeter and underfloor drainage systems. The surface of the floors may be either concrete or asphalt. Perimeter and underfloor drainage systems will be required. It is anticipated the lowest floor slabs will be founded on the shale bedrock. Special procedures will be required for the construction of the basement slabs-on-grade and the drainage systems founded on the shale bedrock, as discussed in the main body of the attached report.

Excavations should be undertaken in accordance with the current Occupational Health and Safety Act (OHSA).

The upper levels of the shale 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 side slopes within the weathered and sound bedrock may be undertaken near vertical but may need be cut back at a 1H:1V gradient in zones of loose rock pieces/slabs.

The excavation of the shale bedrock will likely require line drilling and blasting techniques. Contractors bidding on this project should decide on their own the most preferred rock removal method; hoe ramming or line drilling and blasting regardless of the depth and/or quantity of bedrock that requires excavation. As previously mentioned, special procedures will be required for excavations within the shale bedrock, as discussed in the main body of the attached report.

Foundation excavations for the proposed buildings are anticipated to extend to approximately 9.5 m depth below existing grade (locally deeper in the vicinity of Borehole No. 20-13) and will be below the shale bedrock surface and below the groundwater level and will likely have to be undertaken within the confines of a shoring system. In this regard, seepage of groundwater into the shored excavation should still be anticipated but may be removed by collecting the water at low points within the shored excavation and pumping from sumps. The need for large capacity pumps may be required to remove water from the within the shored excavation.

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

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

1. Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed residential development to be located at 1740-1760 St. Laurent Boulevard, Ottawa, Ontario. Written authorization to proceed with the geotechnical investigation was provided by Mr. Carmine Zayoun on behalf of 11421247 Canada Inc. on June 23, 2020.

The proposed residential development will be located on a 1.8-hectare parcel of land registered by the street address 1740-1760 St. Laurent Boulevard, Ottawa, Ontario. The site is currently occupied by four (4) structures (St. Hubert Restaurant, 168 Sushi and Petro-Canada service station (two buildings)) which will be demolished as part of the proposed development. The development will comprise of the construction of four (4) towers that will include two 12 storey towers and two 15 storey towers. A three-level underground parking garage will be located beneath the four (4) towers with the lowest slab at approximately 9.0 m depth below the ground floor level. The elevation of the ground floor of the proposed buildings is not known but is assumed to be near the elevation of the existing ground surface of the site.

Phase One and Two Environmental Site Assessments (ESAs) were conducted by EXP concurrently with this geotechnical investigation and the results of these assessments are reported in separate documents.

A Phase Two Environmental Site Assessment (ESA) was undertaken in 2019 by EXP at 1760 St. Laurent Boulevard and consists of four (4) boreholes equipped with monitoring wells (MWs 1 to 4). The locations and the borehole logs of these monitoring wells are included in this report. The monitoring wells have been relabelled to MWs 19-01 to 19-04 to reflect the year the boreholes and monitoring well installations were completed.

This geotechnical investigation was undertaken to:

a) Establish the subsurface soil, bedrock and groundwater conditions at the ten (10) borehole locations and the four (4) 2019 monitoring well locations at the site;

b) Classify the site for seismic site response in accordance with the requirements of the 2012 Ontario Building Code (as amended May 2, 2019) and assess the potential for liquefaction of the subsurface soils during a seismic event;

c) Comment on grade-raise restrictions;

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

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

f) Provide lateral earth pressure parameters (for static and seismic conditions) for the subsurface walls (basement walls) of the proposed parking garage;

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

h) Provide pipe bedding requirements for underground services;

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

j) Recommend pavement structure thicknesses for access roadways and surface parking facilities; and

k) Comment on subsurface concrete requirements and corrosion potential of the bedrock to buried metal structures/members.

The comments and recommendations given in this report 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.

2. Site Description

The site is located on the west side of St. Laurent Boulevard south of the Industrial Avenue and St. Laurent Boulevard intersection in Ottawa, Ontario. The location of the site is shown in Figure 1. The site is occupied by four (4) commercial buildings that front onto St. Laurent Boulevard. The north two (2) buildings are the service centre and car wash located at 1740 St. Laurent Boulevard, the central building is occupied by the St. Hubert Restaurant at 1754 St. Laurent Boulevard and the south building is the 138 Sushi Restaurant located at 1760 St. Laurent Boulevard. The rear of the three (3) properties are occupied by outdoor paved and gravel surface parking lots and access roads.

The topography of the site gradually slopes down in a south to north direction and an easterly direction towards St. Laurent Boulevard. The ground surface elevations of the 2020 boreholes indicates the ground surface in the west end of the site slopes down from south to north from Elevation 72.37 m to Elevation 71.07 m and slopes down towards the east to St. Laurent Boulevard to Elevation 69.68 m and Elevation 69.35 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 soil, 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 shale bedrock of the Carlsbad formation. This type of shale is prone to deterioration when exposed to the elements. It also heaves due to a complex mechanism caused in part from the bio-oxidation of the sulphides in the rock, which react with calcite seams to form expanding gypsum. This occurs when oxygen is permitted to enter the rock, usually by lowering of the water table and this process is accelerated by the presence of heat. Therefore, special treatment of the Carlsbad shale bedrock will need to be incorporated into the design and construction of the proposed buildings.

4. Procedure

4.1 Fieldwork

The fieldwork for this geotechnical investigation was undertaken between August 20 and 25, 2020 and consists of ten (10) boreholes (BH Nos. 20-05 to 20-14) advanced to auger refusal and termination depths ranging from 5.9 m to 17.7 m below existing grade.

A Phase Two Environmental Site Assessment (ESA) was undertaken in 2019 by EXP at 1760 St. Laurent Boulevard and consists of four (4) boreholes equipped with monitoring wells (MWs 1 to 4). The locations and the borehole logs of these monitoring wells are included in this report. The monitoring wells have been relabelled as MWs 19-01 to 19-04 to reflect the year the boreholes and monitoring well installations were completed. MWs 19-01 to 19-04 were drilled to termination and auger refusal depths of 6.1 m to 10.6 m below existing grade.

The borehole locations and geodetic elevations were established in the field by a survey crew from EXP. The borehole locations including the monitoring well locations from the 2019 Phase Two Environmental Site Assessment are shown in Figure 2.

The boreholes were drilled with a CME-75 truck-mounted drill rig equipped with continuous flight hollow-stem auger equipment and rock coring capabilities. Standard penetration tests (SPTs) was performed in all the boreholes at depth intervals ranging from a continuous basis to 0.75 m and 1.5 m depth intervals. The soil samples were retrieved by the split-barrel sampler, in accordance with the American Society for Testing and Materials (ASTM). The undrained shear strength of the clayey soils was measured by conducting penetrometer and in-situ vane tests. The presence of the bedrock was proven in three (3) boreholes by conventional coring techniques using NQ-size core barrel. A record of wash water return, colour of wash and any sudden drop of the drill rods were kept during rock coring operations.

Standpipes (19 mm diameter) and monitoring wells (50 mm diameter) with slotted/screened sections were installed in selected boreholes for long-term monitoring of the groundwater levels and sampling of the groundwater. The installation configuration of each standpipe and monitoring well is documented on the respective borehole log. All boreholes were backfilled upon completion of drilling and sampling operations.

4.2 Shear Wave Velocity Sounding Survey

A shear wave velocity sounding survey (seismic shear wave survey) of the site was conducted by Geophysics GPR International Inc. (GPR) on December 17, 2020 and the survey results are presented in Appendix C.

4.3 Laboratory Testing Program

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

The soil samples and rock cores were visually examined in the laboratory by a senior geotechnical engineer. The soil samples were classified in accordance with the Unified Soil Classification System (USCS) and the modified Burmeister System (as per the 2006 Fourth Edition Canadian Foundation Engineering Manual (CFEM)). Photographs of the rock cores were taken.

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

Table I: Summary of Laboratory Testing Program

Type of Test	Number of Tests Completed
Soil Samples	
Moisture Content Determination	93
Unit Weight Determination	13
Grain Size Analysis	7
Atterberg Limit Determination	5
Bedrock Cores	
Unit Weight Determination	9
Unconfined Compressive Strength Test	9
Corrosion Analysis (pH, sulphate, chloride and resistivity)	3

5. Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface conditions and groundwater levels from the monitoring wells (MWs) from the 2019 EXP Phase Two ESA for 1760 St. Laurent Boulevard and the boreholes (BHs) from this geotechnical investigation are given on the attached Borehole Logs, Figure Nos. 3 to 16 inclusive. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted. Boreholes were drilled to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions.

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

A review of the borehole logs including the 2019 monitoring wells indicates the following subsurface conditions with depth and groundwater level measurements.

5.1 Pavement Structure

The monitoring wells (MWs 19-01 to 19-04) and Borehole Nos. 20-5 to 20-10 are located in paved areas. The pavement structure consists of 35 mm to 100 mm thick asphaltic concrete underlain by 150 mm to 700 mm thick granular fill base consisting of sandy gravel to silty sand with crushed and pit-run gravel. Based on the standard penetration test (SPT) N-values of 11 to 24, the granular base fill is in a compact state.

5.2 Fill

Fill is present at the ground surface and beneath the pavement structure of Monitoring Well No. 19-03 and in all of the boreholes with the exception of Borehole No. 20-7 and extends to depths of 0.8 m to 4.5 m (Elevation 70.6 m to Elevation 66.1 m). The fill consists of silty sand with gravel to a mixture of silty sand, clayey silt and silty clay with gravel. The SPT N-values of 2 to 35 indicate the cohesionless sandy portion of the fill is in a very loose to dense state and the cohesive clayey portion has a firm to hard consistency. The moisture content of the fill ranges from 3 percent to 35 percent. The unit weight of the fill is 18.4 kN/m^3 to 24.0 kN/m^3.

The results from the grain-size analysis conducted on one (1) sandy portion of the fill are summarized in Table II. The grain-size distribution curve is shown in Figure 17.

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

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)			Soil Classification (USCS)
		Gravel	Sand	Fines (Silt and Clay)	
BH20-11 – SS2	1.5 – 2.1	5	77	18	Silty Sand (SM)

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

5.3 Sand Layer

The pavement structure and fill from Monitoring Well Nos. 19-02 and 19-03 are underlain by a native sand layer to 1.4 m and 2.0 m depths (Elevation 68.9 m and Elevation 68.3 m). The sand contains some silt and clay. Based on the SPT N-values of 6 and 9, the sand is in a loose state.

5.4 Silty Clay

Native silty clay was encountered below the pavement structure, fill and native sand layer in all four (4) monitoring wells (MWs 19-01 to 19-04) and in Borehole Nos. 20-07 to 20-09 and 20-14. The silty clay was not present in Borehole Nos. 20-05, 20-06 and 20-10 to 20-13. The silty clay was contacted at 0.2 m to 2.0 m depths (Elevation 70.6 m to Elevation 68.1 m) and extends to depths of 1.8 m to 4.5 m (Elevation 68.5 m to Elevation 64.9 m). In Borehole Nos. 20-09 and 20-14, the silty clay has an upper brown to brownish grey desiccated crust to 2.5 m and 3.0 m depths (Elevation 69.0 m and Elevation 66.9 m) underlain by a lower grey silty clay. The undrained shear strength of the upper silty clay crust ranges from 36 kPa to 225 kPa indicating a firm to hard consistency. The undrained shear strength of the grey silty clay is lower at 48 kPa and 72 kPa indicating a firm to stiff consistency. Based on sensitivity values of 4.5 to 8.0, the silty clay crust is considered to be sensitive. The natural moisture content of the upper brown crust ranges from 13 percent to 47 percent and 23 percent to 42 percent for the lower grey silty clay. The natural unit weight of the upper brown crust is 17.5 kN/m^3 to 19.4 kN/m^3. The natural unit weight of one (1) sample of the lower grey silty clay is 18.6 kN/m^3.

The results from the grain-size analysis and Atterberg limit determination conducted on one (1) sample of the brown crust and one (1) sample of the grey portion of the silty clay are summarized in Table III. The grain-size distribution curves are shown in Figures 18 and 19.

Table III: Summary of Results from Grain-Size Analysis and Atterberg Limit Determination
Silty Clay Samples

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)			Atterberg Limits (%)				
		Gravel	Sand	Fines (Silt and Clay)	Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification (USCS)
BH 20-08 – SS4	2.3 - 2.9	0	2	98	44	34	19	15	Upper Brown Crust of the Silty Clay: Low Plasticity (CL)
BH 20-14 – SS6	3.8 – 4.4	0	2	98	42	29	18	11	Lower Grey Silty Clay: Low Plasticity (CL)

Based on a review of the results of the grain-size analysis and Atterberg limits, the upper brown crust and lower grey silty clay may be classified as silty clay of low plasticity (CL) in accordance with the USCS.

5.5 Sand, Silt and Clay Layer

Beneath the silty clay to clay in Monitoring Well No. 19-04, a sand, silt and clay layer was contacted at 2.3 m depth (Elevation 67.7 m) and extends to a 3.1m depth (Elevation 66.9 m). Based on the SPT N-value of 6, this layer is in a loose state.

5.6 Glacial Till

Glacial till was contacted below the fill, silty clay and sand (silt and clay) layer in all of the monitoring wells and boreholes at 1.6 m to 4.5 m depths (Elevation 69.8 m to 64.9 m). The glacial till extends to depths of 5.3 m to 7.0 m (Elevation 65.5 m to Elevation 63.4 m). The composition of the glacial till contains varying amounts of gravel, sand, silt and clay with the soil matrix ranging from silty sand with gravel to clayey sand with gravel to silty clay. The glacial till also contains cobbles, boulders and possible large slab pieces of shale. The SPT N-values of the glacial till are from 2 to 51 indicating a very loose to very dense condition in the cohesionless sandy glacial till and a soft to hard consistency in the cohesive clayey glacial till. High N-values for low sampler penetration, such as 119 for 380 mm sampler penetration were recorded and may be a result of the sampler resting on a cobble, boulder or large slab piece of shale. The natural moisture content of the glacial till ranges from 6 percent to 28 percent. The natural unit weight of the glacial till is 23.9 kN/m^3 to 24.5 kN/m^3.

The results from the grain-size analysis and Atterberg limit determination conducted on four (4) samples of the glacial till are summarized in Table IV. The grain-size distribution curves are shown in Figures 20 to 23.

Table IV: 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 (%)			Atterberg Limits (%)				
		Gravel	Sand	Fines (Silt and Clay)	Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification (USCS)
BH 20-06 – SS8	5.3 – 5.9	2	74	24	-	-	-	-	Silty Sand (SM)
BH 20-07 – SS4 to SS6	2.3 – 4.4	25	33	42	7 - 17	22	14	8	Clayey Sand with Gravel (SC)
BH 20-08 – SS6	3.8 – 4.4	0	3	97	28	23	19	4	Silty Clay of Low Plasticity (CL-ML)
BH 20-14 – SS8	5.3 – 5.9	38	41	21	-	-	-	-	Silty Sand with Gravel (SM)

Based on a review of the results of the grain-size analysis and Atterberg limits, the glacial till may be classified as a silty sand (SM), silty sand with gravel (SM), clayey sand with gravel (SC) and a silty clay of low plasticity (CL-ML) in accordance with the USCS.

5.7 Shale Bedrock

Auger and soil sampler refusal was met in all of the monitoring wells and boreholes at 5.3 m to 7.2 m depths (Elevation 65.5 m to Elevation 63.4 m) on inferred weathered shale bedrock. Auger refusal was likely met on the shale bedrock in Monitoring Well Nos. 19-02 to 19-04 since it was possible to auger 100 mm to 1.4 m into the shale bedrock. The presence of the shale bedrock was proven by coring 3.5 m to 10.1 m of the bedrock in Monitoring Well No. 19-01 and in Borehole Nos. 20-05, 20-10 and 20-13.

A summary of the auger and soil sampler refusal depths on inferred weathered shale bedrock and the depth to bedrock confirmed by augering into the shale bedrock and by coring the bedrock is shown in Table V.

Table V: Summary of Auger and Soil Sampler Refusal and Bedrock Depths (Elevations) in Boreholes

Borehole (BH) No./Monitoring Well (MW) No.	Ground Surface Elevation (m)	Refusal Depth (Elevation) on Inferred Weathered Shale Bedrock (m)	Depth (Elevation) of Bedrock Surface (m)	Comment wrt to Depth (Elevation) of Bedrock Surface
MW 19-01	70.40	7.0 (63.4)	7.6 (62.8)	3.5 m length of bedrock cored below 7.6 m depth
MW 19-02	70.29	6.9 (63.4)	6.9 (63.4)	Augered 1.4 m into the bedrock below 6.9 m depth
MW 19-03	70.30	6.1 (64.2)	6.1 (64.2)	Augered 0.6 m into bedrock below 6.1 m depth
MW 19-04	70.0	6.0 (64.0)	6.0 (64.0)	Augered 100 mm into bedrock below the 6.0 m depth
BH 20-05	72.37	6.9 (65.5)	7.7 (64.7)	9.9 m length of bedrock cored below 7.7 m depth
BH 20-06	71.36	6.8 (64.6)	-	-
BH 20-07	70.27	5.3 (65.0)	-	-
BH 20-08	69.68	6.1 (63.6)	-	-

Table V: Summary of Auger and Soil Sampler Refusal and Bedrock Depths (Elevations) in Boreholes

Borehole (BH) No./Monitoring Well (MW) No.	Ground Surface Elevation (m)	Refusal Depth (Elevation) on Inferred Weathered Shale Bedrock (m)	Depth (Elevation) of Bedrock Surface (m)	Comment wrt to Depth (Elevation) of Bedrock Surface
BH 20-09	69.35	5.9 (63.5)	-	-
BH 20-10	69.86	6.0 (63.9)	6.1 (63.8)	8.6 m length of bedrock cored below 6.1 m depth
BH 20-11	70.68	7.2 (63.5)	-	-
BH 20-12	72.18	6.9 (65.3)	-	-
BH 20-13	71.07	7.0 (64.1)	7.6 (63.5)	10.1m length of bedrock cored below 7.6 m depth
BH 20-14	71.98	6.8 (65.2)	-	-

A review of Table V indicates the depth to bedrock based on augering into the bedrock and coring the bedrock ranges from 6.0 m to 7.6 m (Elevation 64.7 m to Elevation 62.8 m) below existing grade.

Based on the bedrock coring results, the total core recovery (TCR) ranges from 0 percent to 100 percent. The rock quality designation (RQD) ranges from 0 percent to 100 percent indicating the bedrock quality ranges widely from very poor to excellent quality. The bedrock appears to be highly weathered with clay seams and rubble zones (zones of broken rock) in the upper 0.2 m to 1.1 m of the shale bedrock in Monitoring Well No. 19-01, Borehole Nos. 20-05 and 20-10. The weathered very poor quality bedrock is quite extensive in Borehole No. 20-13, with the very poor rock extending 7.7 m below the bedrock surface to sound bedrock. A summary of the estimated depth (elevation) to sound bedrock at the monitoring well and borehole locations is shown in Table VI. Photographs of the rock cores are shown in Appendix B.

Table VI: Summary of Estimated Depth (Elevation) of Sound Bedrock in Boreholes

Borehole (BH) No./Monitoring Well (MW) No.	Ground Surface Elevation (m)	Depth (Elevation) of Bedrock Surface (m)	Estimated Depth (Elevation) to Sound Bedrock	Estimated Thickness of Weathered Bedrock (m)
MW 19-01	70.40	7.0 (63.4)	7.6 (62.8)	0.6
BH 20-05	72.37	6.9 (65.5)	7.9 (64.5)	1.0
BH 20-10	69.86	6.0 (63.9)	7.2 (62.7)	1.2
BH 20-13	71.07	7.0 (64.1)	14.7 (56.4)	7.7

Unit weight determination and unconfined compressive strength tests were conducted on nine (9) rock core sections and the results are summarized in Table VII.

Table VII: Summary of Unconfined Compressive Strength Test Results – Bedrock Cores				
Borehole (BH) No. – Run No.	**Depth (m)**	**Unit Weight (kN/m^3)**	**Unconfined Compressive Strength (MPa)**	**Classification of Rock with respect to Strength**
BH 20-05 – Run 2	9.6 – 9.7	25.7	59.8	Strong
BH 20-05 – Run 4	13.0 – 13.1	25.6	37.2	Medium Strong
BH 20-05 – Run 6	16.5 – 16.6	25.6	23.8	Weak
BH 20-05 – Run 7	16.4 – 16.5	25.6	23.1	Weak
BH 20-10 – Run 3	8.8 – 8.9	26.8	73.3	Strong
BH 20-10 – Run 6	13.7 – 13.9	26.1	78.0	Strong
BH 20-13 – Run 3	10.8 – 10.9	27.4	26.5	Medium Strong
BH 20-13 – Run 6	16.1 – 16.2	25.8	20.5	Weak
BH 20-13 – Run 7	16.7 – 16.9	25.9	31.2	Medium Strong

A review of the test results in Table VII indicates the strength of the rock may be classified as weak to strong in accordance with the Canadian Foundation Engineering Manual (CFEM), Fourth Edition, 2006.

As previously mentioned, the site is underlain by shale bedrock of the Carlsbad formation. This type of shale is prone to deterioration when exposed to the elements. It also heaves due to a complex mechanism caused in part from the bio-oxidation of the sulphides in the rock, which react with calcite seams to form expanding gypsum. This occurs when oxygen is permitted to enter the rock, usually by lowering of the water table and this process is accelerated by the presence of heat. Therefore, special treatment of the Carlsbad shale bedrock will need to be incorporated into the design and construction of the proposed buildings.

5.8 Groundwater Level Measurements

A summary of the groundwater level measurements taken in the monitoring wells and boreholes is shown in Table VIII.

Table VIII: Summary of Groundwater level Measurements			
Borehole (BH) /Monitoring Well (MW) No.	Ground Surface Elevation (m)	Date of Measurement (Elapsed Time in Days from Date of Installation)	Groundwater Depth Below Ground Surface (Elevation), m
MW 19-01	70.40	September 3, 2020 (289 days)	1.5 (68.9)
MW 19-02	70.29	September 3, 2020 (289 days)	2.1 (68.2)
MW 19-03	70.30	September 3, 2020 (288 days)	2.4 (67.9)
MW 19-04	70.00	September 3, 2020 (288 days)	1.2 (68.8)

Table VIII: Summary of Groundwater level Measurements

Borehole (BH) /Monitoring Well (MW) No.	Ground Surface Elevation (m)	Date of Measurement (Elapsed Time in Days from Date of Installation)	Groundwater Depth Below Ground Surface (Elevation), m
BH 20-05 (shallow standpipe installed to 7.6 m depth; slotted section within soil)	72.37	September 3, 2020 (13 days)	3.1 (69.3)
		September 10, 2020 (20 days)	3.2 (69.2)
BH 20-05 (deep standpipe installed to bottom of borehole at 17.6 m depth with slotted section within bedrock)	72.37	September 3, 2020 (20 days)	3.0 (69.4)
		September 10, 2020 (20 days)	3.0 (69.4)
BH 20-10	69.86	September 3, 2020 (10 days)	1.8 (68.1)
BH 20-11	70.68	September 3, 2020 (9 days)	1.7 (69.0)
BH 20-13	71.07	September 3, 2020 (13 days)	2.6 (68.5)
	71.07	September 10, 2020 (20 days)	2.6 (68.5)
BH 20-14	71.98	September 3, 2020 (9 days)	1.7 (70.3)

The groundwater level ranges from 1.2 m to 3.2 m (Elevation 70.3 m to Elevation 67.9 m).

Water levels were determined in the boreholes and 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 borehole information indicates that the subsurface conditions at the site consist of a surficial pavement structure and fill underlain by sand, silt and clay layers, silty clay, glacial till and shale bedrock contacted at 6.0 m to 7.7 m (Elevation 64.7 m to Elevation 62.8 m) below existing grade. The groundwater level ranges from 1.2 m to 3.2 m depths (Elevation 70.3 m to Elevation 67.9 m).

Based on a review of the available design information and borehole data, it is considered that the appropriate foundation to support the proposed buildings is spread and strip footings founded on the sound shale bedrock. The anticipated founding depth of the footing is assumed to be approximately at a 9.5 m depth, which is 0.5 m below the assumed 9.0 m depth of the lowest floor slab.

The results of the shear wave velocity sounding survey (seismic shear wave survey) conducted by GPR is shown in Appendix C. Based on the results of the survey and the recommendation that the proposed buildings be supported by footings founded on the sound bedrock, the shear wave velocity for footings founded on sound bedrock was determined to be 1506.2 m/s. For a shear wave velocity of 1506.2 m/s, Table 4.1.8.4.A of the 2012 Ontario Building Code (as amended May 2, 2019) indicates the site classification for seismic site response is Class A. Therefore, for footings founded on the sound bedrock, the site classification for seismic site response is Class A.

6.2 Liquefaction Potential of Soils

It is anticipated that all subsurface soils on site including the fill and native soils will be excavated down to the bedrock and removed from site for the construction of the proposed new buildings. Since all subsurface soils will be excavated and removed from the site, liquefaction of the soils on site during a seismic event is not a concern.

7. Grade Raise Restrictions

From a geotechnical perspective there are no restrictions to raising the grades at the site since it is anticipated that all subsurface soils will be excavated down to the bedrock, removed from the site and replaced with imported granular fill compacted to the specified degree of compaction indicated in this report.

8. Foundation Considerations

Based on a review of the available design information and borehole data, it is considered that the appropriate foundation to support the proposed buildings is spread and strip footings founded on the sound shale bedrock. The anticipated founding depth of the footing is assumed to be approximately at a 9.5 m depth, which is 0.5 m below the assumed 9.0 m depth of the lowest floor slab.

Footings founded on the sound bedrock, competent and free of weathered zones, loose material, clay seams, fractures and voids may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) 2.0 MPa. The estimated depth to sound bedrock at the borehole locations where the bedrock was cored is summarized in Table IX.

Table IX: Summary of Estimated Depth (Elevation) of Sound Bedrock in Boreholes

Borehole (BH) No./Monitoring Well (MW) No.	Ground Surface Elevation (m)	Depth (Elevation) of Bedrock Surface (m)	Estimated Depth (Elevation) to Sound Bedrock	Estimated Thickness of Weathered Bedrock (m)
MW 19-01	70.40	7.0 (63.4)	7.6 (62.8)	0.6
BH 20-05	72.37	6.9 (65.5)	7.9 (64.5)	1.0
BH 20-10	69.86	6.0 (63.9)	7.2 (62.7)	1.2
BH 20-13	71.07	7.0 (64.1)	14.7 (56.4)	7.7

Based on a review of Table IX, the shale bedrock in Borehole No. 20-13 from its surface at a 7.0 m depth (Elevation 64.1 m) to a 14.7 m depth (Elevation 56.4 m) is of a very poor quality and is not considered capable of supporting footings designed for the factored ULS value of 2.0 MPa. It is not known whether there is a fault in this area or this is a localized condition. Therefore, all broken bedrock in the vicinity of this borehole will have to be sub-excavated below the anticipated design footing founding depth of approximately 9.5 m to competent bedrock expected at 14.7 m depth (Elevation 56.4 m) and the excavation backfilled using concrete (with a compressive strength of 25 MPa) to the design founding elevation of the footing and the footing cast on the concrete backfill. The lateral extent of the area requiring this treatment will be best established in the field during construction and should be allowed for as part of the construction budget.

The Serviceability Limit State (SLS) bearing pressure of the sound bedrock, required to produce 25 mm settlement of the footing will be much larger than the recommended value for factored geotechnical resistance at ULS. Therefore, the factored geotechnical resistance at ULS will govern the design.

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

As indicated in Section 5.7 of this report, the shale bedrock of the Carlsbad formation is prone to swelling under certain conditions of heat and humidity. It is also prone to rapid deterioration especially for the portion of the shale bedrock below the groundwater table that is exposed to the elements. Therefore, the base and sides of the exposed shale bedrock in all footing excavations should be cleaned of any soil or deleterious material, examined by a geotechnical engineer and the approved shale subgrade covered with a skim coat of concrete within the same day of its first exposure. Alternatively, the surface of the shale bedrock may be kept wet at all times. For reasons given previously, the concrete for the footings should be poured flush with the rock surfaces (base and sides of the footing trench within the bedrock). Alternatively, the shale bedrock exposed in the sides of the footing trenches may be sealed by spraying gunnite.

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

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

8.1 Uplift Resistance for Footings

Rock anchors may be required for the footings to resist uplift forces. The following criteria is recommended for the design of the rock anchors:

Grouted and pre-stressed anchors may fail in one or more of the following manners:

a) Failure of the grout/tendon bond;

b) Failure of the steel tendon or top anchorage;

c) Failure of the rock/grout bond; or,

d) Failure of the rock mass.

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

Failure of the Rock/Grout Bond:

- In limit states design, the unfactored ultimate limit state (ULS) bond stress of the sound rock/grout interface may be taken as 1000 kPa (1.0 MPa). For semi-empirical analysis, the factored ULS bond stress is 300 kPa using a geotechnical resistance factor of 0.3. Alternatively, a factored ULS bond strength of 400 kPa (includes a geotechnical resistance factor of 0.4) may be used provided proof tests are conducted on all rock anchors. The unconfined compressive strength of the grout is assumed to be 35 MPa.
- The unbonded length of the anchor should be equal to the height of the theoretical rock cone and less than half the bonded length. The weathered zone of the bedrock identified in Table IX should be included within the unbonded length of the anchor when calculating the anchor capacity.
- The bonded length of an anchor is taken typically at a minimum of 3.0 m.
- Neither the spacing nor the stagger length between anchors should be less than 0.5 times the fixed anchor length. If the spacing of less than 0.5 times the fixed anchor length is unavoidable, the fixed anchor length should be increased in proportion to the ratio of 0.5 times the fixed anchor length to the spacing or stagger length of the anchors.

Failure of the Rock Mass:

- The embedment depth of the anchors should be designed using the submerged weight of a geometric rock cone around the anchor. The submerged weight of the rock cone should not be less than the ultimate capacity of the anchor. The rock cone should be assumed to have an apex angle of 60 degrees and the apex of the cone should be assumed at the midpoint of the bonded length of the anchor. Where the embedment cones of adjacent anchors overlap, the combined embedment cones for a group of anchors should be used to determine the anchor group resistance to rock mass failure.
- The submerged unit weight of the bedrock equal to 16.2 kN/m^3 should be used in the calculations.

Corrosion Protection of the Anchors:

- Corrosion protection of the anchors should be in accordance with the Ontario Provincial Standard Specification (OPSS) 942.

Testing of Rock Anchors:

Pre-production or design performance tests of permanent rock anchors should be in accordance with OPSS 942. Pre-production performance tests should be conducted on selected rock anchors.

Proof load tests should be conducted on all anchors and should be in accordance with OPSS 942.

Installation of the Rock Anchors:

- The boreholes encountered highly fractured bedrock with RQD values less than 50 percent and loss of wash water during coring of the bedrock. Therefore, the contractor should anticipate that significant grout takes may occur during grouting operations.
- The contractor should undertake grouting operations in such a manner as to prevent damage to existing nearby structures.

9. Floor Slab and Drainage Requirements

The lowest floor level of the parking garages for the proposed buildings will be located at an approximate 9.0 m depth below the ground floor slab which is assumed to be near the existing grade. Based on the borehole information, the lowest floor slabs of the buildings will be founded on the shale 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 garages is anticipated to be located below the groundwater level. Therefore, underfloor and perimeter drainage systems will be required for the proposed below grade parking garages.

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

For floor slabs (concrete or asphalt) founded on the shale bedrock, special procedures will be required during slab construction. The shale bedrock of the Carlsbad formation is known to heave due to a complex mechanism caused in part by the bio-oxidation of sulphides in the rock which then react with the calcite seams to form expanding gypsum. This occurs when oxygen is permitted to enter the rock, usually by lowering the water table. Cracking of the floor slab due to heaving of the shale has occurred in some structures in the Ottawa area. It is therefore recommended that the water table at the site should be maintained above the shale surface. The invert of the underfloor drains should be set at least 150 mm above the shale bedrock surface. In addition, a mud (skim) coat of concrete at least 50 mm thick should be placed on the surface of the shale as a seal prior to placement of the granular fill. Weep holes should be provided in the skim concrete layer to facilitate drainage. Any granular fill to be placed under the floor slab should be compacted to at least 98 percent of the SPMDD. Any elevator pits and sumps should be constructed as water- tight structures instead of trying to locally depress the groundwater table around them which may result in dewatering of the shale.

The finished exterior grade 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 shale bedrock. The exposed shale bedrock should be examined by a geotechnical engineer and any loose/soft zones of the bedrock should be excavated and removed. Upon approval, the bedrock subgrade should be prepared as noted above.

Following approval and preparation 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 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 lower floor level has been determined.

9.2 Lowest Floor Level as a Paved Surface

The subgrade is anticipated to consist of shale bedrock. The exposed shale bedrock should be examined by a geotechnical engineer and any loose/soft zones of the bedrock should be excavated and removed. Upon approval, the bedrock subgrade should be prepared as noted above.

Following approval and preparation 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 Basement Walls

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

The equations given below assume that the backfill against the subsurface walls will be free-draining granular material and that subsurface drains will be provided to prevent build-up of hydrostatic pressure behind the wall. Equation (i) will be applicable to the portion of the subsurface wall in the overburden (soil). Equation (ii) will be applicable to the portion of the subsurface wall in the bedrock where the earth pressure will be considerably reduced due to the narrow backfill between the subsurface wall and the rock face resulting in an arching effect (Spangler & Handy, 1984).

Lateral static earth pressure, p:

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

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) due to narrow earth backfill between subsurface wall and rock face at depth z:

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

where

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

B = backfill width (m)

z = depth from top of wall (m)

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

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

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

The lateral dynamic 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 (m)

γ = unit weight of soil = 22 kN/m^3

$\frac{a_h}{g}$ = seismic coefficient = 0.32

F_b = thrust factor = 1.0

The dynamic thrust acts approximately at 0.63H above the base of the wall.

All subsurface walls should be waterproofed.

11. Excavations and De-Watering Requirements

11.1 Excess Soil Management

Ontario Regulation 406/19 made under the Environmental Protection Act (November 28, 2019) has been implemented on January 1, 2021. The new regulation dictates the testing protocol that will be required for the management and disposal of excess soils. As set forth in the regulation, specific analytical testing protocols will need to be implemented and followed based on the volume of soil to be managed. 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

Excavation of the soils may be undertaken using heavy equipment capable of removing debris within the fill as well as cobbles, boulders and slabs of shale bedrock within the glacial till.

All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils on site are considered to be Type 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 in the soils, the excavation side slopes are expected to slough and eventually stabilize at a slope of 2H:1V to 3H:1V.

It is anticipated that due to the significant depth of the excavation for the proposed buildings and the proximity of the excavation to existing buildings and infrastructure, the excavations will likely have to be undertaken within the confines of a shoring system. The shoring system may consist of steel H soldier pile and timber lagging system, interlocking sheeting system and/or secant pile shoring system.

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

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

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

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

Soldier Pile and Timber Lagging System

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

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

where
P = the pressure, at any depth, h, below the ground surface
k = applicable earth pressure coefficient; active lateral earth pressure coefficient = 0.33
'at rest' lateral earth pressure coefficient = 0.50
γ = unit weight of soil to be retained, estimated at 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 system

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

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

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 exposed shale bedrock surface along all excavation walls should be shotcreted within the same day of exposure to protect the rock face from rapid deterioration due to exposure to the elements, as previously discussed.

The shoring system will require lateral restraint by tiebacks in the form of grouted rock anchors designed as per Section 8.1 in this report.

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.

11.2.2 Rock Excavation

The excavations are anticipated to extend below the bedrock surface. In this case, the shale 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 sideslopes in the weathered and shale bedrock may be undertaken near vertical but may need be cut back at a 1H:1V gradient in zones of loose rock pieces/slabs.

The excavation of the shale bedrock to extensive depths below the bedrock surface will likely require line drilling and blasting techniques. Contractors bidding on this project should decide on their own the most preferred rock removal method; hoe ramming or line drilling and blasting.

The exposed shale bedrock surface along all excavation walls should be shotcreted within the same day of exposure to protect the rock face from rapid deterioration due to exposure to the elements, as previously discussed.

Rock Support

Excavations within the weathered bedrock may be undertaken with near vertical sides subject to review by a geotechnical engineer. The weathered and fractured rock face 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 the shotcrete mentioned above. Excavations that will extend a significant depth 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.

Vibration Control

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

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

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

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

It is recommended that a hydrogeological study (with a geotechnical component) be undertaken for the purpose of estimating the volume of groundwater anticipated to enter the unshored (worst case) and shored excavation (which permits drainage) and the zone of influence resulting from dewatering of the excavation. The zone of influence may be used to determine the impact, if any, dewatering of the excavation may have on nearby existing infrastructure and buildings. If it is determined that the zone of influence extends to nearby existing infrastructure and buildings, the geotechnical component of the hydrogeological study would involve estimating settlements of the nearby existing infrastructure and buildings as a result of lowering the groundwater table at the site and providing recommendations to minimize the estimated settlements.

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

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

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

12. Pipe Bedding Requirements

For site servicing, it is anticipated that the subgrade for the proposed municipal services may consist of fill, sand, silty clay, glacial till and shale bedrock.

The pipe bedding for the municipal services should be in accordance with City of Ottawa specifications, drawings and special provisions. The bedding and cover material should be compacted to a minimum of 95 percent standard Proctor maximum dry density (SPMDD).

The bedding thickness may be increased in areas where the subgrade is subject to disturbance. Trench base stabilization techniques, such as the removal of loose material, placement of sub-bedding, consisting of Ontario Provincial Standard Specification (OPSS) 1010 Granular B Type II completely wrapped in a non-woven geotextile, may be used if trench base disturbance becomes a problem in wet or soft/loose areas.

Depending on-site condition, removal of some of the existing soil may be required and replaced with OPSS 1010 Granular A material for pipe bedding construction. This requirement would have to be established in the field by the geotechnical personnel. In areas where the fill subgrade consists of weathered shale bedrock with slabs or pieces of rock that may contain voids, it is recommended the voids be filled OPSS Granular A material and the surface of the filled in rock be covered with a separation membrane, such as Terrafix 270R or equivalent prior to the placement of the pipe bedding material.

To minimize the potential for bending stresses within the pipe, a transition zone treatment should be provided in areas where the pipe subgrade changes from overburden to bedrock and vice versa. In areas where the surface of the bedrock slopes at a steeper gradient than 3H:1V, the bedrock should be excavated and additional bedding material placed to create a 3H:1V transition zone.

If the subgrade for the underground service pipes will be the expansive shale bedrock, special procedures for the installation of the underground services, as previously discussed for footing and slab-on-grade construction on the shale bedrock, may be required. EXP can provide additional comments and recommendations regarding the installation of the service pipes on the shale bedrock subgrade, once pipe invert elevations are known.

The municipal services should be installed in short open trench sections that are excavated and backfilled the same day.

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

The soils to be excavated from the site will comprise of fill, silty sand, silty clay, glacial till and shale bedrock. From a geotechnical perspective, these soils and the shale bedrock are not considered suitable for reuse as backfill material in the interior or exterior of the building. It may be possible to use portions of the soils above the groundwater level outside the building area in landscaped areas, provided all debris, cobbles and boulders are removed from the material prior to placement. However, these soils are subject to moisture absorption due to precipitation and must be protected at all time from the elements.

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

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

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

14. Access Roads and Parking Areas

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

Table X: Recommended Pavement Structure Thicknesses

Pavement Layer	**Compaction Requirements**	**Light Duty Parking Areas**	**Heavy Duty Parking Areas and Access Roads**
Asphaltic Concrete (PG 58-34)	92% to 97 % MRD	65 mm – SP12.5 Cat B or HL3	40 mm – 12.5 Cat B/HL3 50 mm – 19 Cat B/HL8
Granular A Base (OPSS 1010) (crushed limestone)	100% SPMDD	150 mm	150 mm
Granular B Sub-base, Type II (OPSS 1010)	100% SPMDD	300 mm	450 mm
SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698-12e2 MRD denotes Maximum Relative Density, ASTM D2041			

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

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

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

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

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

(4) The 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, 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 the subgrade level. These areas may require the use of thicker/coarser sub-base material and 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) for Granular A and Granular B Type II and should be compacted to 100 percent SPMDD. The asphaltic concrete used and its placement should meet OPSS 1151 requirements. It should be placed and compacted in accordance with OPSS 311 and 313.

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

15. Corrosion Potential

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

Table XI: pH, Chloride, Sulphate and Resistivity Test Results on Bedrock Cores

Borehole – Run No.	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
BH 5 – Run 3	10.9 – 11.1	8.42	0.0055	0.0025	2640
BH 10 – Run 4	10.4 – 10.5	9.41	0.0011	0.0029	2380
BH 13 – Run 6	15.8 – 15.9	9.36	0.0030	0.0020	1740

The results indicate the shale bedrock has a negligible sulphate attack on subsurface concrete. The concrete should be in accordance with CSA A.23.1-14.

The results of the resistivity tests indicate the shale bedrock is mildly corrosive to corrosive to bare steel as per the National Association of Corrosion Engineers (NACE). Appropriate measures should be undertaken to protect buried steel elements from corrosion.

16. Tree Planting Restrictions

It is anticipated that all subsurface soils on site including the fill and native soils will be excavated down to the bedrock and removed from site for the construction of the proposed new buildings. Since all subsurface soils will be excavated and removed from the site, there are no tree planting restrictions from a geotechnical perspective.

17. Additional Studies

The following additional study is recommended for the proposed development:

- Hydrogeological Study (with geotechnical component) – To estimate the volume of groundwater anticipated flowing into an unshored (worst case) and shored excavation (which permits drainage) and estimate the zone of influence resulting from the groundwater lowering. If the zone of influence from groundwater lowering extends to existing structures and infrastructure, the geotechnical component of the hydrogeological study would involve estimating the settlement of the existing structures and infrastructure and providing recommendations to minimize the estimated settlements.
- Consideration should be given to conducting an additional geotechnical investigation consisting of boreholes to better delineate the lateral extent of the very poor quality of bedrock identified in Borehole No. 20-13.

18. Earthworks Quality Control During Construction

All earthworks activities from construction of footing foundations to subgrade preparation to the placement and compaction of fill soils should be inspected by geotechnical personnel to ensure that construction proceeds in accordance with the project specifications.

19. General Comments

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

The information contained in this report is not intended to reflect on environmental aspects of the soils. Should specific information be required, including for example, the presence of pollutants, contaminants or other hazards in the soil, additional testing may be required.

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

LICENSED PROFESSIONAL ENGINEER
FEB. 2, 2021
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37232568
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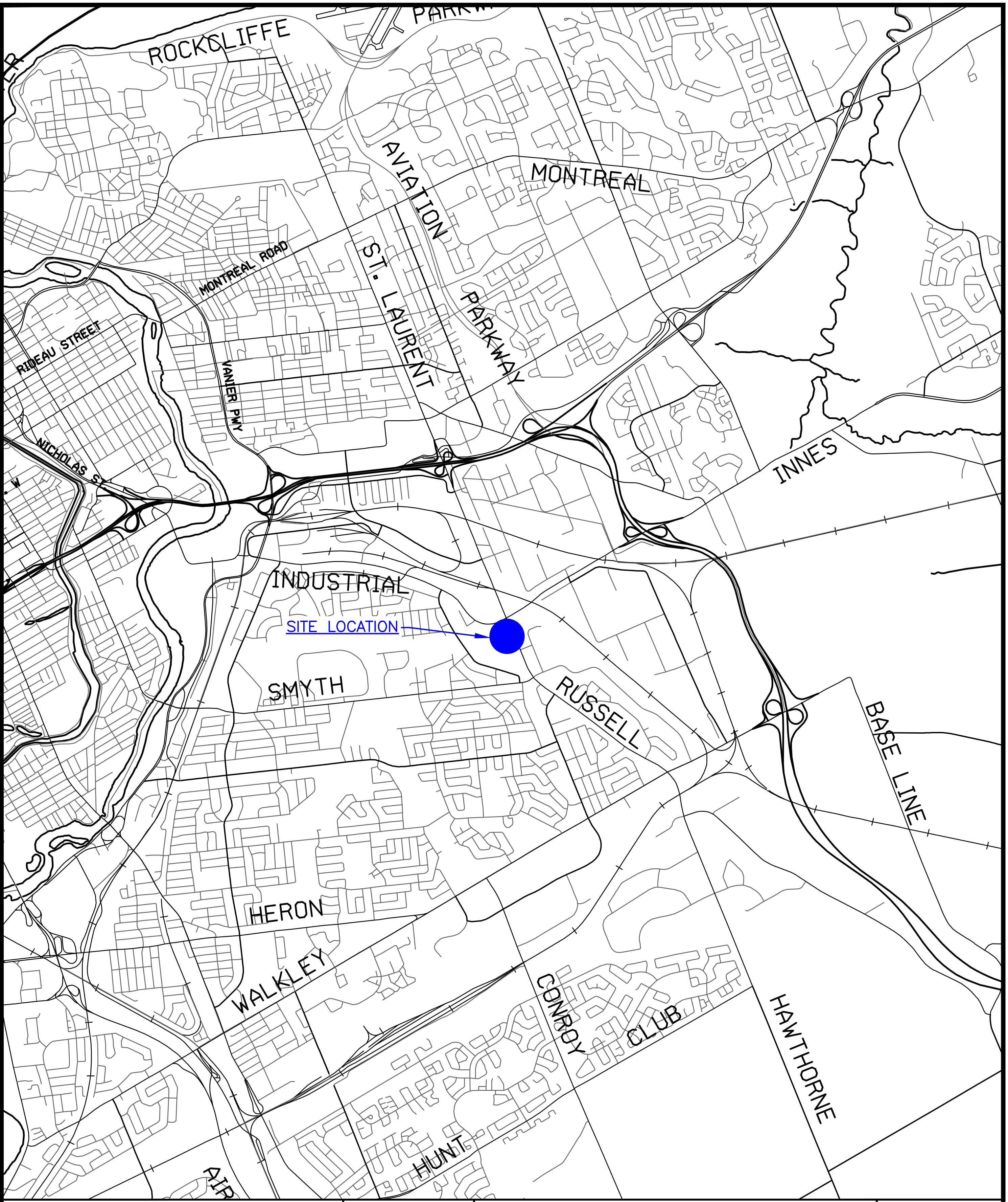
Figures



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DESIGN	EXP
DRAWN	JH
DATE	AUG 2020
FILE NO	260579-A0

1740–1760 ST. LAURENT BOULEVARD
GEOTECHNICAL INVESTIGATION

SITE LOCATION PLAN

SCALE
1:50,000

SKETCH NO

FIG 1

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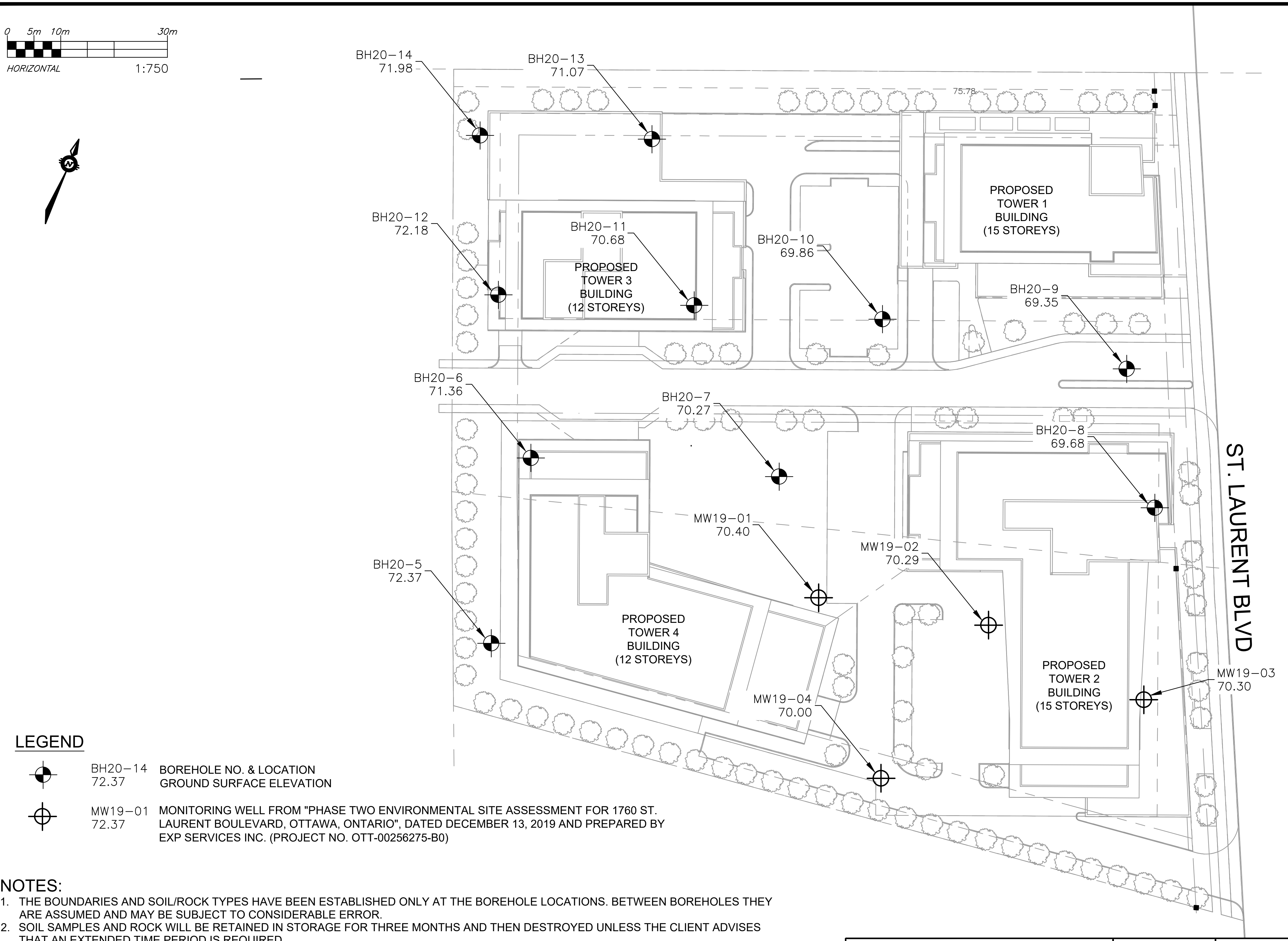

LEGEND

◑	BH20−14 72.37	BOREHOLE NO. & LOCATION GROUND SURFACE ELEVATION
⊕	MW19−01 72.37	MONITORING WELL FROM "PHASE TWO ENVIRONMENTAL SITE ASSESSMENT FOR 1760 ST. LAURENT BOULEVARD, OTTAWA, ONTARIO", DATED DECEMBER 13, 2019 AND PREPARED BY EXP SERVICES INC. (PROJECT NO. OTT-00256275-B0)

NOTES:

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

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exp.

DESIGN EXP
DRAWN JH
DATE AUG 2020
FILE NO 260579−A0

1740−1760 ST. LAURENT BOULEVARD
GEOTECHNICAL INVESTIGATION

BOREHOLE LOCATION PLAN

SCALE 1:750
SKETCH NO
FIG 2

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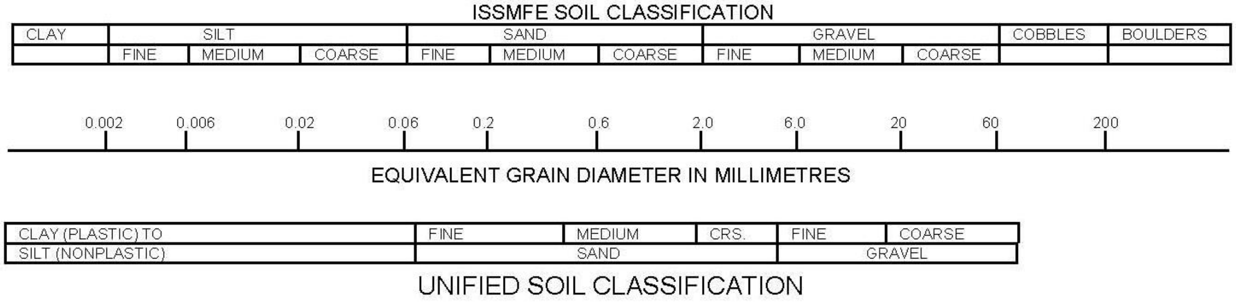
Notes On Sample Descriptions

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



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 MW19-01

Project No: OTT-000256275-B0

Project: Phase II Environmental Site Assessment

Location: 1760 St. Laurent Boulevard, Ottawa

Figure No. 3

Page. 1 of 2

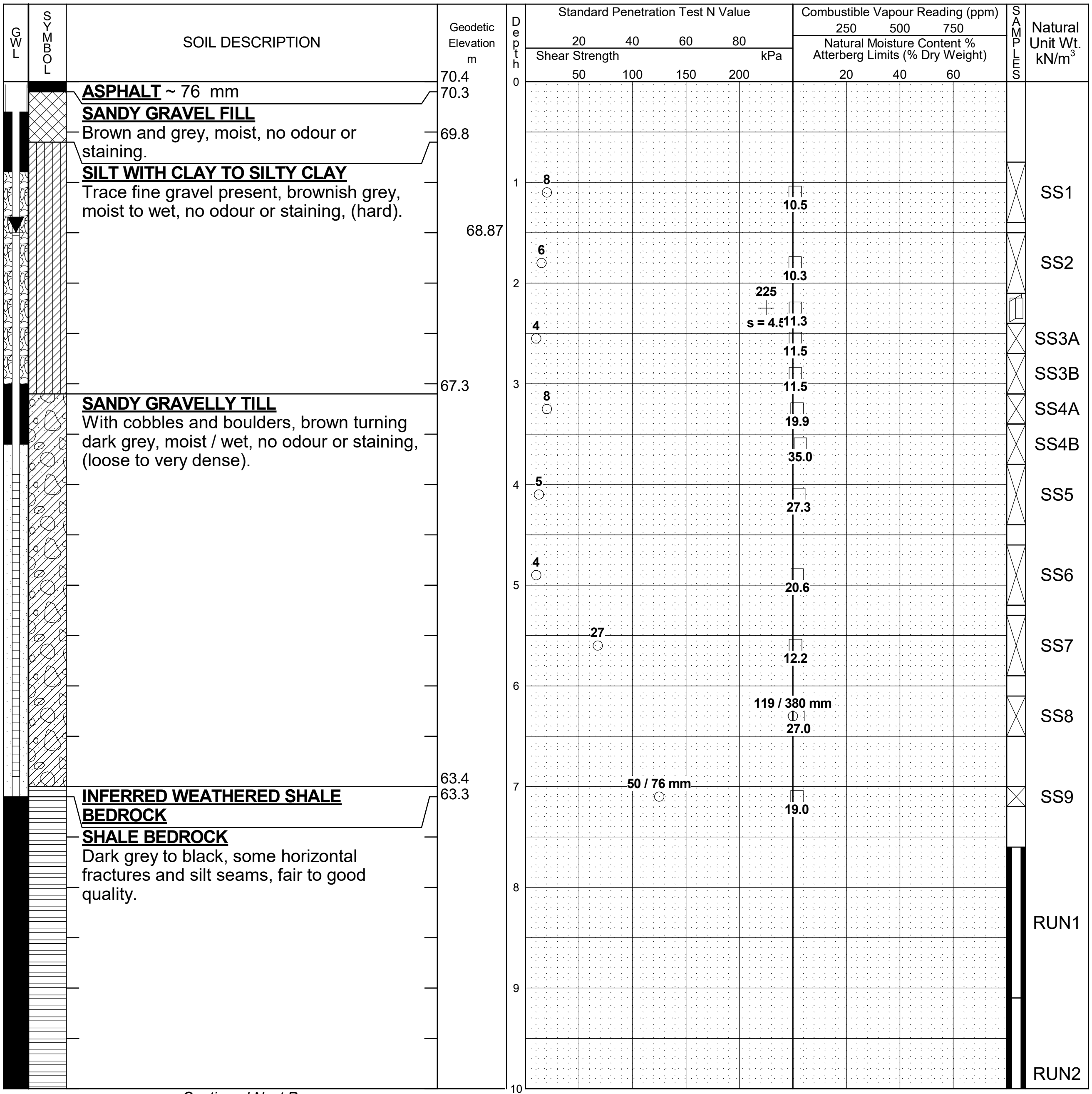
Date Drilled: November 18th, 2019

Drill Type: CME 55 Rubber Track

Datum: Geodetic Elevation

Logged by: MD   Checked by: PS

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
70.4 70.3	**ASPHALT** ~ 76 mm
69.8	**SANDY GRAVEL FILL** Brown and grey, moist, no odour or staining.
68.87 67.3	**SILT WITH CLAY TO SILTY CLAY** Trace fine gravel present, brownish grey, moist to wet, no odour or staining, (hard).
63.4	**SANDY GRAVELLY TILL** With cobbles and boulders, brown turning dark grey, moist / wet, no odour or staining, (loose to very dense).
63.3	**INFERRED WEATHERED SHALE BEDROCK**
	SHALE BEDROCK Dark grey to black, some horizontal fractures and silt seams, fair to good quality.

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A flushmount monitoring well with a 51 mm slotted standpipe was installed in the borehole 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-000256275-B0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Nov. 22, 2019	1.7	-
Nov. 25, 2019	2.1	-
Sept. 3, 2020	1.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.57 - 9.07	100	71
2	9.07 - 10.59	100	84

Log of Borehole MW19-01

Project No: OTT-000256275-B0

Figure No. 3

Project: Phase II Environmental Site Assessment

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
			60.4	10				
			59.8					
		End of Borehole at 10.59 m Depth, Borehole Terminated						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A flushmount monitoring well with a 51 mm slotted standpipe was installed in the borehole 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-000256275-B0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Nov. 22, 2019	1.7	-
Nov. 25, 2019	2.1	-
Sept. 3, 2020	1.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.57 - 9.07	100	71
2	9.07 - 10.59	100	84

Log of Borehole MW19-02

Project No: OTT-000256275-B0

Project: Phase II Environmental Site Assessment

Location: 1760 St. Laurent Boulevard, Ottawa

Figure No. 4

Page. 1 of 1

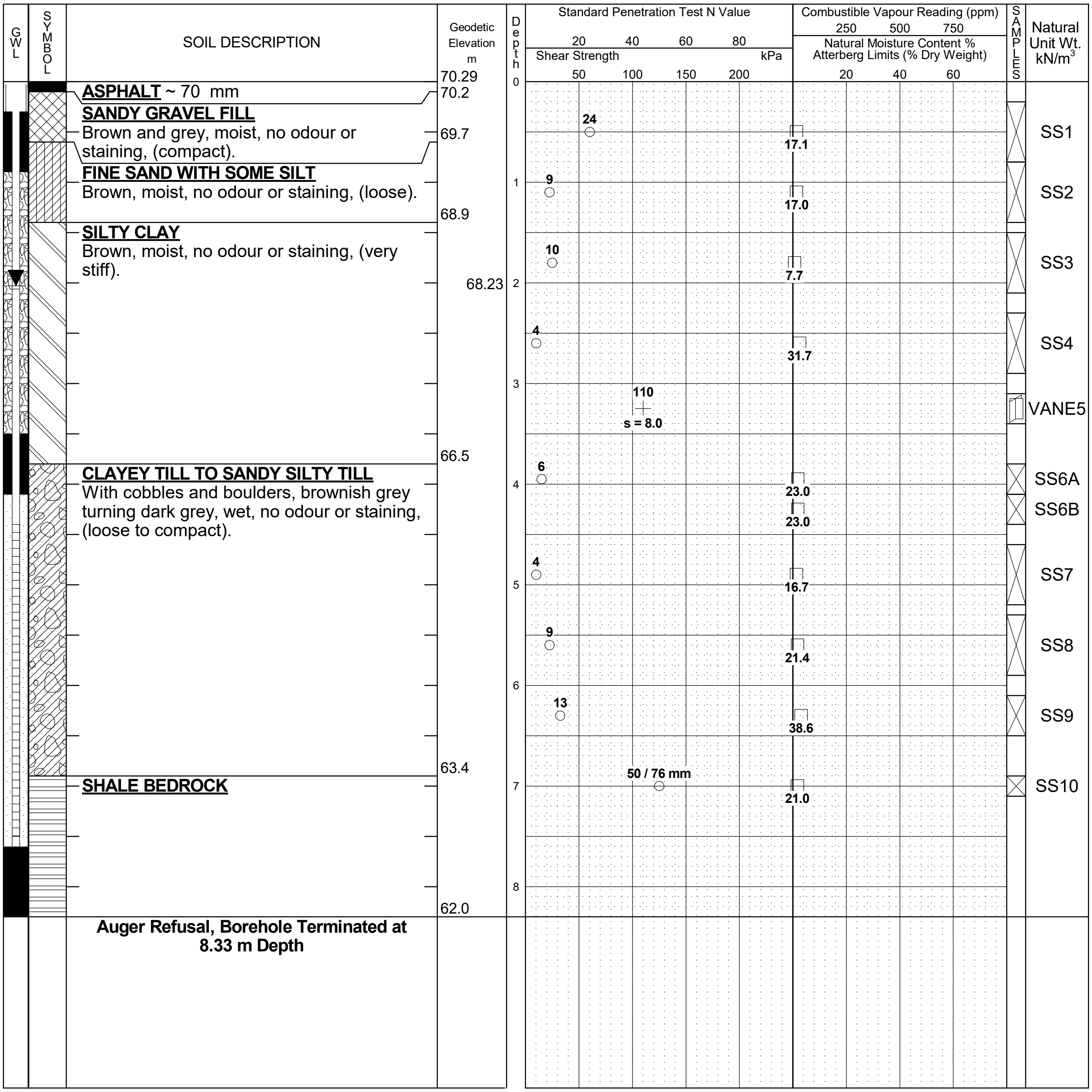
Date Drilled: November 18th, 2019

Drill Type: CME 55 Rubber Track

Datum: Geodetic Elevation

Logged by: MD   Checked by: PS

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 ~ 70 mm

SANDY GRAVEL FILL
Brown and grey, moist, no odour or staining, (compact).

FINE SAND WITH SOME SILT
Brown, moist, no odour or staining, (loose).

SILTY CLAY
Brown, moist, no odour or staining, (very stiff).

CLAYEY TILL TO SANDY SILTY TILL
With cobbles and boulders, brownish grey turning dark grey, wet, no odour or staining, (loose to compact).

SHALE BEDROCK

Auger Refusal, Borehole Terminated at 8.33 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A flushmount monitoring well with a 51 mm slotted standpipe was installed in the borehole 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-000256275-B0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Nov. 22, 2019	2.2	-
Nov. 25, 2019	2.3	-
Sept. 3, 2020	2.1	

CORE DRILLING RECORD

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

Log of Borehole MW19-03

Project No: OTT-000256275-B0

Project: Phase II Environmental Site Assessment

Location: 1760 St. Laurent Boulevard, Ottawa

Figure No. 5

Page. 1 of 1

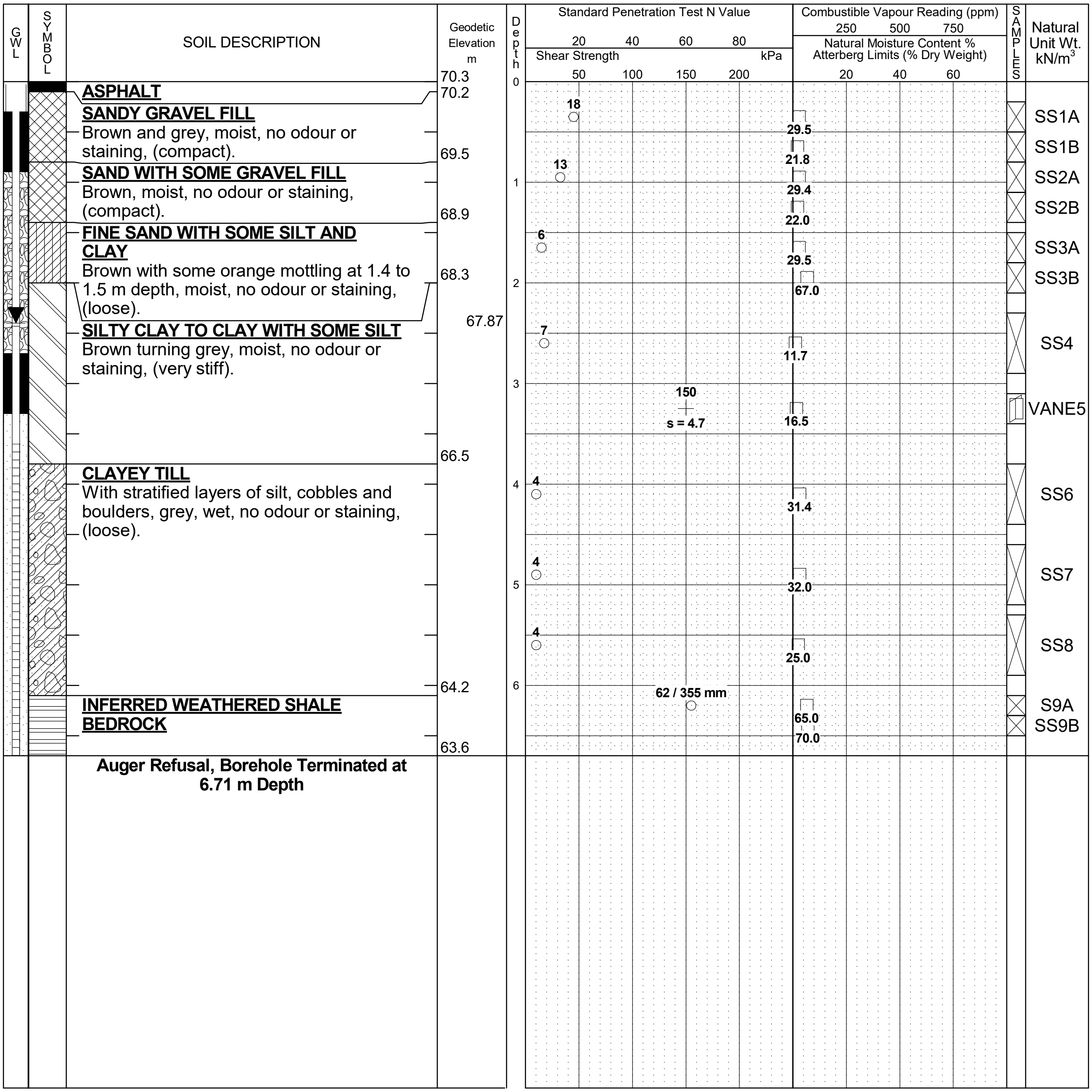
Date Drilled: November 19th, 2019

Drill Type: CME 55 Rubber Track

Datum: Geodetic Elevation

Logged by: MD Checked by: PS

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	▲



Geodetic Elevation m	SOIL DESCRIPTION
70.3	
70.2	**ASPHALT**
	SANDY GRAVEL FILL Brown and grey, moist, no odour or staining, (compact).
69.5	**SAND WITH SOME GRAVEL FILL** Brown, moist, no odour or staining, (compact).
68.9	**FINE SAND WITH SOME SILT AND CLAY** Brown with some orange mottling at 1.4 to 1.5 m depth, moist, no odour or staining, (loose).
68.3	**SILTY CLAY TO CLAY WITH SOME SILT** Brown turning grey, moist, no odour or staining, (very stiff).
67.87	
66.5	**CLAYEY TILL** With stratified layers of silt, cobbles and boulders, grey, wet, no odour or staining, (loose).
64.2	**INFERRED WEATHERED SHALE BEDROCK**
63.6	

Auger Refusal, Borehole Terminated at 6.71 m Depth

SAMPLES	SPT N Value / Shear Strength	Natural Moisture Content %
SS1A	18	29.5
SS1B		21.8
SS2A	13	29.4
SS2B		22.0
SS3A	6	29.5
SS3B		67.0
SS4	7	11.7
VANE5	150 s = 4.7	16.5
SS6	4	31.4
SS7	4	32.0
SS8	4	25.0
S9A	62 / 355 mm	65.0
SS9B		70.0

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A flushmount monitoring well with a 51 mm slotted standpipe was installed in the borehole 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-000256275-B0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Nov. 22, 2019	2.6	-
Nov. 25, 2019	2.7	-
Sept. 3, 2020	2.4	

CORE DRILLING RECORD

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

Log of Borehole MW19-04

Project No: OTT-000256275-B0

Figure No. 6

Project: Phase II Environmental Site Assessment

Page. 1 of 1

Location: 1760 St. Laurent Boulevard, Ottawa

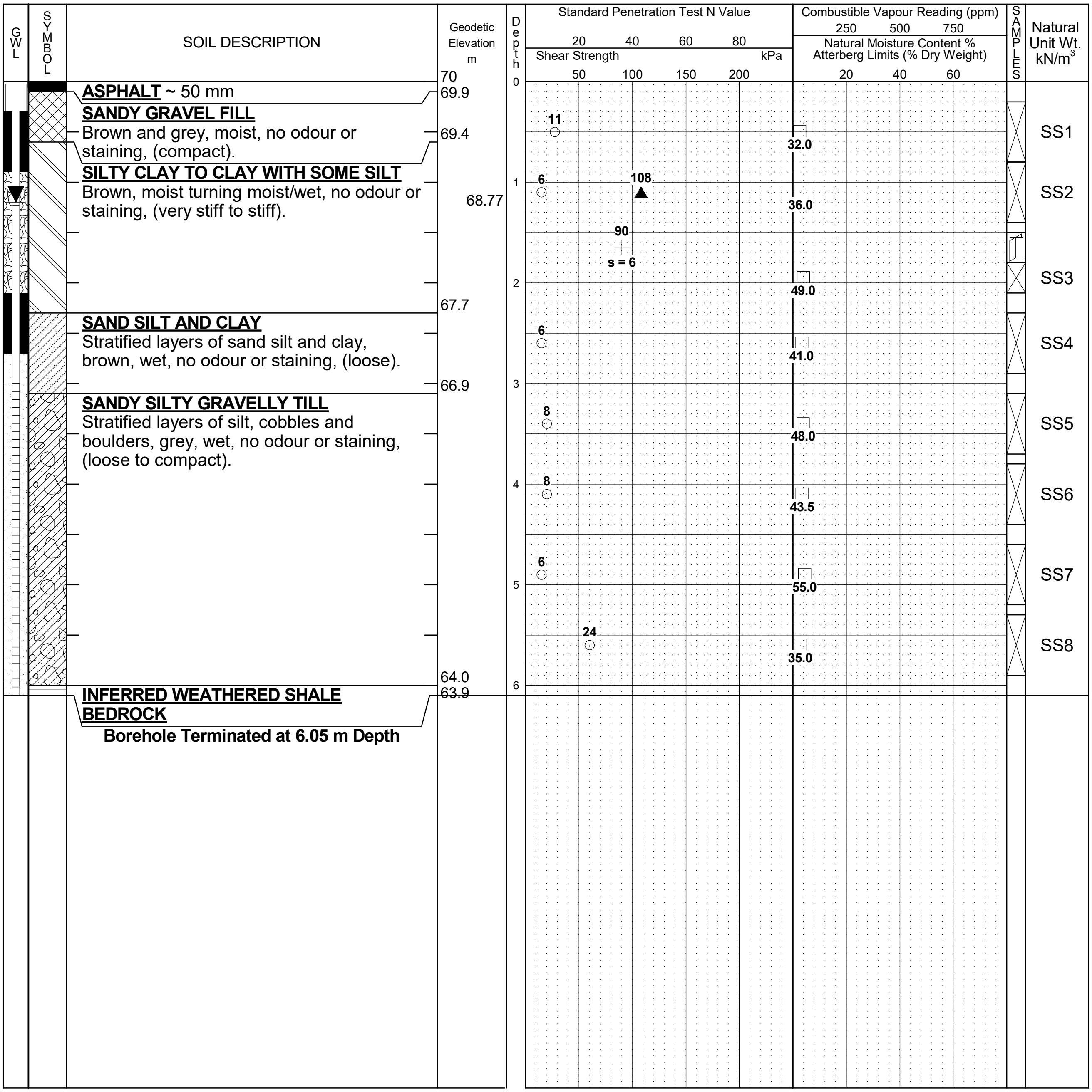
Date Drilled: November 19th, 2019

Drill Type: CME 55 Rubber Track

Datum: Geodetic Elevation

Logged by: MD Checked by: PS

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 ~ 50 mm — 69.9

SANDY GRAVEL FILL
Brown and grey, moist, no odour or staining, (compact). — 69.4

SILTY CLAY TO CLAY WITH SOME SILT
Brown, moist turning moist/wet, no odour or staining, (very stiff to stiff). — 68.77 — 67.7

SAND SILT AND CLAY
Stratified layers of sand silt and clay, brown, wet, no odour or staining, (loose). — 66.9

SANDY SILTY GRAVELLY TILL
Stratified layers of silt, cobbles and boulders, grey, wet, no odour or staining, (loose to compact). — 64.0

INFERRED WEATHERED SHALE BEDROCK — 63.9

Borehole Terminated at 6.05 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A flushmount monitoring well with a 51 mm slotted standpipe was installed in the borehole 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-000256275-B0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Nov. 22,2019	1.5	-
Nov. 25, 2019	2.1	-
Sept. 3, 2020	1.2	

CORE DRILLING RECORD

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

Log of Borehole BH20-05

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 7

Page. 1 of 2

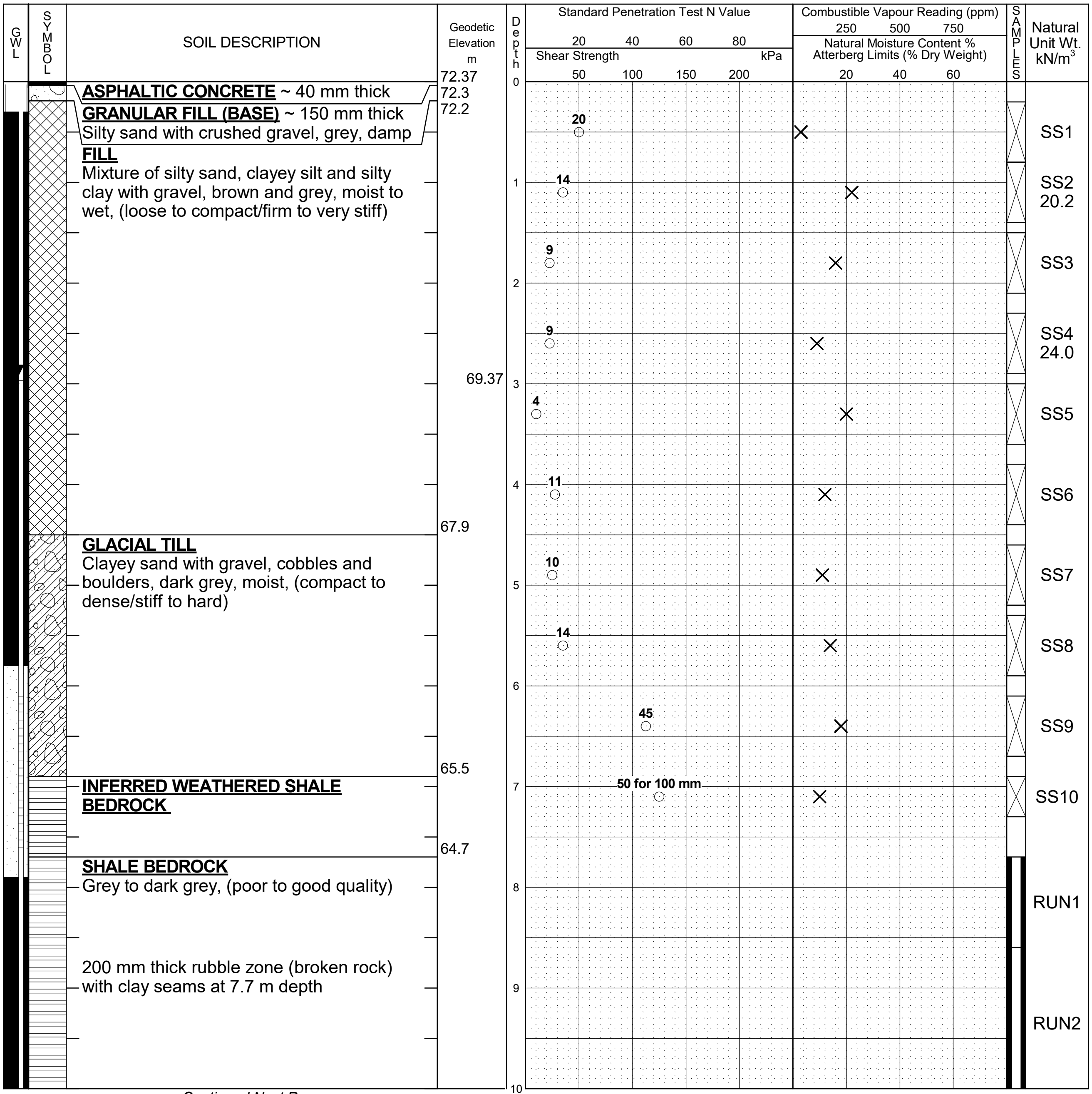
Date Drilled: 'August 21, 2020

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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	▲



72.37

72.3 **ASPHALTIC CONCRETE** ~ 40 mm thick

72.2 **GRANULAR FILL (BASE)** ~ 150 mm thick
Silty sand with crushed gravel, grey, damp

FILL
Mixture of silty sand, clayey silt and silty clay with gravel, brown and grey, moist to wet, (loose to compact/firm to very stiff)

69.37

67.9 **GLACIAL TILL**
Clayey sand with gravel, cobbles and boulders, dark grey, moist, (compact to dense/stiff to hard)

65.5 **INFERRED WEATHERED SHALE BEDROCK**

64.7 **SHALE BEDROCK**
Grey to dark grey, (poor to good quality)

200 mm thick rubble zone (broken rock) with clay seams at 7.7 m depth

Continued Next Page

LOG OF BOREHOLE LOGS OF BOREHOLES - 1740&1760 ST LAURENT.GPJ TROW OTTAWA.GDT 9/22/20

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Two (2) - 19 mm diameter standpipes installed to 7.6 m and 17.6 m depths as shown.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Aug. 21, 2020	2.5	5.5
Sept. 3, 2020	3.1/3.0	
Sept. 10, 2020	3.2/3.0	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.7 - 8.6	97	76
2	8.6 - 10.1	100	70
3	10.1 - 11.6	100	67
4	11.6 - 13.2	100	19
5	13.2 - 14.7	100	47
6	14.7 - 16.1	100	15
7	16.1 - 17.6	100	71

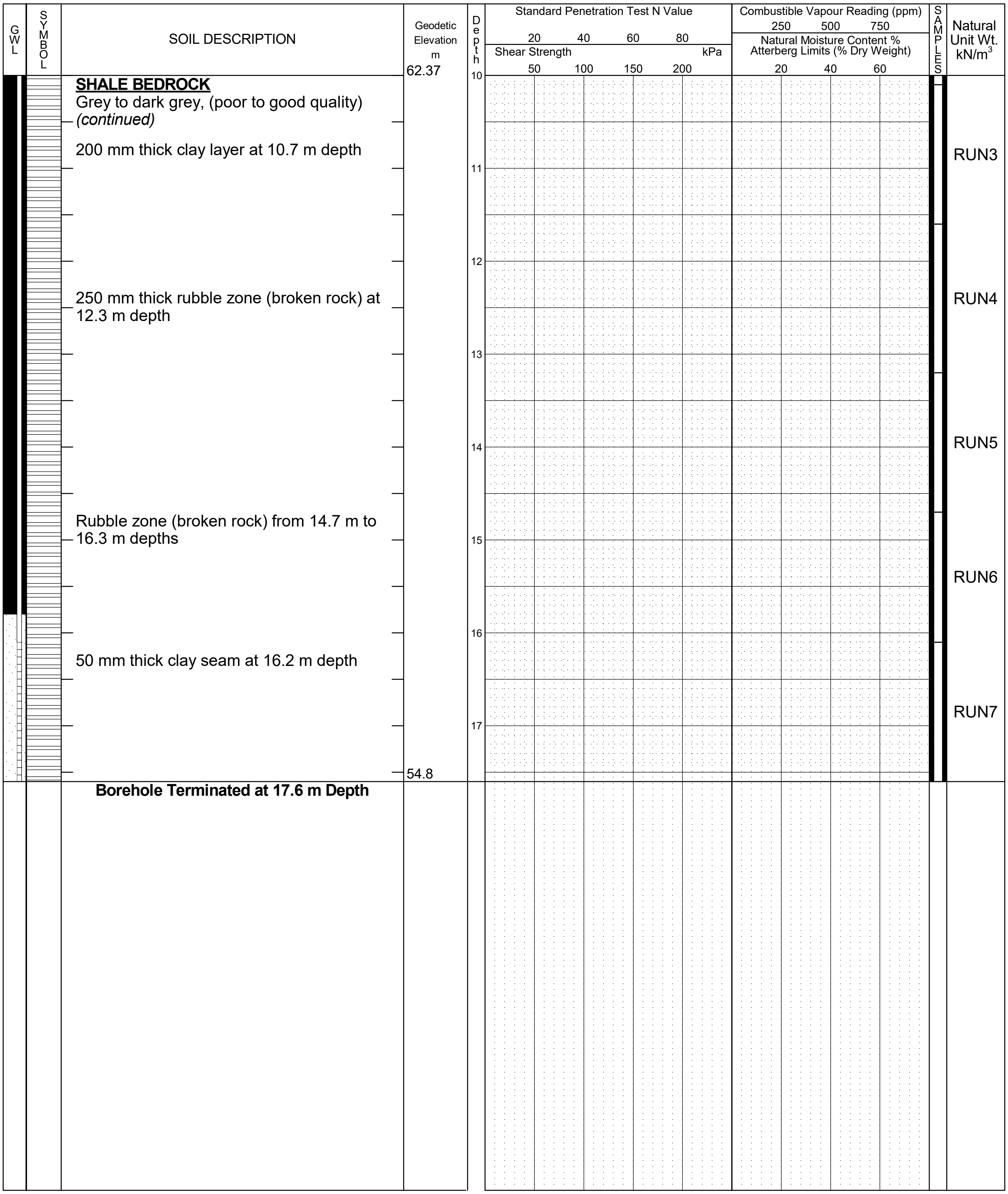
Log of Borehole BH20-05

Project No: OTT-00260579-A0

Project: Residential Development

Figure No. 7

Page. 2 of 2



SHALE BEDROCK
Grey to dark grey, (poor to good quality) *(continued)*

200 mm thick clay layer at 10.7 m depth

250 mm thick rubble zone (broken rock) at 12.3 m depth

Rubble zone (broken rock) from 14.7 m to 16.3 m depths

50 mm thick clay seam at 16.2 m depth

Borehole Terminated at 17.6 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Two (2) - 19 mm diameter standpipes installed to 7.6 m and 17.6 m depths as shown.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Aug. 21, 2020	2.5	5.5
Sept. 3, 2020	3.1/3.0	
Sept. 10, 2020	3.2/3.0	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.7 - 8.6	97	76
2	8.6 - 10.1	100	70
3	10.1 - 11.6	100	67
4	11.6 - 13.2	100	19
5	13.2 - 14.7	100	47
6	14.7 - 16.1	100	15
7	16.1 - 17.6	100	71

Log of Borehole **BH20-06**

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 8

Page. 1 of 1

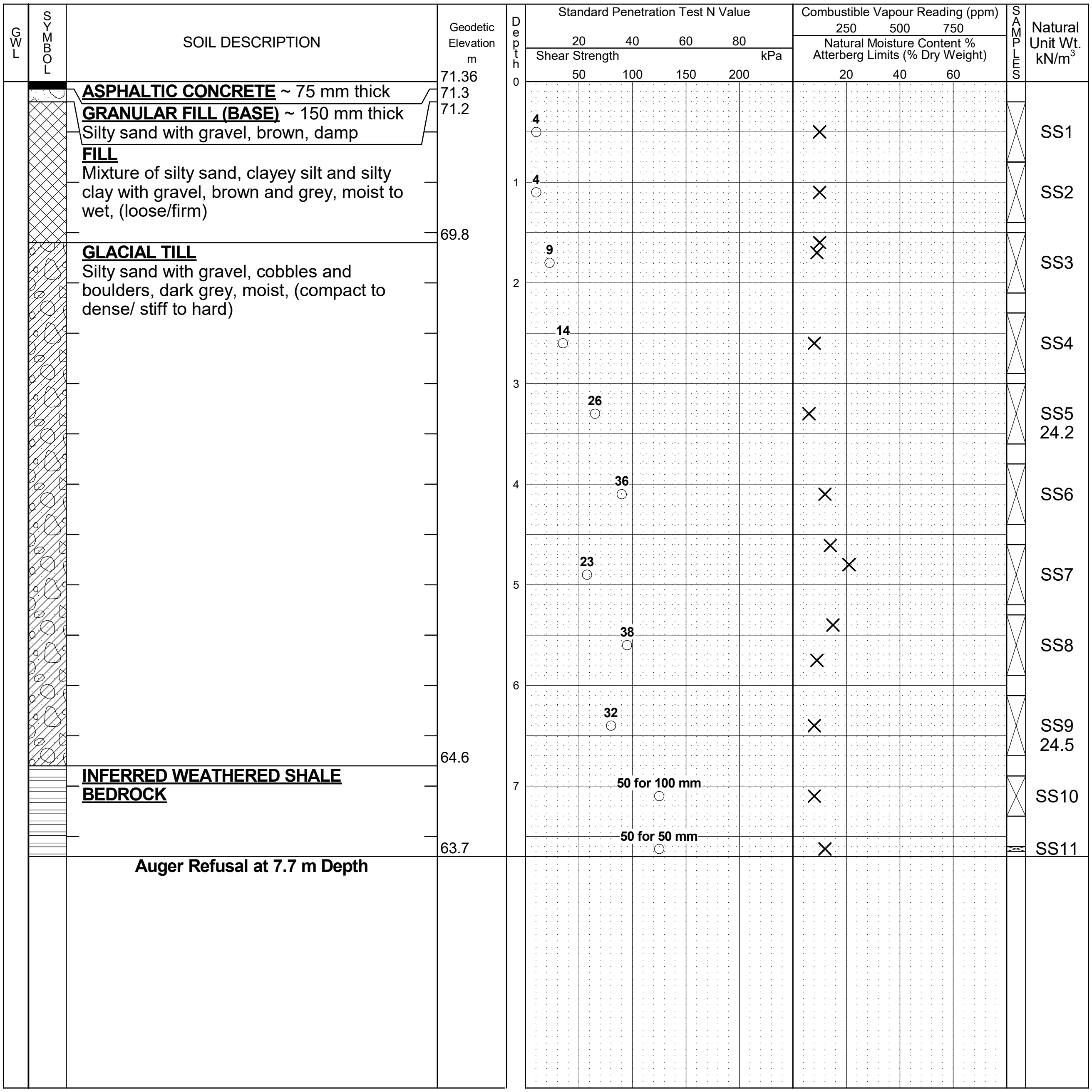
Date Drilled: 'August 20, 2020

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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	▲



Geodetic Elevation m	SOIL DESCRIPTION
71.36	
71.3	**ASPHALTIC CONCRETE** ~ 75 mm thick
71.2	**GRANULAR FILL (BASE)** ~ 150 mm thick Silty sand with gravel, brown, damp
	FILL Mixture of silty sand, clayey silt and silty clay with gravel, brown and grey, moist to wet, (loose/firm)
69.8	**GLACIAL TILL** Silty sand with gravel, cobbles and boulders, dark grey, moist, (compact to dense/ stiff to hard)
64.6	**INFERRED WEATHERED SHALE BEDROCK**
63.7	**Auger Refusal at 7.7 m Depth**

Samples	SPT N Value	Natural Unit Wt. kN/m³
SS1	4	
SS2	4	
SS3	9	
SS4	14	
SS5	26	24.2
SS6	36	
SS7	23	
SS8	38	
SS9	32	24.5
SS10	50 for 100 mm	
SS11	50 for 50 mm	

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

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

Log of Borehole BH20-07

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 9

Page. 1 of 1

Date Drilled: 'August 20, 2020

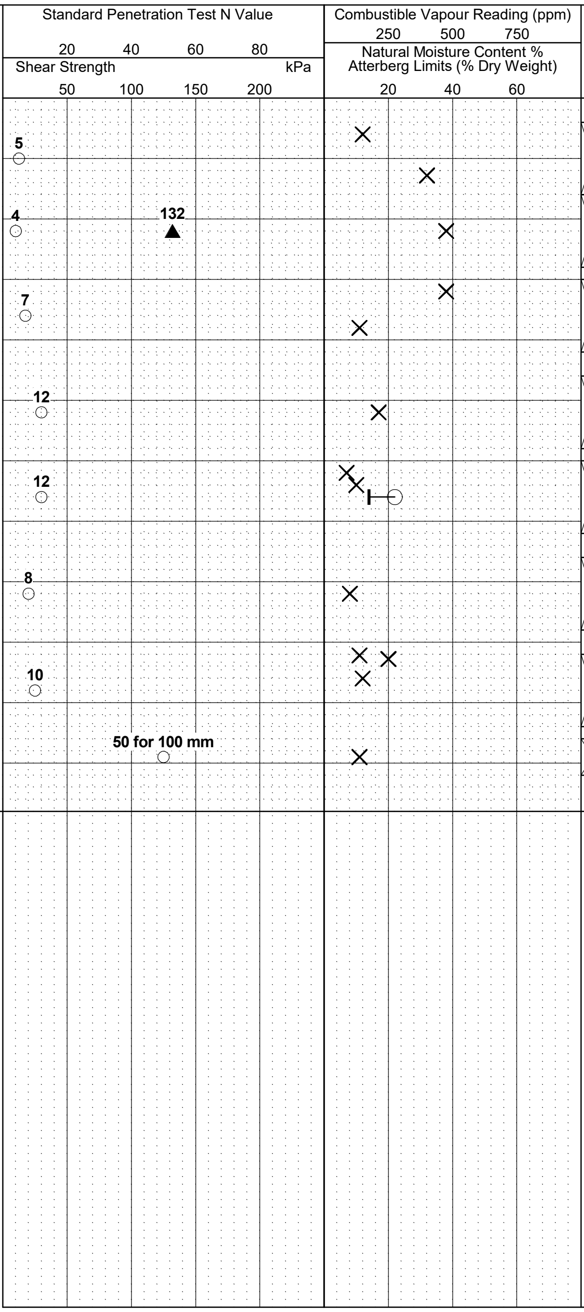
Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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
			70.27	0
		ASPHALTIC CONCRETE ~ 35 mm thick	70.2	
		GRANULAR FILL (BASE) ~ 150 mm thick Silty sand with crushed gravel, grey, damp	70.1	
		SILTY CLAY Brown, moist, (very stiff)		1
		GLACIAL TILL Clayey sand with gravel, cobbles and boulders, brown, moist, (loose to compact/stiff)	68.5	2
		GLACIAL TILL Clayey sand with gravel, shale fragments, cobbles and boulders, dark grey, moist to wet, (loose to compact/stiff)	67.2	3
				4
		with a 150 mm thick silty sand layer at 4.6 m depth		5
		INFERRED WEATHERED SHALE BEDROCK	65.0	
			64.4	
		Auger Refusal at 5.9 m Depth		



SAMPLES	Natural Unit Wt. kN/m³
SS1	
SS2	18.6
SS3	
SS4	
SS5	
SS6	
SS7	
SS8	

LOG OF BOREHOLE LOGS OF BOREHOLES - 1740&1760 ST LAURENT.GPJ TROW OTTAWA.GDT 9/22/20

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

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

Log of Borehole BH20-08

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 10

Page. 1 of 1

Date Drilled: 'August 20, 2020

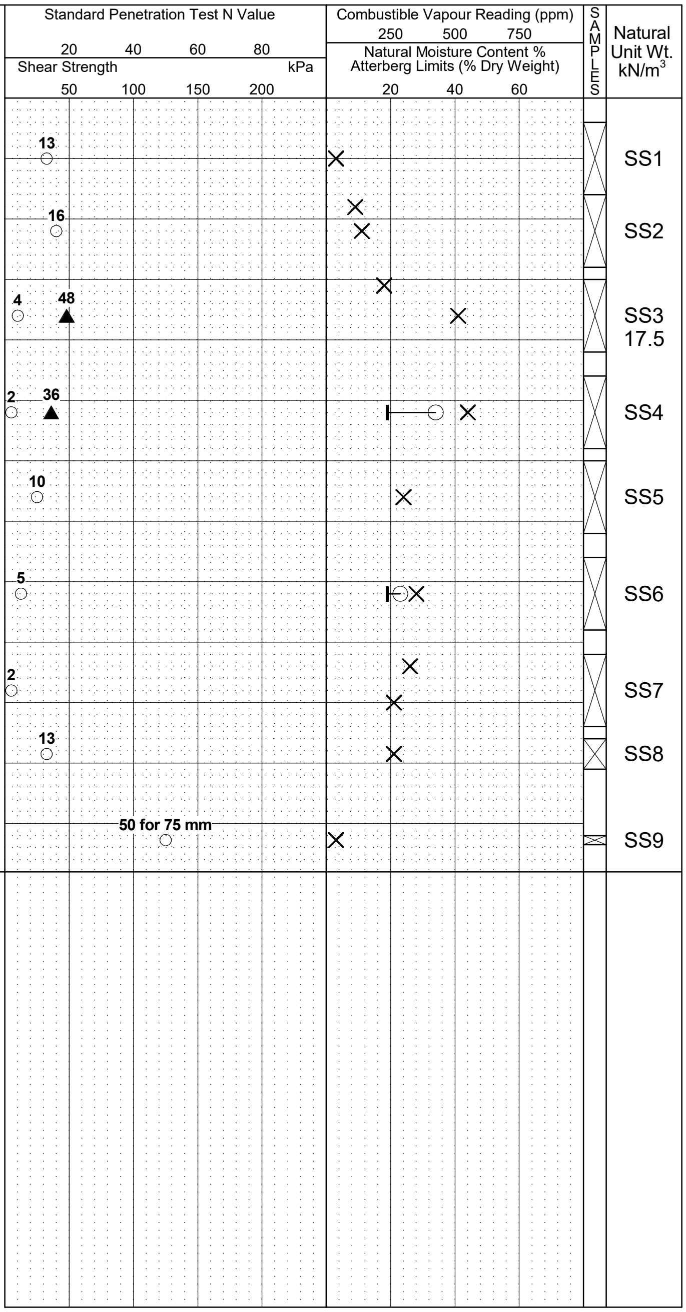
Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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	▲

GWL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	SAMPLES	Natural Unit Wt. kN/m³
		69.68	0		
	ASPHALTIC CONCRETE ~ 50 mm thick	69.6			
	GRANULAR FILL (BASE) ~ 450 mm thick Silty sand with crushed gravel, grey, damp	69.2		SS1	
	FILL Mixture of silty sand, clayey silt and silty clay with gravel, brown and grey, moist to wet, (compact/stiff to very stiff)	68.1	1	SS2	
	SILTY CLAY Low plasticity, brown, moist, (firm)	66.7	2	SS3 SS4	17.5
	GLACIAL TILL Silty clay with gravel, low plasticity, cobbles and boulders, grey to dark grey, (very loose to compact/soft to stiff)	63.6	3 4 5 6	SS5 SS6 SS7 SS8	
	INFERRED WEATHERED SHALE BEDROCK	63.3		SS9	
	Auger Refusal at 6.4 m Depth				



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

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

Log of Borehole BH20-09

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 11

Page. 1 of 1

Date Drilled: 'August 25, 2020

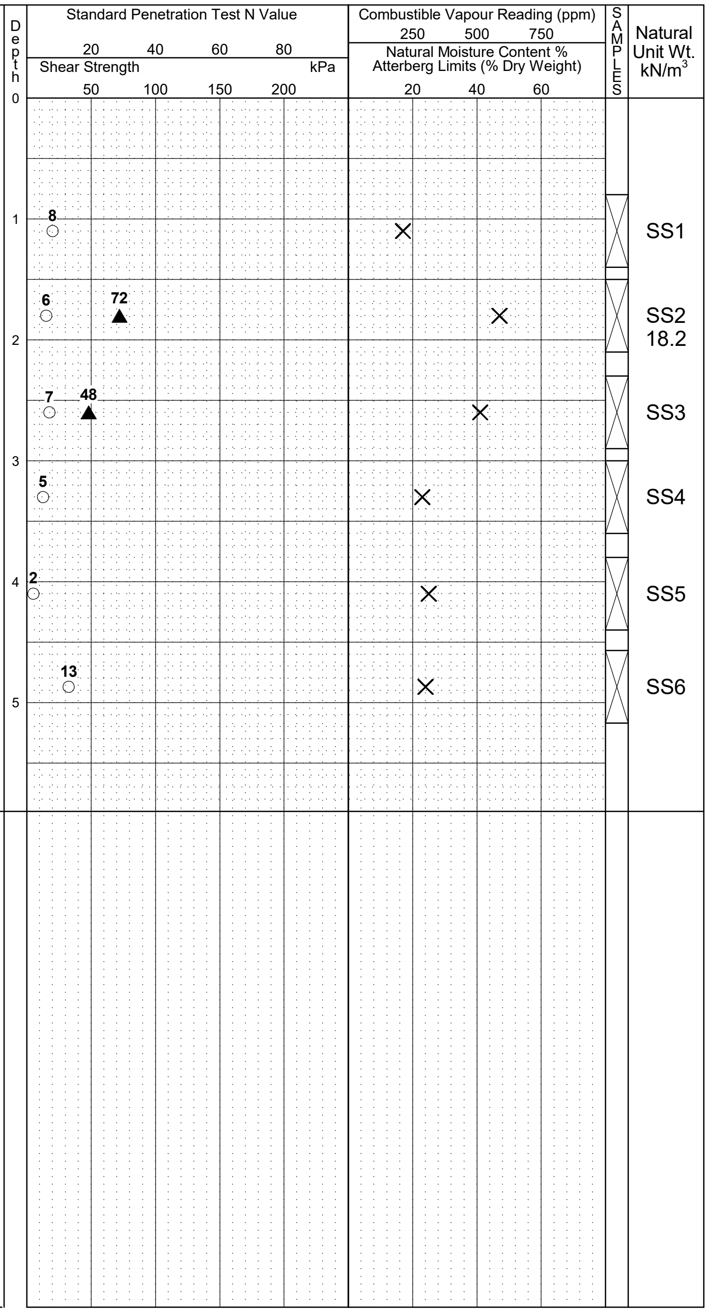
Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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
	69.35
ASPHALTIC CONCRETE ~ 60 mm thick	69.3
GRANULAR FILL (BASE) ~ 200 mm thick Silty sand with crushed gravel, grey, damp	69.1
FILL Silty sand with gravel, brown, moist	68.6
SILTY CLAY Brown, moist, (stiff)	66.9
SILTY CLAY With silt partings, grey, wet, (soft to firm)	64.9
GLACIAL TILL Clayey sand with gravel, cobbles and boulders, grey, wet, (compact/stiff)	63.5
Auger Refusal at 5.9 m Depth	



SS1	8		
SS2	6	72	18.2
SS3	7	48	
SS4	5		
SS5	2		
SS6	13		

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
completion	2.5	5.8

CORE DRILLING RECORD

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

Log of Borehole BH20-10

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 12

Page. 1 of 2

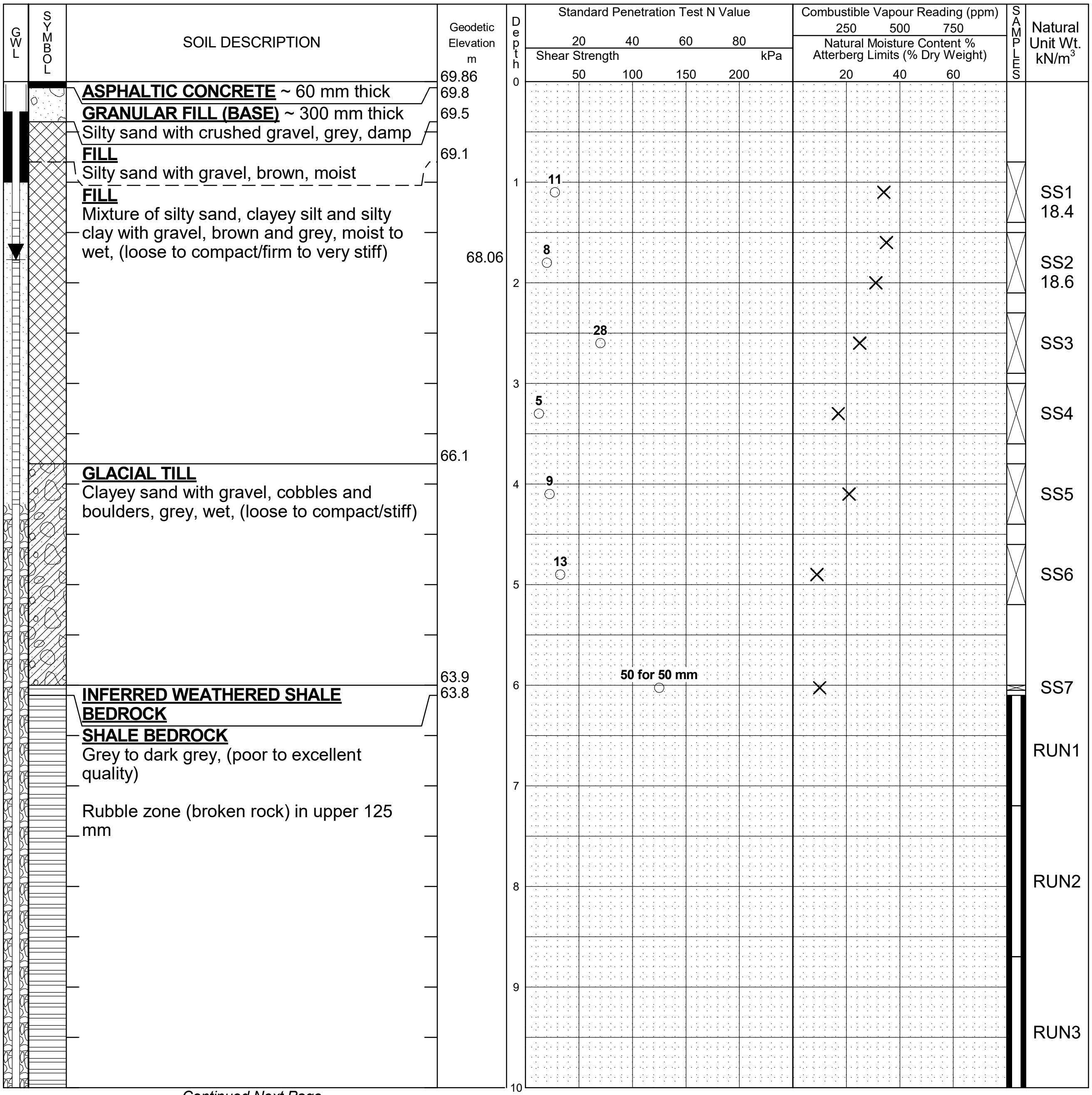
Date Drilled: 'August 24, 2020

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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	▲



ASPHALTIC CONCRETE ~ 60 mm thick

GRANULAR FILL (BASE) ~ 300 mm thick
Silty sand with crushed gravel, grey, damp

FILL
Silty sand with gravel, brown, moist

FILL
Mixture of silty sand, clayey silt and silty clay with gravel, brown and grey, moist to wet, (loose to compact/firm to very stiff)

GLACIAL TILL
Clayey sand with gravel, cobbles and boulders, grey, wet, (loose to compact/stiff)

INFERRED WEATHERED SHALE BEDROCK

SHALE BEDROCK
Grey to dark grey, (poor to excellent quality)

Rubble zone (broken rock) in upper 125 mm

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50 mm diameter monitoring well installed to a 4.2 m depth as shown.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Aug. 24, 2020	2.3	6.0
Sept. 3, 2020	1.8	
Sept. 10, 2020	1.8	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.1 - 7.2	100	36
2	7.2 - 8.7	100	68
3	8.7 - 10.2	98	98
4	10.2 - 11.7	100	95
5	11.7 - 13.2	100	92
6	13.2 - 14.7	100	96

Log of Borehole BH20-10

Project No: OTT-00260579-A0

Project: Residential Development

Figure No. 12

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
		SHALE BEDROCK Grey to dark grey, (poor to excellent quality) Rubble zone (broken rock) in upper 125 mm *(continued)*	59.86	10 11				RUN4
				12 13				RUN5
			55.2	14				RUN6
		Borehole Terminated at 14.7 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50 mm diameter monitoring well installed to a 4.2 m depth as shown.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Aug. 24, 2020	2.3	6.0
Sept. 3, 2020	1.8	
Sept. 10, 2020	1.8	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.1 - 7.2	100	36
2	7.2 - 8.7	100	68
3	8.7 - 10.2	98	98
4	10.2 - 11.7	100	95
5	11.7 - 13.2	100	92
6	13.2 - 14.7	100	96

Log of Borehole BH20-11

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 13

Page. 1 of 1

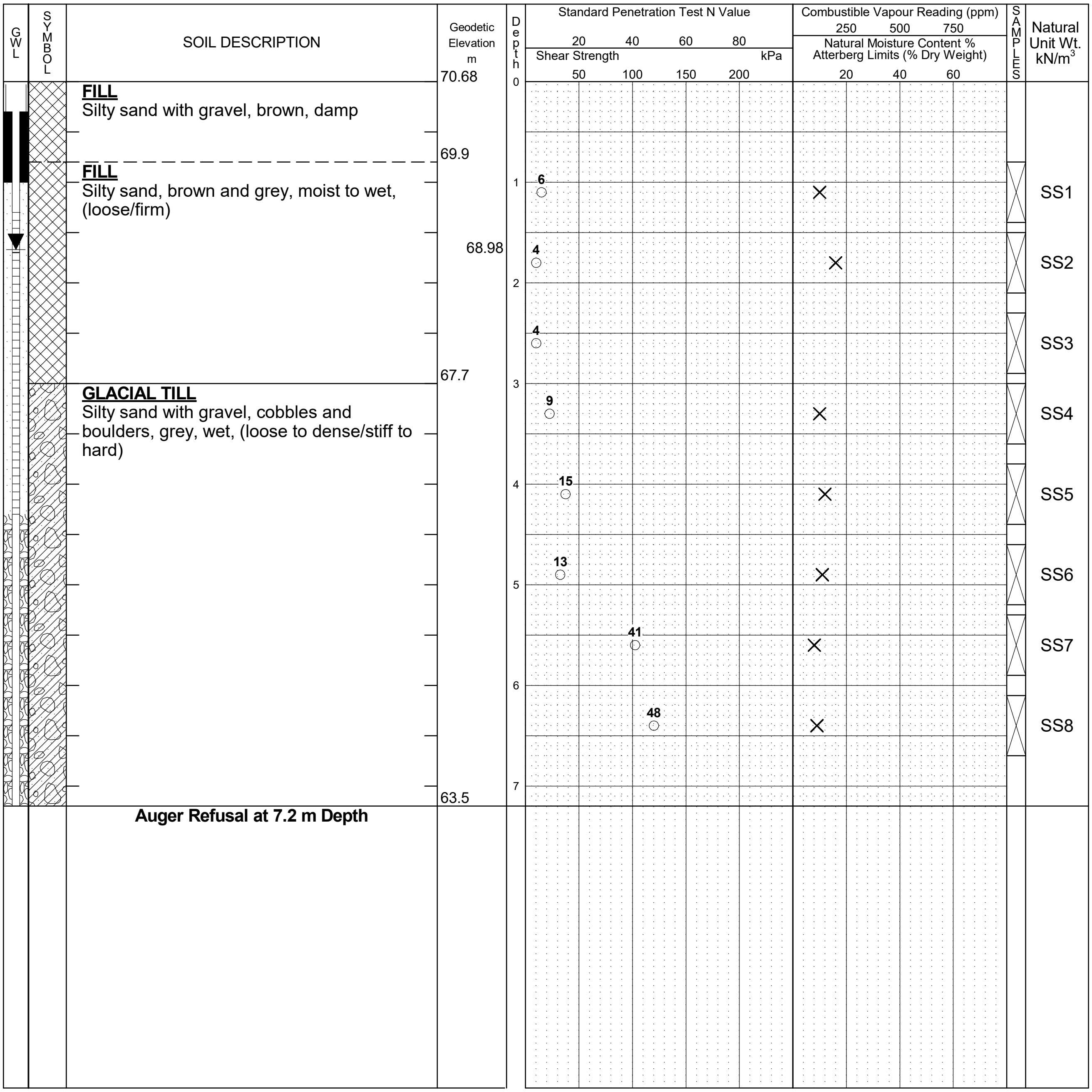
Date Drilled: 'August 25, 2020

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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	▲



FILL
Silty sand with gravel, brown, damp

FILL
Silty sand, brown and grey, moist to wet, (loose/firm)

GLACIAL TILL
Silty sand with gravel, cobbles and boulders, grey, wet, (loose to dense/stiff to hard)

Auger Refusal at 7.2 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50 mm diameter monitoring well installed to a 4.2 m depth as shown.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Aug. 25, 2020	3.0	-
Sept. 3, 2020	1.7	

CORE DRILLING RECORD

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

Log of Borehole BH20-12

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 14

Page. 1 of 1

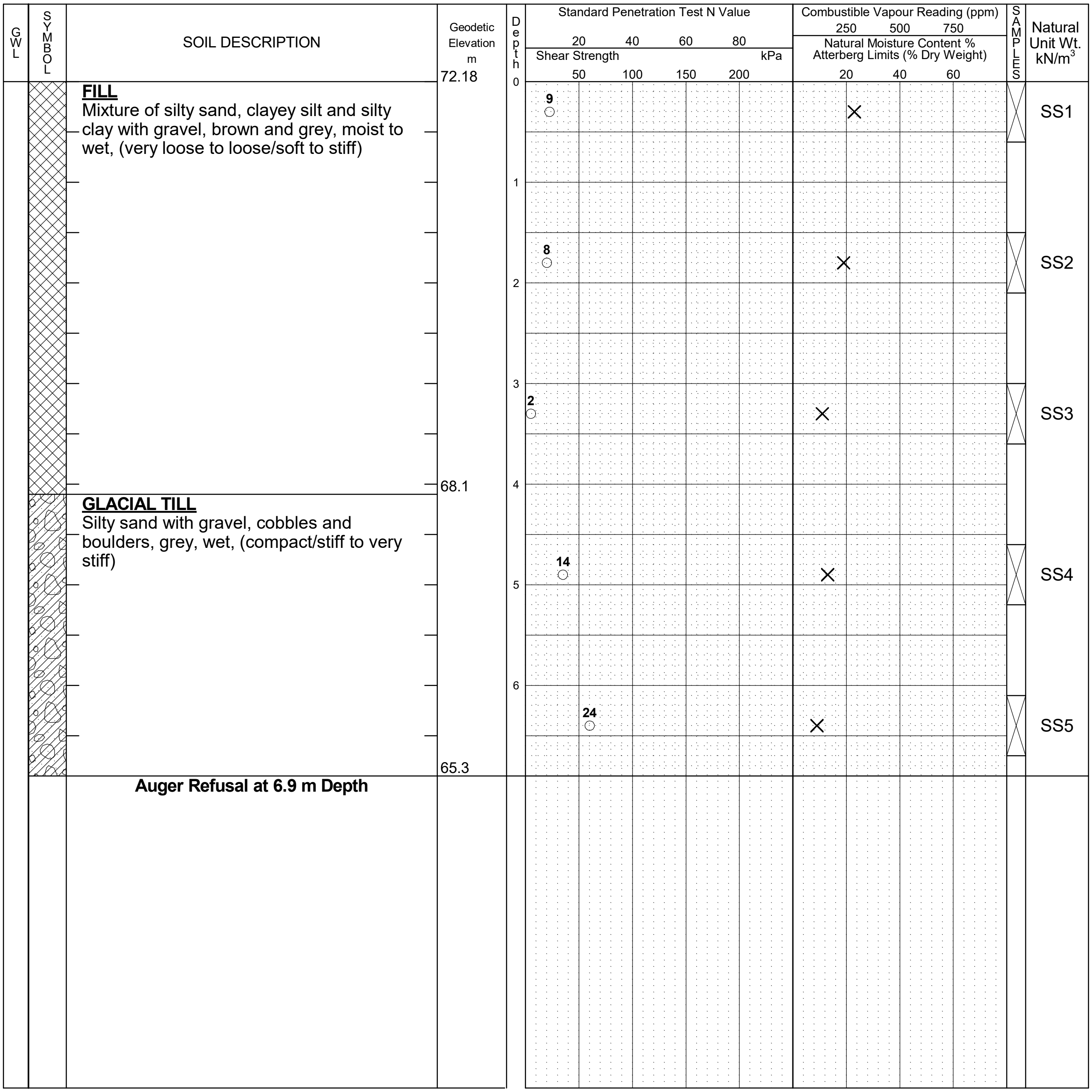
Date Drilled: 'August 25, 2020

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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		



FILL

Mixture of silty sand, clayey silt and silty clay with gravel, brown and grey, moist to wet, (very loose to loose/soft to stiff)

GLACIAL TILL

Silty sand with gravel, cobbles and boulders, grey, wet, (compact/stiff to very stiff)

Auger Refusal at 6.9 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
completion	3.0	6.7

CORE DRILLING RECORD

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

Log of Borehole BH20-13

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 15

Page. 1 of 2

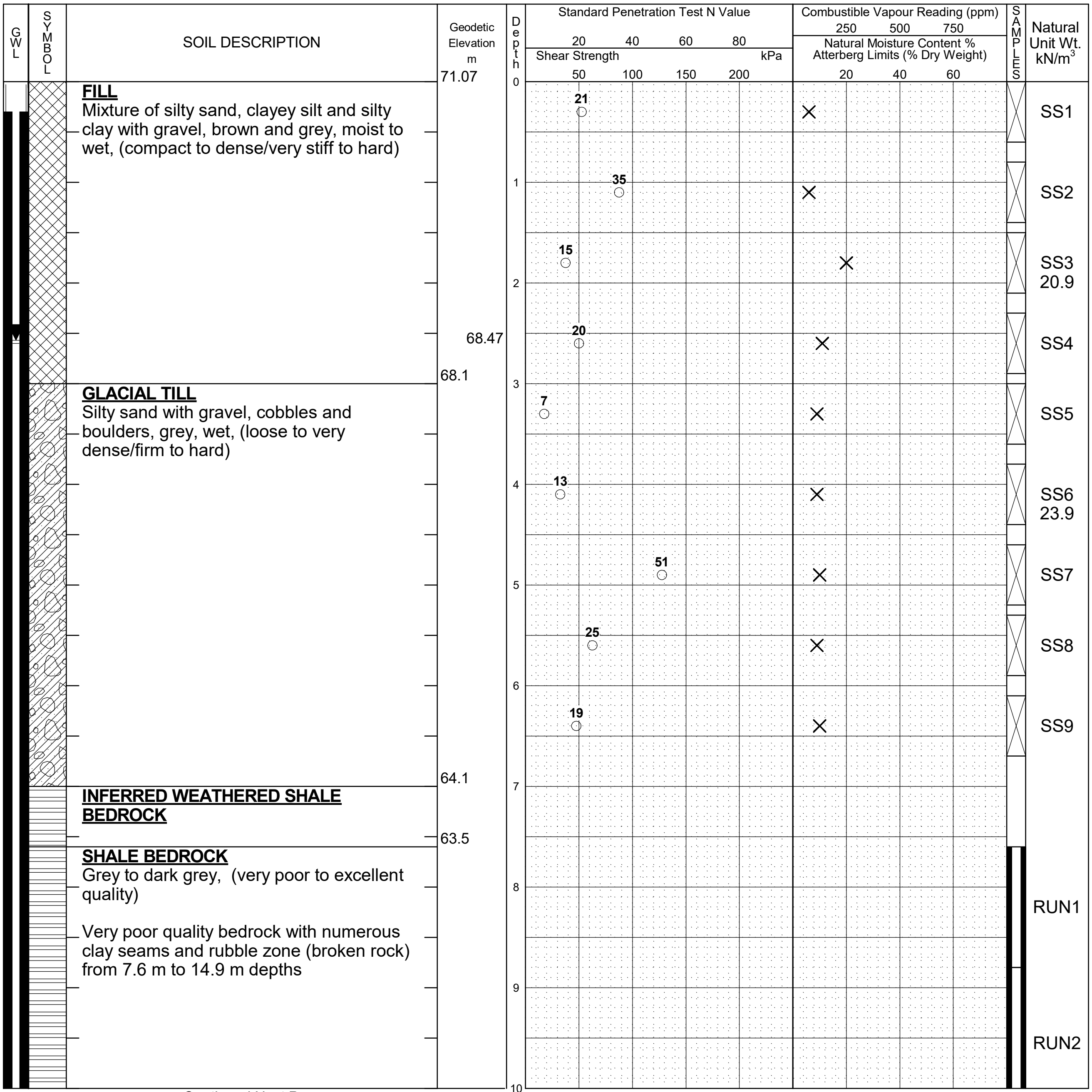
Date Drilled: August 20 and 21, 2020

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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		



FILL
Mixture of silty sand, clayey silt and silty clay with gravel, brown and grey, moist to wet, (compact to dense/very stiff to hard)

GLACIAL TILL
Silty sand with gravel, cobbles and boulders, grey, wet, (loose to very dense/firm to hard)

INFERRED WEATHERED SHALE BEDROCK

SHALE BEDROCK
Grey to dark grey, (very poor to excellent quality)

Very poor quality bedrock with numerous clay seams and rubble zone (broken rock) from 7.6 m to 14.9 m depths

Continued Next Page

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe installed to a 17.7 m depth as shown.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
-	-	-
Sept. 3, 2020	2.6	
Sept. 10, 2020	2.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.6 - 8.8	22	0
2	8.8 - 10.3	71	0
3	10.3 - 11.8	92	13
4	11.8 - 13.4	92	6
5	13.4 - 14.7	67	0
6	14.7 - 16.3	100	77
7	16.3 - 17.7	100	100

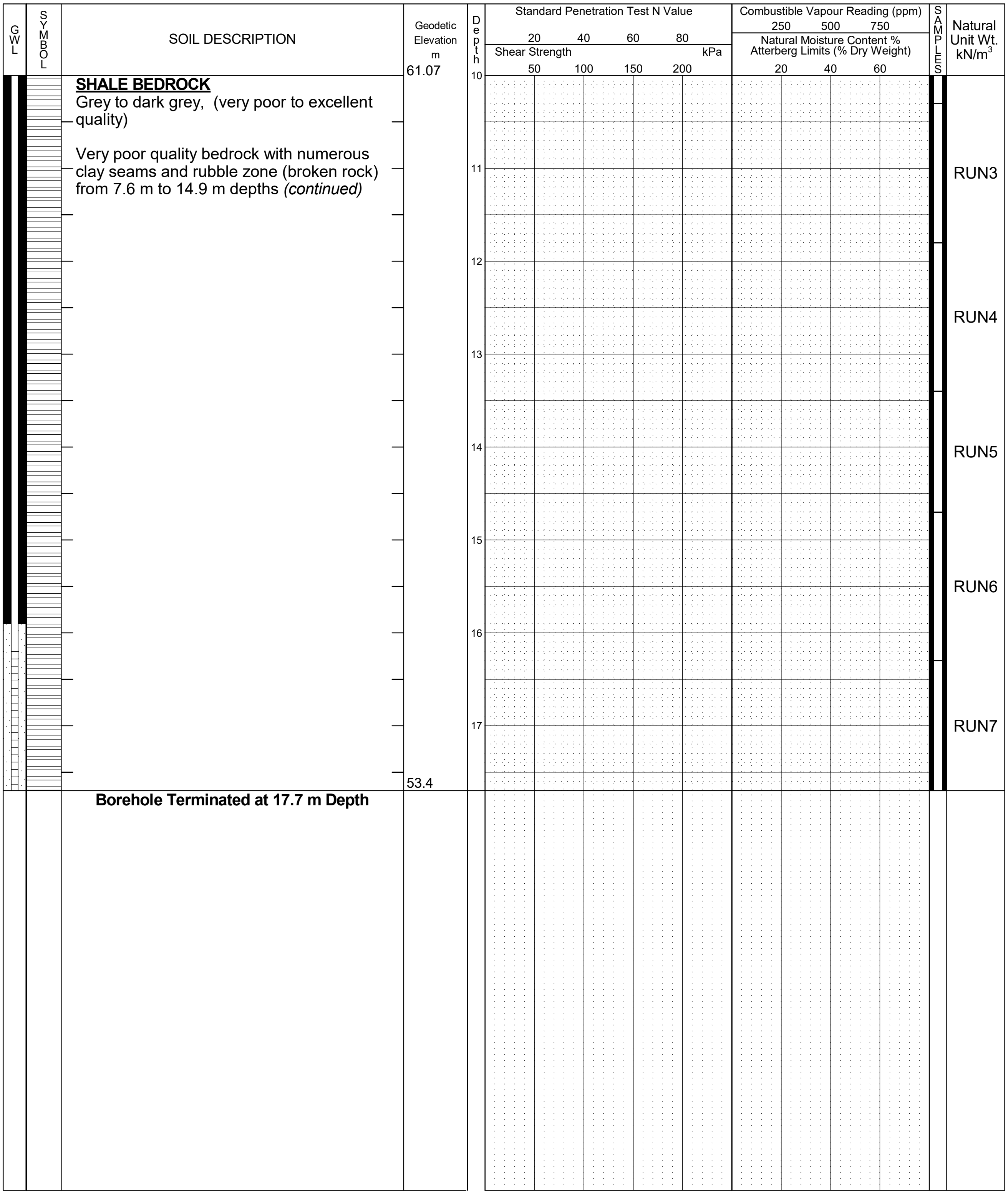
Log of Borehole BH20-13

Project No: OTT-00260579-A0

Figure No. 15

Project: Residential Development

Page. 2 of 2



SHALE BEDROCK
Grey to dark grey, (very poor to excellent quality)

Very poor quality bedrock with numerous clay seams and rubble zone (broken rock) from 7.6 m to 14.9 m depths *(continued)*

Borehole Terminated at 17.7 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe installed to a 17.7 m depth as shown.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
-	-	-
Sept. 3, 2020	2.6	
Sept. 10, 2020	2.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	7.6 - 8.8	22	0
2	8.8 - 10.3	71	0
3	10.3 - 11.8	92	13
4	11.8 - 13.4	92	6
5	13.4 - 14.7	67	0
6	14.7 - 16.3	100	77
7	16.3 - 17.7	100	100

Log of Borehole **BH20-14**

Project No: OTT-00260579-A0

Project: Residential Development

Location: 1740 - 1760 St. Laurent Boulevard, Ottawa, On.

Figure No. 16

Page. 1 of 1

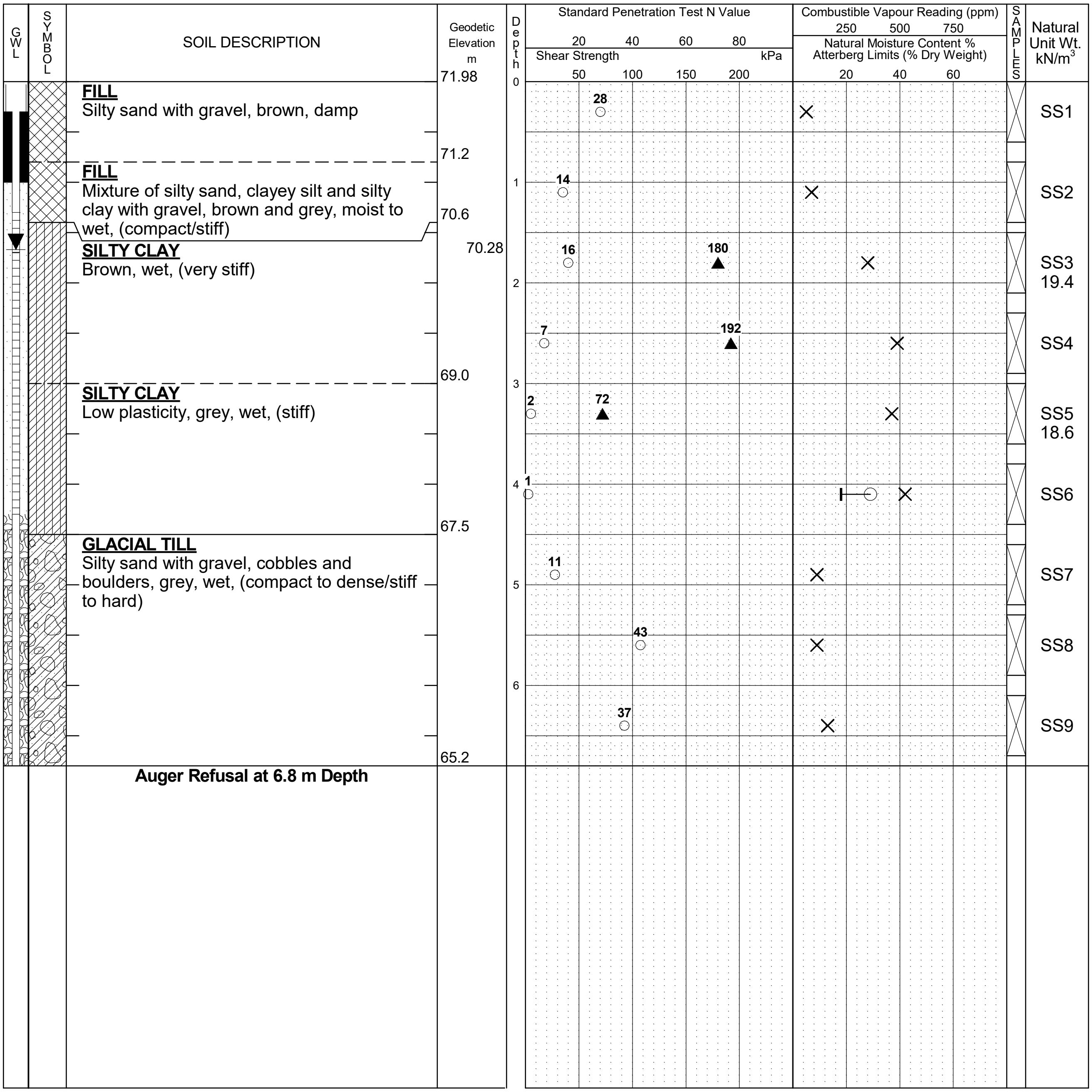
Date Drilled: 'August 25, 2020

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

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)
	71.98
FILL Silty sand with gravel, brown, damp	71.2
FILL Mixture of silty sand, clayey silt and silty clay with gravel, brown and grey, moist to wet, (compact/stiff)	70.6
SILTY CLAY Brown, wet, (very stiff)	70.28 69.0
SILTY CLAY Low plasticity, grey, wet, (stiff)	67.5
GLACIAL TILL Silty sand with gravel, cobbles and boulders, grey, wet, (compact to dense/stiff to hard)	65.2

Auger Refusal at 6.8 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50 mm diameter monitoring well installed to a 4.2 m depth as shown.
3. Fieldwork supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00260579-A0

WATER LEVEL RECORDS

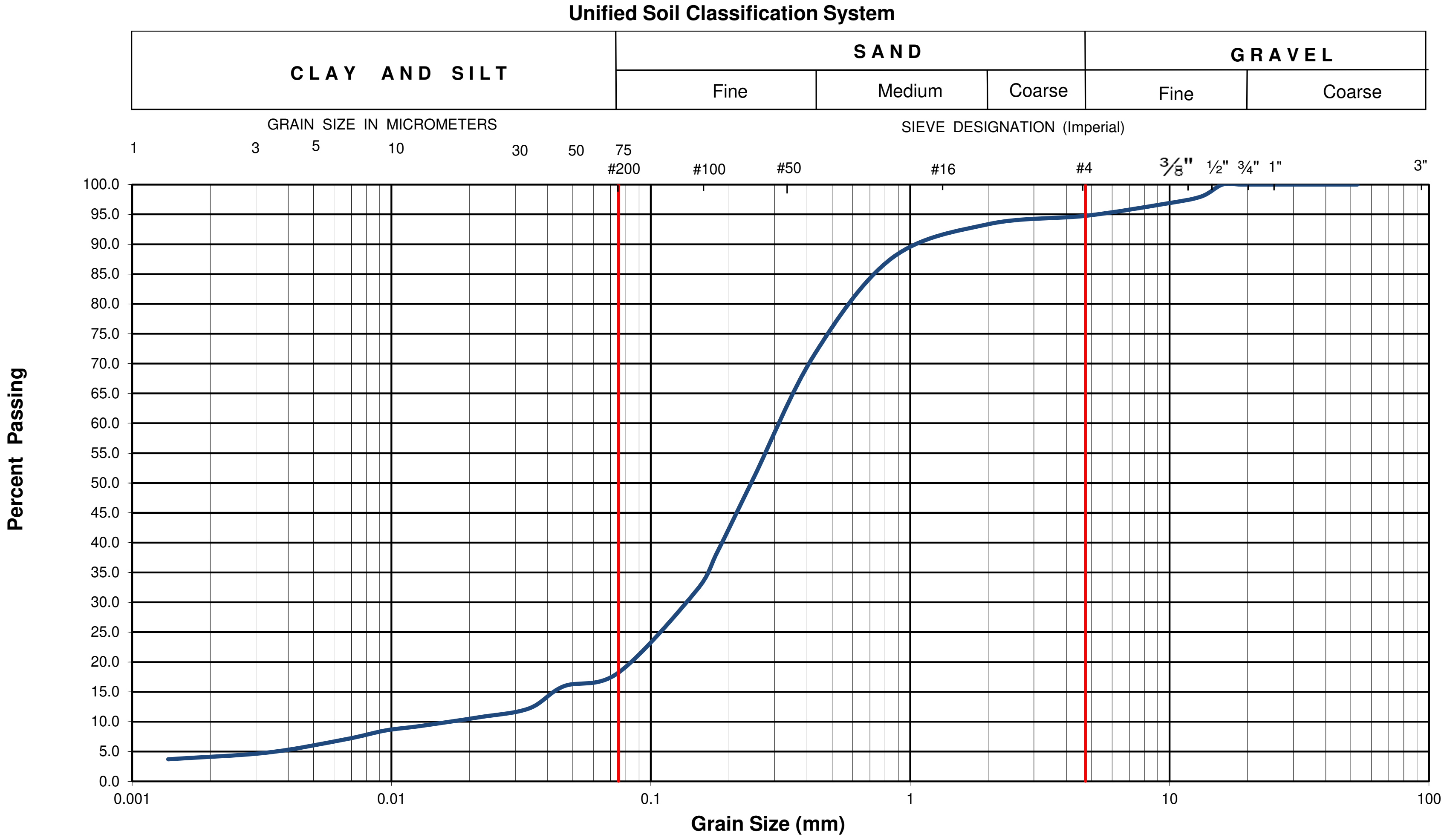
Date	Water Level (m)	Hole Open To (m)
Aug. 25, 2020	3.0	-
Sept. 3, 2020	1.7	

CORE DRILLING RECORD

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

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

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



EXP Project No.:	**OTT-00260579**	**Project Name :**	**Geotechnical Investigation - Proposed Residential Development**				
Client :	**Heafy Group**	**Project Location :**	**1740-1760 St. Laurent Blvd. Ottawa,Ontario**				
Date Sampled :	**August 25, 2020**	**Borehole No:**	**BH20-11**	**Sample No.:**	**SS2**	**Depth (m) :**	**1.5-2.1**
Sample Description :		**% Silt and Clay**	**18**	**% Sand**	**77**	**% Gravel**	**5**
Sample Description :	**FILL: Silty Sand (SM)**					**Figure :**	**17**

EXP Services Inc.
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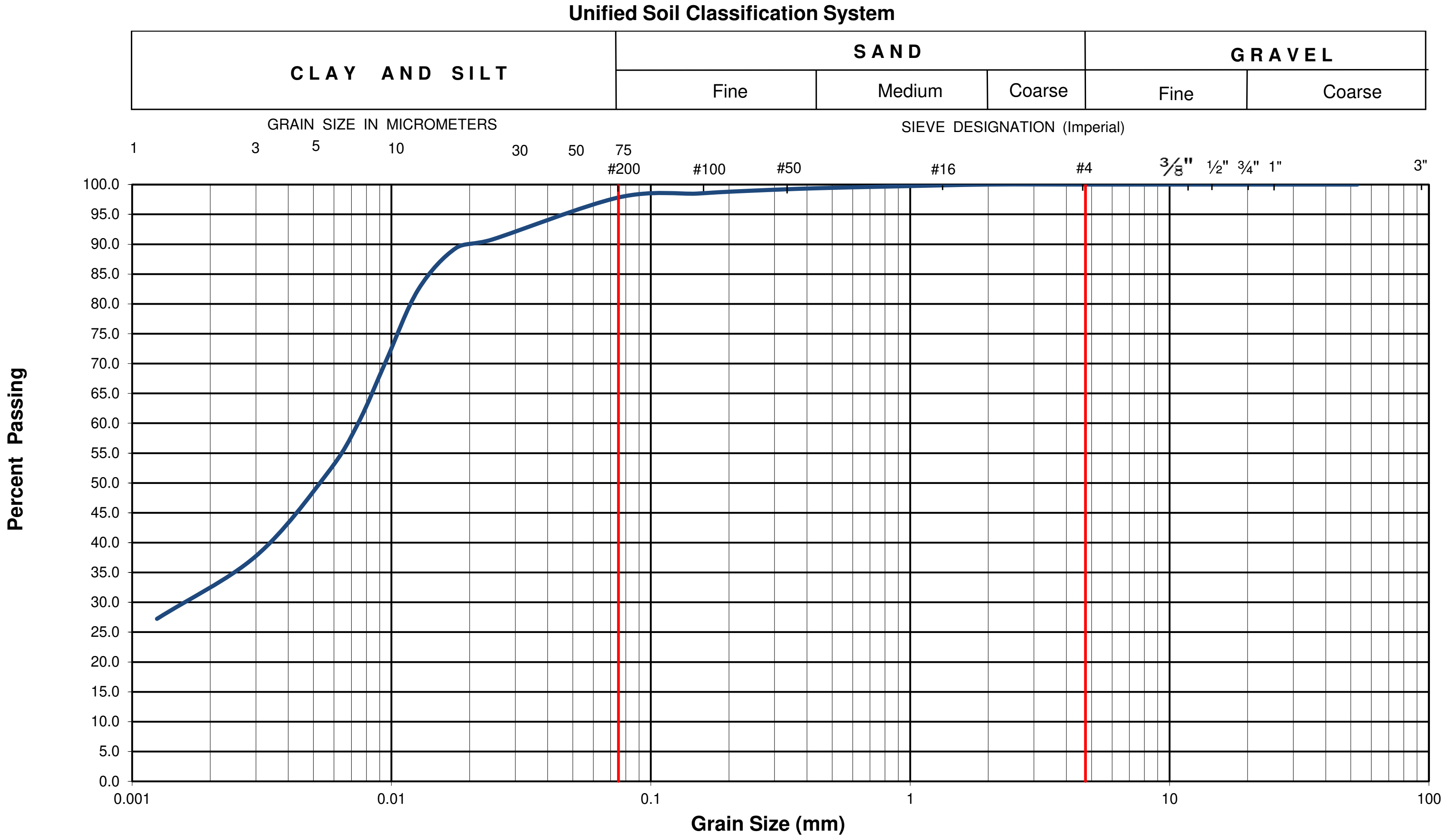
Ottawa, ON K2B 8H6

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



EXP Project No.:	OTT-00260579	**Project Name :**	Geotechnical Investigation - Proposed Residential Development					
Client :	Heafy Group	**Project Location :**	1740-1760 St. Laurent Blvd. Ottawa, Ontario					
Date Sampled :	August 25, 2020	**Borehole No:**	BH20-08	**Sample No.:**	SS4		**Depth (m) :**	2.3-2.9
Sample Description :		**% Silt and Clay**	98	**% Sand**	2	**% Gravel** 0	**Figure :**	18
Sample Description :	BROWN SILTY CLAY CRUST: Low Plasticity (CL)							

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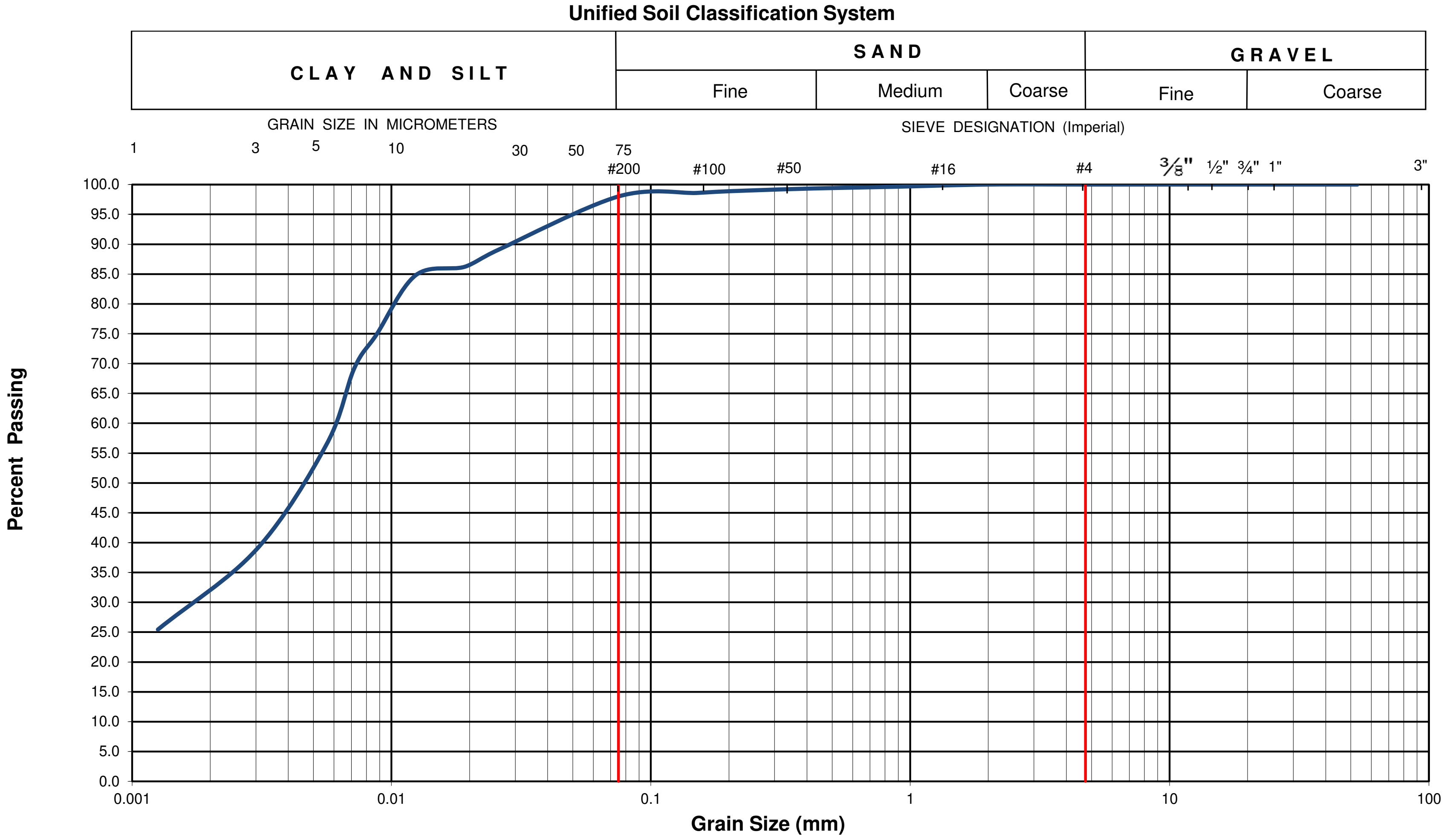
Ottawa, ON K2B 8H6

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



EXP Project No.:	**OTT-00260579**	**Project Name :**	**Geotechnical Investigation - Proposed Residential Development**						
Client :	**Heafy Group**	**Project Location :**	**1740-1760 St. Laurent Blvd, Ottawa, Ontario**						
Date Sampled :	**August 25, 2020**	**Borehole No:**	**BH20-14**		**Sample No.:**	**SS6**		**Depth (m) :**	**3.8-4.4**
Sample Description :		**% Silt and Clay**	**98**	**% Sand**	**2**	**% Gravel**	**0**	**Figure :**	**19**
Sample Description :		**GREY SILTY CLAY: Low Plasticity (CL)**							

EXP Services Inc.
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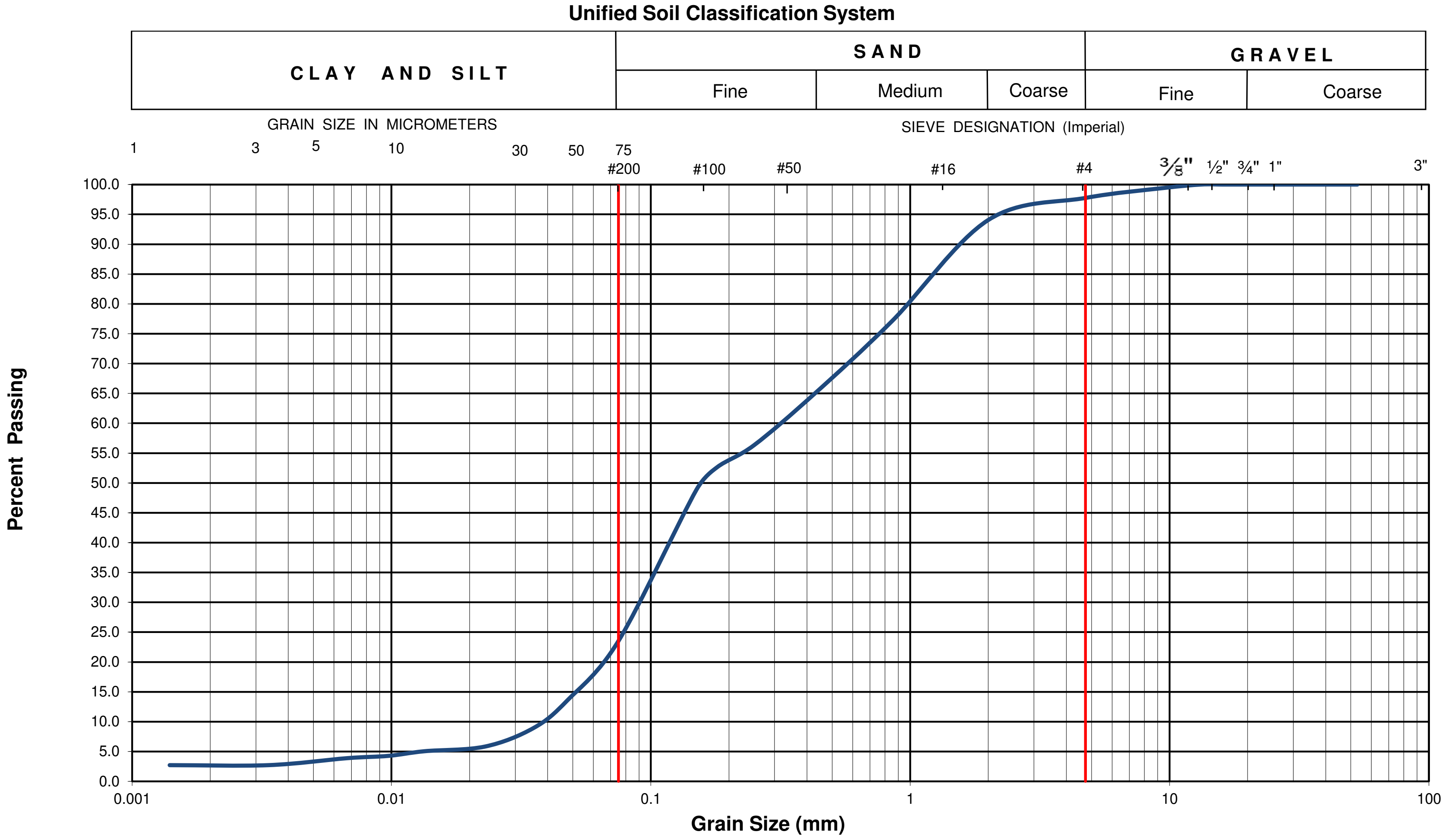
Ottawa, ON   K2B 8H6

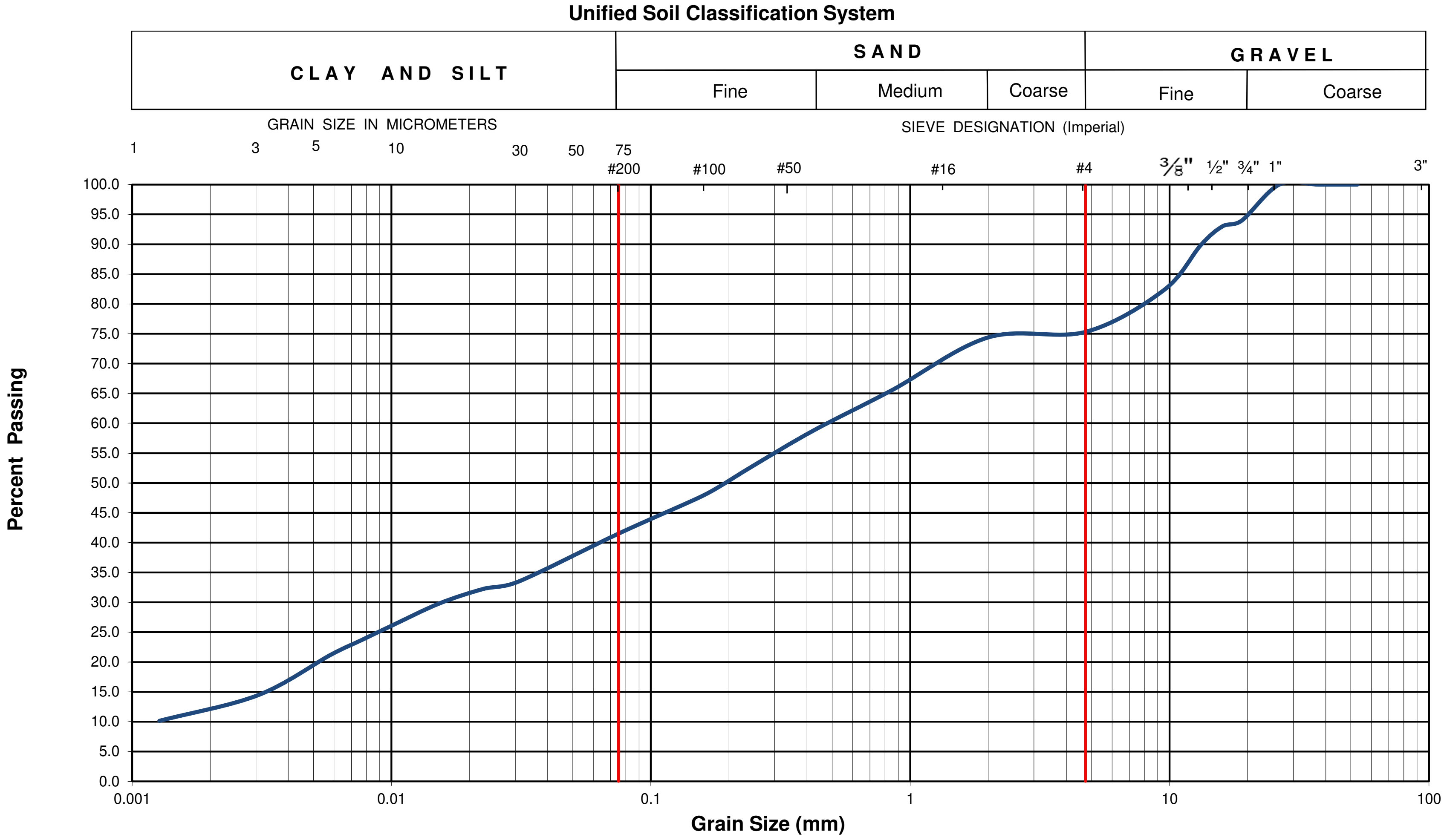
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00260579**	**Project Name :**	**Geotechnical Investigation - Proposed Residential Development**						
Client :	**Heafy Group**	**Project Location :**	**1740-1760 St. Laurent Blvd. Ottawa, Ontario**						
Date Sampled :	**August 25, 2020**	**Borehole No:**	**BH20-06**		**Sample No.:**	**SS8**		**Depth (m) :**	**5.3-5.9**
Sample Description :		**% Silt and Clay**	**24**	**% Sand**	**74**	**% Gravel**	**2**	**Figure :**	**20**
Sample Description :	**GLACIAL TILL: Silty Sand (SM)**								

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

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



EXP Project No.:	**OTT-00260579**	**Project Name :**	**Geotechnical Investigation - Proposed Residential Development**						
Client :	**Heafy Group**	**Project Location :**	**1740-1760 St. Laurent Blvd. Ottawa, Ontario**						
Date Sampled :	**August 25, 2020**	**Borehole No:**	**BH20-07**	**Sample No.:**	**SS4-SS6**			**Depth (m) :**	**2.3-4.4**
Sample Description :		**% Silt and Clay**	**42**	**% Sand**	**33**	**% Gravel**	**25**	**Figure :**	**21**
Sample Description :	**GLACIAL TILL: Clayey Sand with Gravel (SC)**								

EXP Services Inc.
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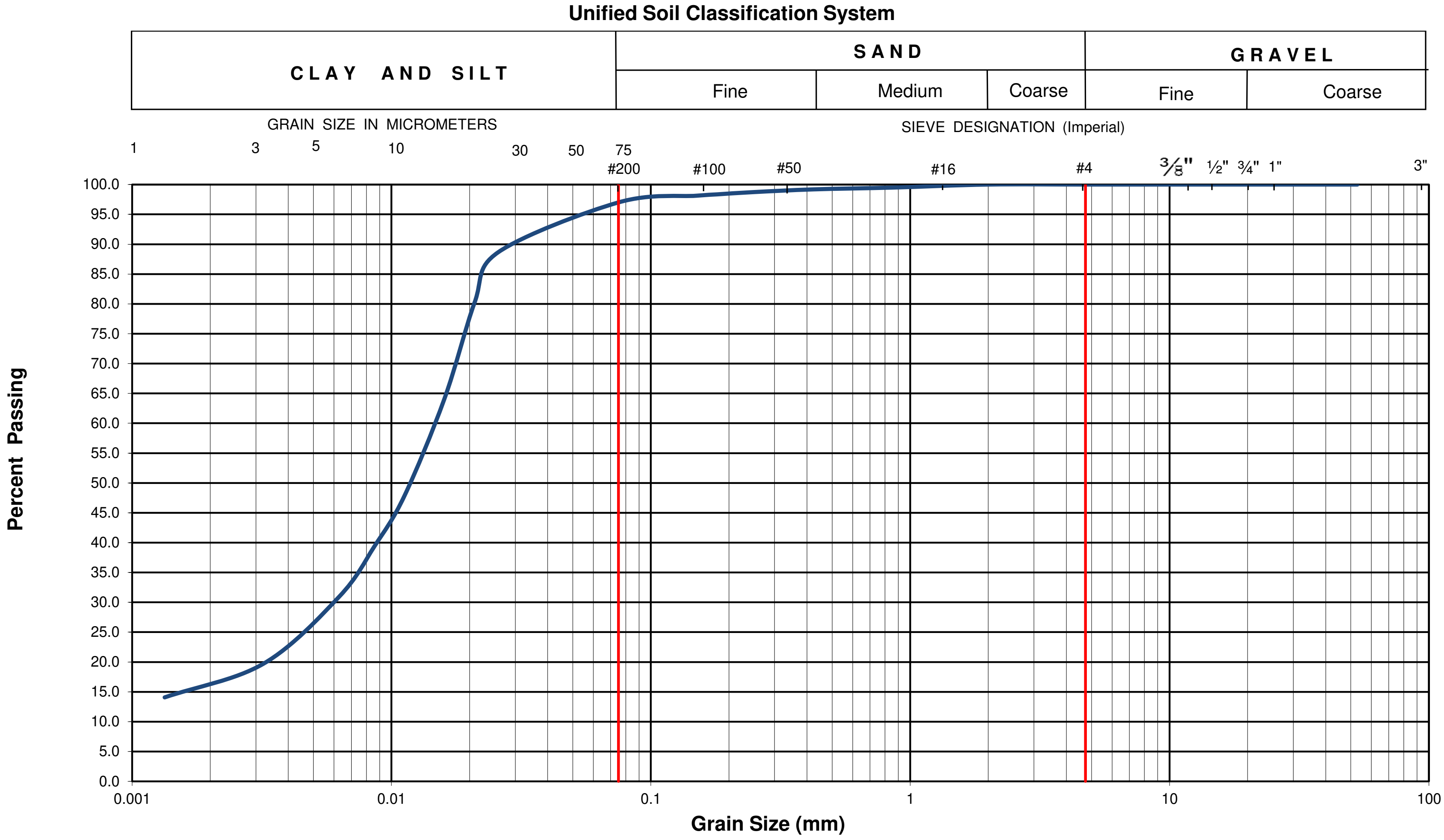
Ottawa, ON K2B 8H6

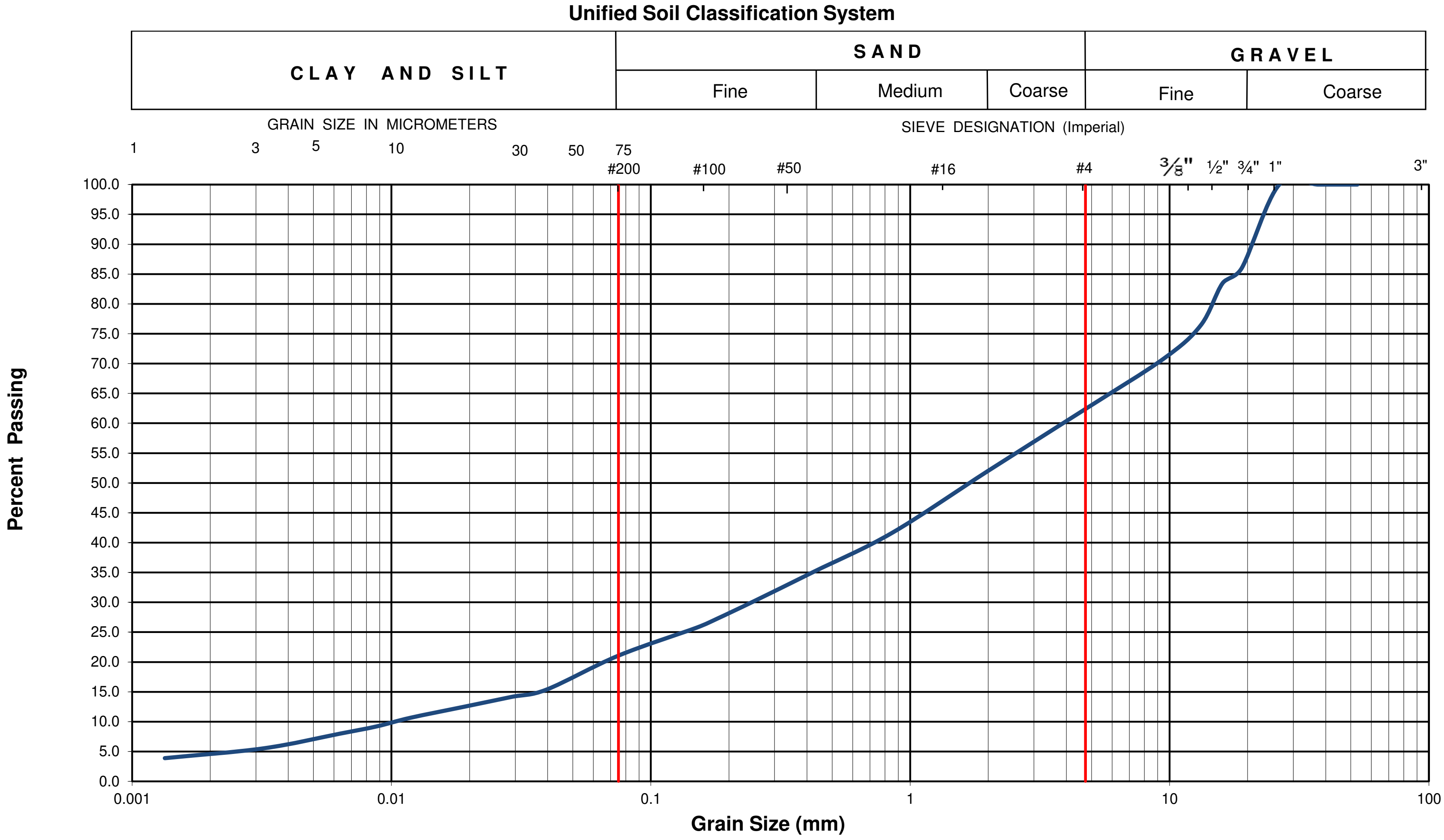
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-00260579	**Project Name :**	Geotechnical Investigation - Proposed Residential Development				
Client :	Heafy Group	**Project Location :**	1740-1760 St. Laurent Blvd. Ottawa, Ontario				
Date Sampled :	August 25, 2020	**Borehole No:**	BH20-08	**Sample No.:**	SS6	**Depth (m) :**	3.8-4.4
Sample Description :		**% Silt and Clay** 97	**% Sand** 3	**% Gravel**	0	**Figure :**	22
Sample Description :	GLACIAL TILL: Silty Clay of Low Plasticity (CL-ML)						

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

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



EXP Project No.:	**OTT-00260579**	**Project Name :**	**Geotechnical Investigation - Proposed Residential Development**						
Client :	**Heafy Group**	**Project Location :**	**1740-1760 St. Laurent Blvd. Ottawa, Ontario**						
Date Sampled :	**August 25, 2020**	**Borehole No:**	**BH20-14**		**Sample No.:**	**SS8**		**Depth (m) :**	**5.3-5.9**
Sample Description :		**% Silt and Clay**	**21**	**% Sand**	**41**	**% Gravel**	**38**	**Figure :**	**23**
Sample Description :	**GLACIAL TILL: Silty Sand with Gravel (SM)**								

Appendix A – Laboratory Certificate of Analysis

AGAT Laboratories

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

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

ATTENTION TO: **Susan Potyondy**

PROJECT: **OTT-260579**

AGAT WORK ORDER: **20Z647765**

SOIL ANALYSIS REVIEWED BY: **Nivine Basily, Inorganics Report Writer**

DATE REPORTED: **Sep 15, 2020**

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 following analysis, unless expressly agreed otherwise in writing. Please contact your Client Project Manager if you require additional sample storage time.*
- *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.*

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: 20Z647765

PROJECT: OTT-260579

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

CLIENT NAME: EXP SERVICES INC

ATTENTION TO: Susan Potyondy

SAMPLING SITE:1760 & 1740 St. Laurent Blvd, Ottawa, ON

SAMPLED BY:EXP

Inorganic Chemistry (Soil)

DATE RECEIVED: 2020-09-08

DATE REPORTED: 2020-09-15

				BH5 Run 3	BH10 Run 4	BH14 Run 6
			SAMPLE DESCRIPTION:	35'9"-36'4"	34'2"-34'7"	51'10"-52'3"
			SAMPLE TYPE:	Soil	Soil	Soil
			DATE SAMPLED:	2020-08-20	2020-08-20	2020-08-20
Parameter	**Unit**	**G / S**	**RDL**	**1426448**	**1426452**	**1426453**
Chloride (2:1)	µg/g		2	25	29	20
Sulphate (2:1)	µg/g		2	55	11	30
pH (2:1)	pH Units		NA	8.42	9.41	9.36
Resistivity (2:1) (Calculated)	ohm.cm		1	2640	2380	1740

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

1426448-1426453 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 *)

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
NIVINE BASILY
CHEMIST

Certified By:

Quality Assurance

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-260579
SAMPLING SITE:1760 & 1740 St. Laurent Blvd, Ottawa, ON

AGAT WORK ORDER: 20Z647765
ATTENTION TO: Susan Potyondy
SAMPLED BY:EXP

Soil Analysis

RPT Date: Sep 15, 2020			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Inorganic Chemistry (Soil)															
Chloride (2:1)	1426448	1426448	25	25	0.0%	< 2	99%	70%	130%	104%	80%	120%	106%	70%	130%
Sulphate (2:1)	1426448	1426448	55	55	0.0%	< 2	96%	70%	130%	102%	80%	120%	101%	70%	130%
pH (2:1)	1426448	1426448	8.42	8.47	0.6%	NA	97%	90%	110%						

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

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
NIVINE BASILY
CHEMIST

Certified By: Nivine Basily

Method Summary

CLIENT NAME: EXP SERVICES INC **AGAT WORK ORDER: 20Z647765**

PROJECT: OTT-260579 **ATTENTION TO: Susan Potyondy**

SAMPLING SITE:1760 & 1740 St. Laurent Blvd, Ottawa, ON **SAMPLED BY:EXP**

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
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
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
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 #: 20Z647765

Cooler Quantity: one - no ice

Arrival Temperatures: 21.5 | 21.5 | 21.6

(LT- 9.8 | (on ice)

Custody Seal Intact: ☐ Yes ☐ No ☐ N/A

Notes:

Report Information:

Company: EXP Services

Contact: Susan Polyondy

Address: 2650 Queensview Drive Suite 100
Ottawa ON K2B 8H6

Phone: 613-688-1899 Fax:

Reports to be sent to:

1. Email: Susan.Polyondy@exp.com
2. Email:

Regulatory Requirements: ☐ No Regulatory Requirement

(Please check all applicable boxes)

☐ Regulation 153/04

Table ______ Indicate One
☐ Ind/Com
☐ Res/Park
☐ Agriculture

Soil Texture (Check One)
☐ Coarse
☐ Fine

☐ Sewer Use
☐ Sanitary
☐ Storm

Region ______ Indicate One

☐ MISA

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

______ Indicate One

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

Report Guideline on Certificate of Analysis ☐ Yes ☐ No

Turnaround Time (TAT) Required:

Regular TAT ☒ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

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

OR Date Required (Rush Surcharges May Apply): ______

Please provide prior notification for rush TAT

**TAT is exclusive of weekends and statutory holidays*

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

Project Information:

Project: OTT-260579

Site Location: 1760 & 1740 St. Laurent Blvd, Ottawa, ON

Sampled By: EXP

AGAT Quote #: ______ PO: ______

Please note: If quotation number is not provided, client will be billed full price for analysis.

Invoice Information:

Bill To Same: Yes ☒ No ☐

Company:

Contact:

Address:

Email:

Sample Matrix Legend

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

O. Reg 153

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI Y / N	Metals and Inorganics	☐ All Metals ☐ 153 Metals (excl. Hydrides) ☐ Hydride Metals ☐ 153 Metals (incl. Hydrides)	ORPs: ☐ B-HWS ☐ Cl ☐ CN ☐ Cr^{6+} ☐ EC ☐ FOC ☐ Hg ☐ pH ☐ SAR	Full Metals Scan	Regulation/Custom Metals	Nutrients: ☐ TP ☐ NH_3 ☐ TKN ☐ NO_3 ☐ NO_2 ☐ NO_3+NO_2	Volatiles: ☐ VOC ☐ BTEX ☐ THM	PHCs F1 - F4	ABNs	PAHs	PCBs: ☐ Total ☐ Aroclors	Organochlorine Pesticides	TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	Sewer Use	pH	Chloride	Sulphate	Electro Ressistivity		Potentially Hazardous or High Concentration (Y/N)
BH 5 Run 3 35'9"-36'4"	Aug 20/20																				✓	✓	✓	✓		
BH 10 Run 4 34'2"-34'7"	Aug 20/20																				✓	✓	✓	✓		
BH 14 Run 6 51'10"-52'3"	Aug 20/20																				✓	✓	✓	✓		

Samples Relinquished By (Print Name and Sign): Ryan DiGiuseppe Date: Sept 4/20 Time: 7:15 pm

Samples Received By (Print Name and Sign): (Berthelet) Date: 2020/09/08 Time: 8h00

Samples Relinquished By (Print Name and Sign): W/O to FedEx Date: 2020/09/08 Time: 10h00

Samples Received By (Print Name and Sign): Date: Time:

Samples Relinquished By (Print Name and Sign): Date: Time:

Samples Received By (Print Name and Sign): Shanna Date: Sept 9/2020 Time: 8:40am

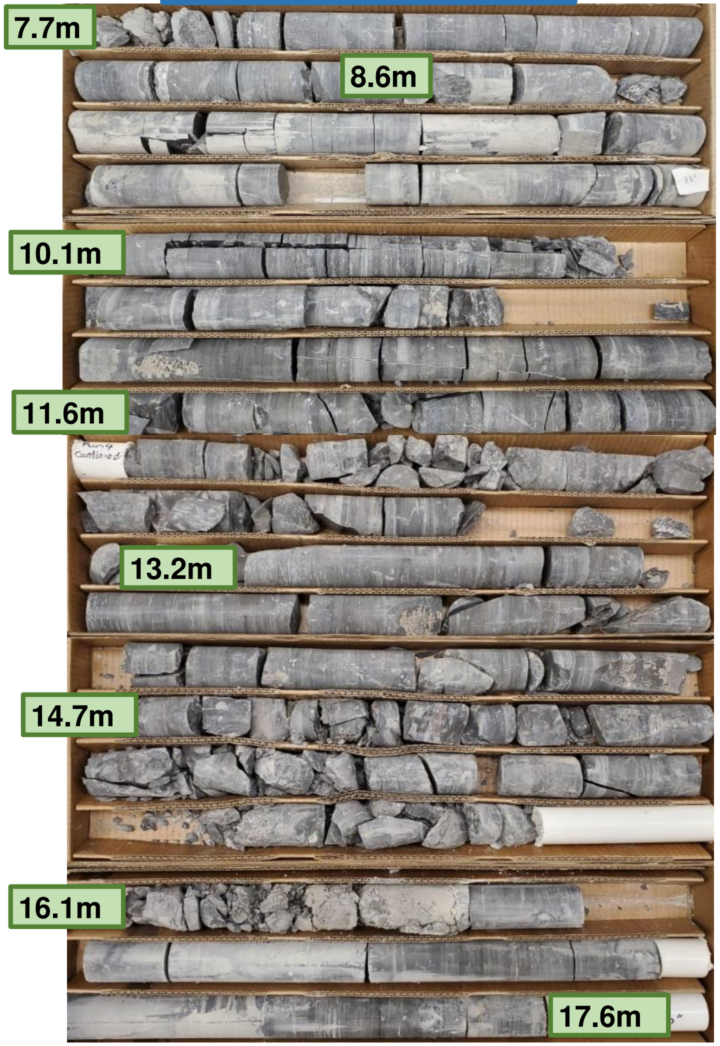
Page ______ of ______

Nº: T101276

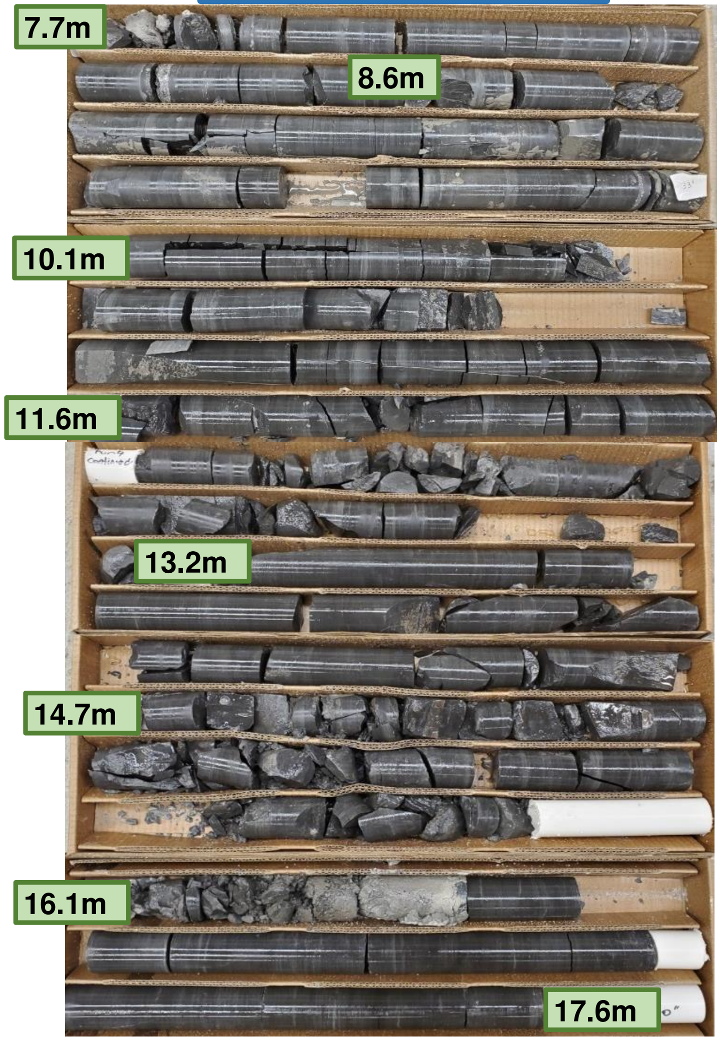
Pink Copy - Client | Yellow Copy - AGAT | White Copy- AGAT

Appendix B – Bedrock Core Photographs

DRY BEDROCK CORES

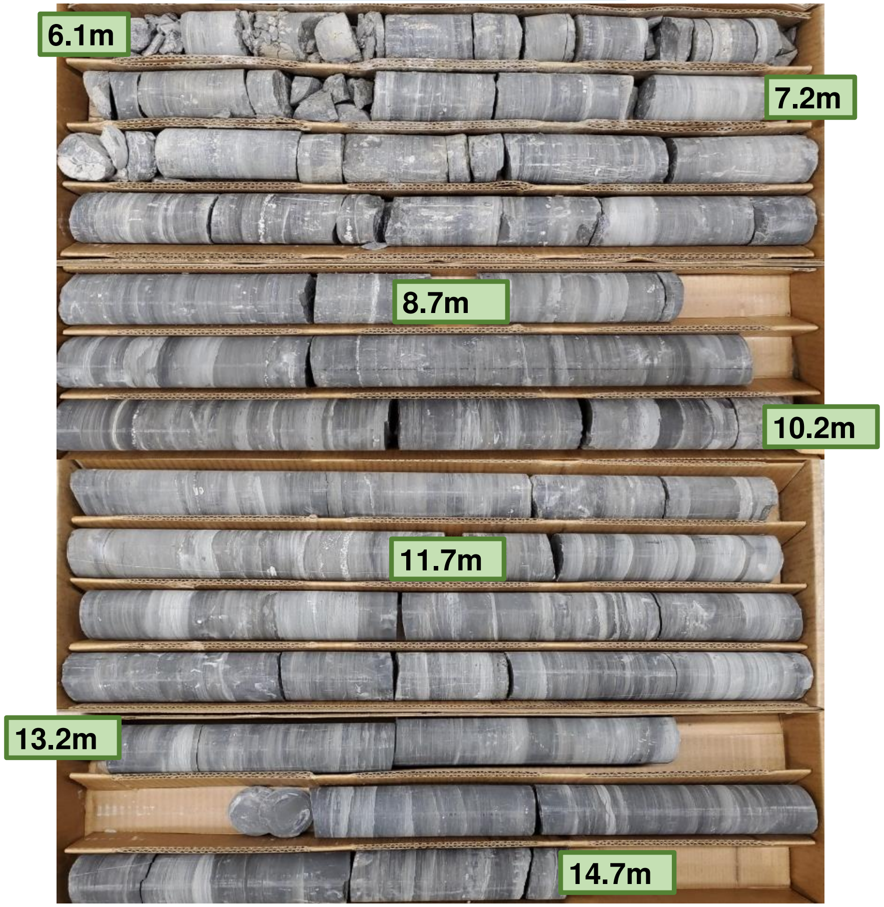


WET BEDROCK CORES



borehole no. **BH20-5**	core runs Run 1: 7.7 m - 8.6 m Run 2: 8.6 m - 10.1 m Run 3:10.1 m - 11.6 m Run 4: 11.6 m - 13.2 m Run 5: 13.2 m - 14.7 m Run 6: 14.7 m - 16.1 m Run 7: 16.1 m - 17.6 m	PROJECT Residential Development, 1740-1760 St. Laurent Boulevard, Ottawa, Ontario	project no. OTT-00260579-A0
date cored Aug 21, 2020		ROCK CORE PHOTOGRAPHS	FIG. B-1

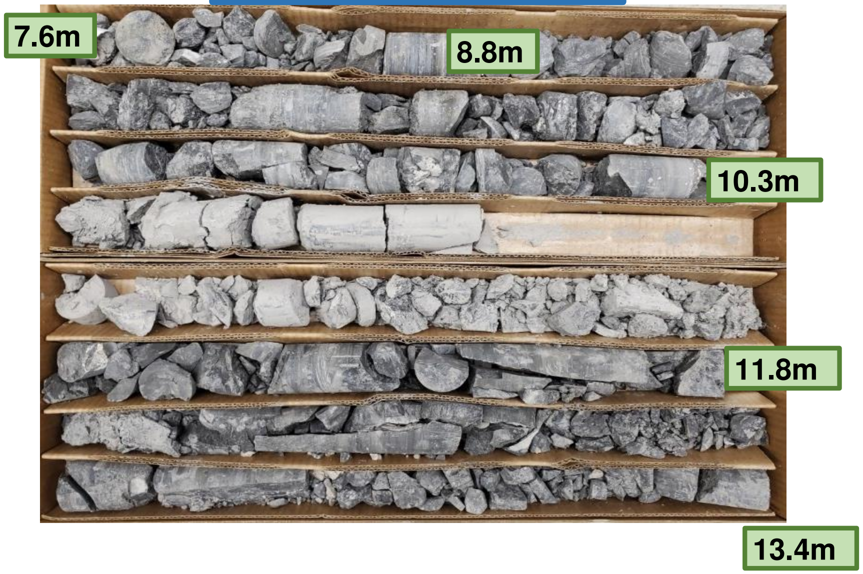
DRY BEDROCK CORES



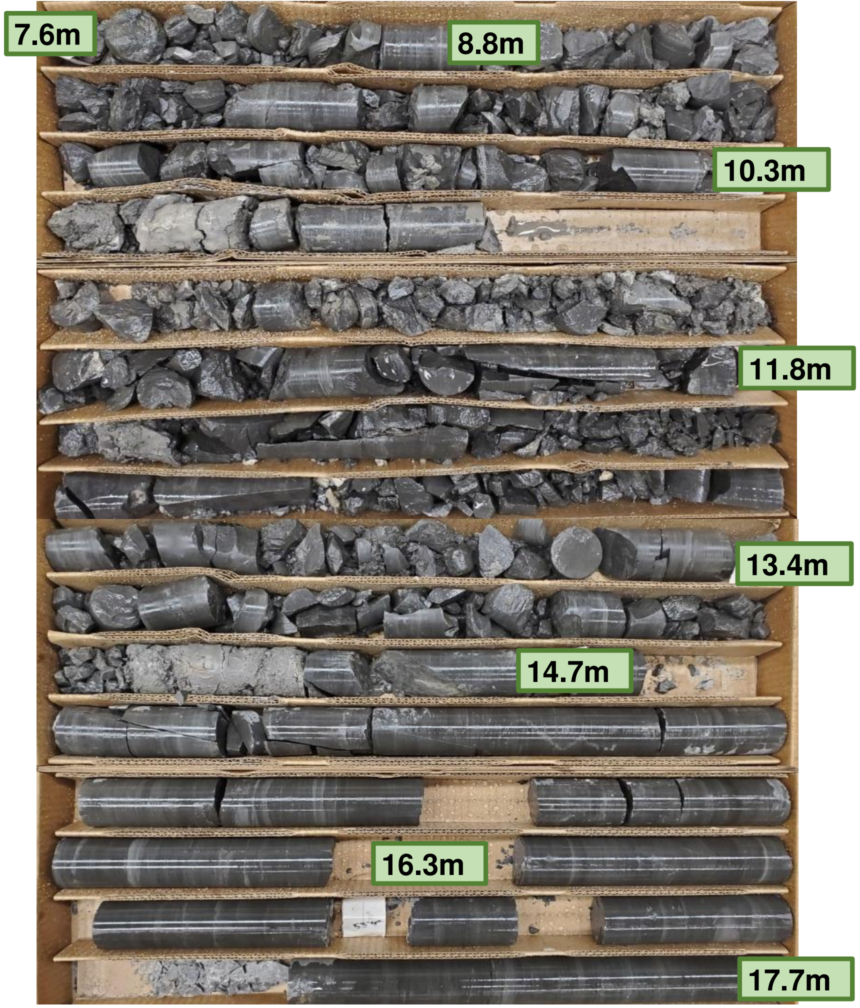
WET BEDROCK CORES

borehole no. **BH20-10**	core runs Run 1: 6.1 m - 7.2 m Run 2: 7.2 m - 8.7 m Run 3: 8.7 m - 10.2 m Run 4: 10.2 m - 11.7 m Run 5: 11.7 m - 13.2 m Run 6: 13.2 m - 14.7 m	PROJECT Residential Development, 1740-1760 St. Laurent Boulevard, Ottawa, Ontario	project no. OTT-00260579-A0
date cored Aug 24, 2020		ROCK CORE PHOTOGRAPHS	FIG. B-2

DRY BEDROCK CORES



WET BEDROCK CORES



borehole no.

BH20-13

date cored

Aug 25, 2020

core runs

Run 1: 7.6 m - 8.8 m
Run 2: 8.8 m - 10.3 m
Run 3: 10.3 m - 11.8 m
Run 4: 11.8 m - 13.4 m
Run 5: 13.4 m - 14.7 m
Run 6: 14.7 m - 16.3 m
Run 7: 16.3 m - 17.7 m

PROJECT

Residential Development, 1740-1760 St. Laurent Boulevard, Ottawa, Ontario

ROCK CORE PHOTOGRAPHS

project no.

OTT-00260579-A0

FIG. B-3

Appendix C – Shear Wave Velocity Sounding Survey Report

GEOPHYSICS GPR INTERNATIONAL INC.

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January 15th, 2021

Transmitted by email: ismail.taki@exp.com
Our Ref.: GPR-20-02643a

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

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
1740 – 1760, St. Laurent Boulevard, Ottawa (ON)

[Project: OTT-00260579-A0]

Dear Sir,

Geophysics GPR International inc. has been mandated by **exp** Services inc. to carry out seismic shear wave surveys on a property located between 1740 and 1760 St. Laurent Boulevard, 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 December 17th, 2020, by Mrs. Karyne Faguy, B.Sc. geoph. and Mr. Emmanuel Truchot, tech. 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 tables and graphs.

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 waves ("ground roll"). The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones' spread axis. Conversely, the 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 allows deeper Vs soundings, but generally with a lower resolution for the surface portion. 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 phase velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding).

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

INTERPRETATION

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

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

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

SURVEY DESIGN

The seismic acquisition spreads were laid out on a grass strip separating two parking lots, with basic geophone spacing of 3 metres for the main spread, using 24 geophones (Figure 2). A shorter seismic spread, with geophone spacing of 1.0 metre, was dedicated to the near surface materials.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and 50 µs for the seismic refraction. The records included a pre-triggering portion of 10 ms. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

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

The seismic records were produced with a seismograph Terraloc MK6 (from ABEM Instrument), and the geophones were 4.5 Hz. An 8 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic spreads.

RESULTS

From seismic refraction (V_P), the rock depth was calculated between 8.6 and 10.5 metres (± 1 metre). The V_S for the upper portion of the sound rock was evaluated between 1960 and 2085 m/s. These results were used as initial parameters for the basic geophysical model, prior to the MASW dispersion curves modeling and inversions.

The MASW calculated V_S results are illustrated at Figure 5.

The $\bar{V}_{S30}$ 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 848.2 m/s (cf. 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 unconsolidated materials between the rock surface and the bottom of the foundation.

In the case there would be 2.6 metres or less of unconsolidated materials between the rock surface and the bottom of the foundation, the minimal $\overline{V}_{S30}$* value would be greater than 1500 m/s, allowing to use the Site Class "A" (cf. Table 2).

CONCLUSION

Geophysical surveys were carried out on a property located between 1740 and 1760 St. Laurent Boulevard, in Ottawa (ON), to identify the Site Class. The seismic surveys used the MASW and the SPAC analysis, as well as the seismic refraction method to calculate the $\overline{V}_{S30}$ value. Its calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 848 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, and the Building Code, O. Reg. 332/12. It must be noted that Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated materials between the rock surface and the bottom of the spread footing or mat foundation.

In the case there would be 2.6 metres or less of unconsolidated materials between the rock and the bottom of the foundation, the minimal $\overline{V}_{S30}$* value would be greater than 1500 m/s, allowing to use the Site Class “A”. As the footings are planned to be on sound rock, 9 to 10 metres below grade, the Site Class “A” could be used.

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

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

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

P.Eng.

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

J.-L. ARSENAULT
100148122
2021-01-15

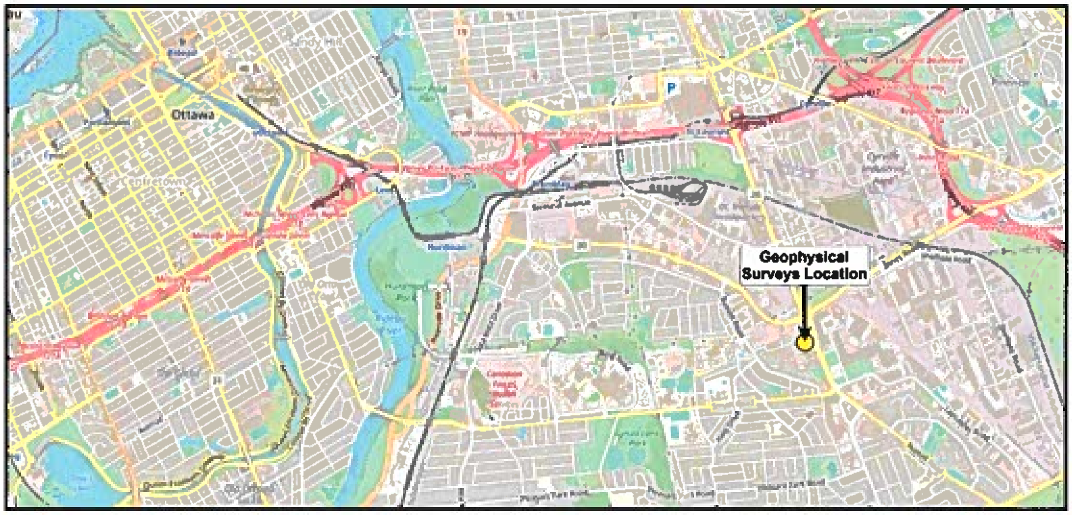


Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

Figure 2: Location of the seismic spreads
(source: *Google Earth™*

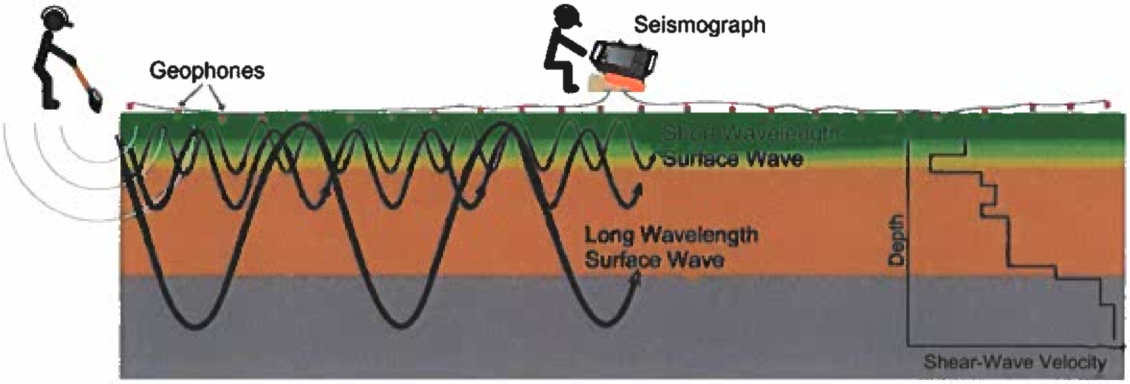
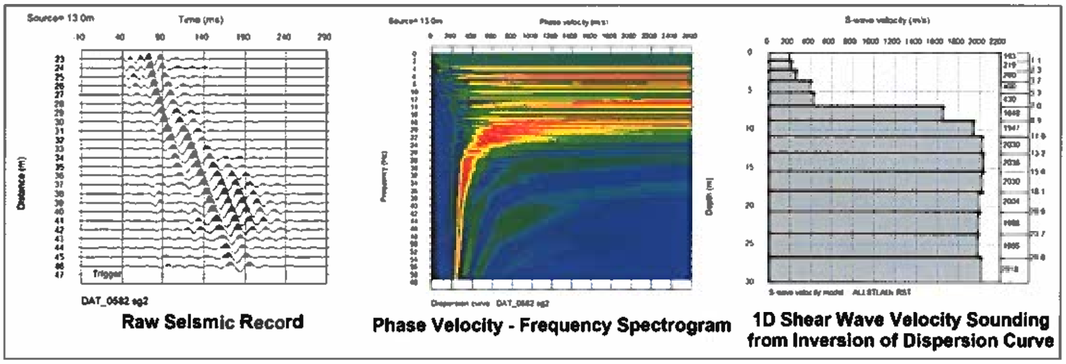


Figure 3: MASW Operating Principle



Raw Seismic Record

Phase Velocity - Frequency Spectrogram

1D Shear Wave Velocity Sounding from Inversion of Dispersion Curve

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

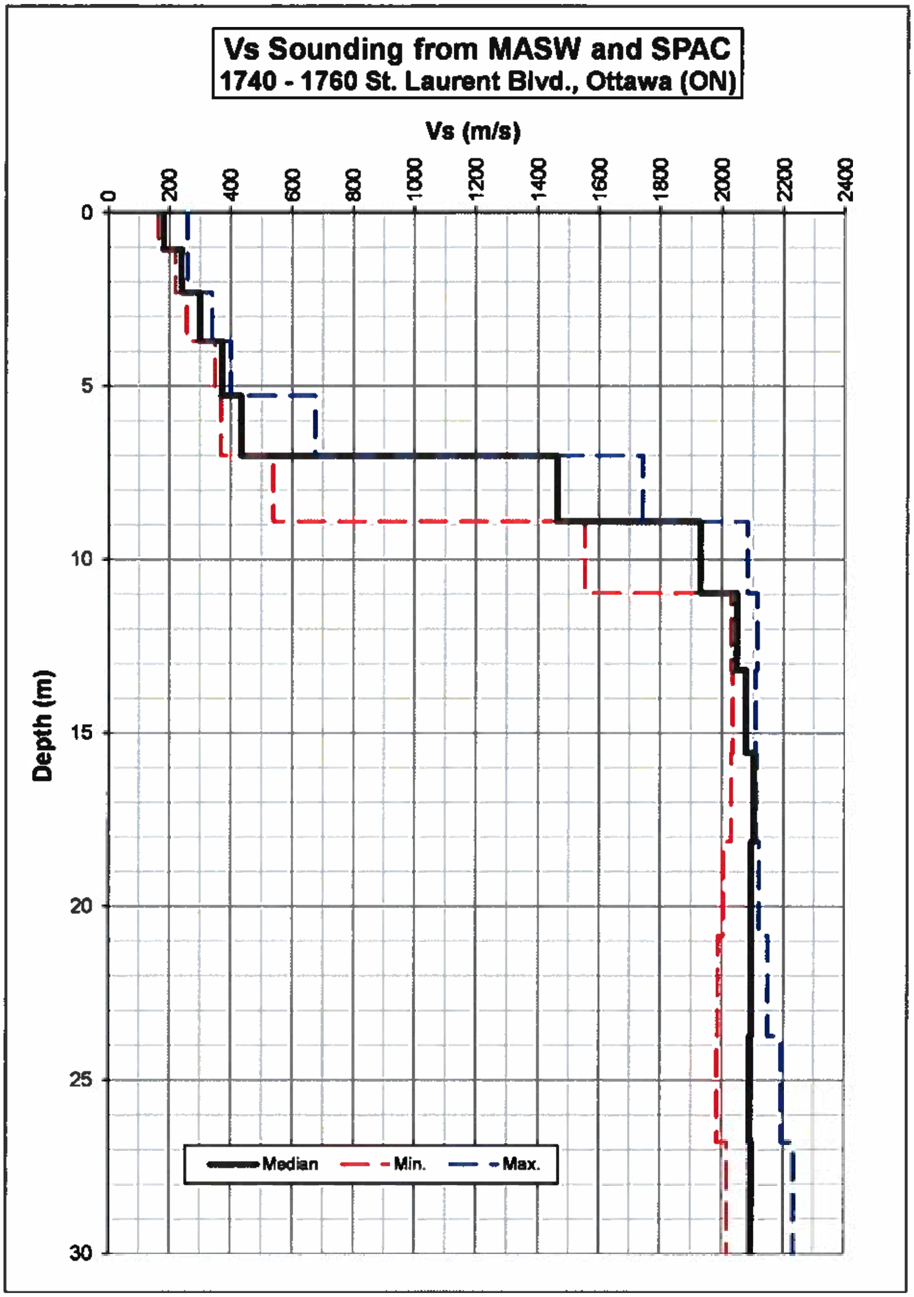


Figure 5: MASW Shear-Wave Velocity Sounding

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	165.2	182.7	258.7	**Grade Level during Seismic Surveys (December 17, 2020)**				
1.07	219.9	240.2	258.8	1.07	1.07	0.005863	0.005863	182.7
2.31	256.5	299.0	339.2	1.24	2.31	0.005146	0.011009	209.6
3.71	348.7	372.3	400.7	1.40	3.71	0.004686	0.015695	236.3
5.27	369.2	434.5	674.4	1.57	5.27	0.004206	0.019901	265.0
7.01	538.1	1463.5	1743.5	1.73	7.01	0.003984	0.023885	293.3
8.90	1554.6	1929.8	2082.9	1.90	8.90	0.001295	0.025180	353.5
10.96	2030.8	2047.7	2114.6	2.06	10.96	0.001068	0.026248	417.6
13.19	2036.1	2077.4	2109.9	2.23	13.19	0.001087	0.027334	482.4
15.58	2030.8	2102.3	2111.8	2.39	15.58	0.001151	0.028485	546.9
18.13	2004.6	2095.1	2120.3	2.55	18.13	0.001215	0.029700	610.5
20.85	1988.3	2094.6	2149.2	2.72	20.85	0.001298	0.030998	672.7
23.74	1985.0	2090.1	2192.3	2.88	23.74	0.001377	0.032375	733.2
26.79	2018.3	2094.4	2233.7	3.05	26.79	0.001459	0.033834	791.7
30				3.21	30.00	0.001535	0.035369	848.2

V_{S30} (m/s)	**848.2**
Class	**B (1)**

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

TABLE 2

V_{S30}* Calculation for the Site Class (soil thickness limit for 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	165.2	182.7	258.7	**Limit for Site Class A (2.6 metres of unconsolidated material)**				
1.07	219.9	240.2	258.8					
2.31	256.5	299.0	339.2					
3.71	348.7	372.3	400.7					
4.40	348.7	372.3	400.7					
5.27	369.2	434.5	674.4	0.87	0.87	0.002349	0.002349	372.3
7.01	538.1	1463.5	1743.5	1.73	2.61	0.003984	0.006333	411.4
8.90	1554.6	1929.8	2082.9	1.90	4.50	0.001295	0.007628	590.1
10.96	2030.8	2047.7	2114.6	2.06	6.56	0.001068	0.008696	754.6
13.19	2036.1	2077.4	2109.9	2.23	8.79	0.001087	0.009782	898.2
15.58	2030.8	2102.3	2111.8	2.39	11.18	0.001151	0.010933	1022.3
18.13	2004.6	2095.1	2120.3	2.55	13.73	0.001215	0.012148	1130.4
20.85	1988.3	2094.6	2149.2	2.72	16.45	0.001298	0.013446	1223.5
23.74	1985.0	2090.1	2192.3	2.88	19.34	0.001377	0.014824	1304.4
26.79	2018.3	2094.4	2233.7	3.05	22.39	0.001459	0.016283	1374.8
34.40				7.61	30.00	0.003636	0.019918	1506.2

V_{S30}* (m/s)	**1506.2**
Class	**A**

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