Updated Geotechnical Investigation

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Executive Summary

EXP Services Inc. (EXP) is pleased to present the updated geotechnical investigation report completed for the proposed development to be located at 780 Baseline Road, Ottawa, Ontario (Figure 1). Terms and conditions of this assignment were outlined in EXP's proposal number OTT-22005690-AB dated March 15, 2022. Authorization to proceed with this geotechnical investigation was provided by 780 Baseline Inc.

As part of this assignment, from 2021 to 2023 EXP has completed Phase One and Two Environmental Site Assessments (ESAs) and a hydrogeological investigation titled Groundwater Impact Assessment (GIA) for the entire site. The results of these assessments are provided in separate reports.

It is our understanding that current plans call for only the south portion of the site to be developed and this stage is identified as the Phase I development. The rest of the site is occupied by the existing single-storey commercial plaza building and will remain unchanged during Phase 1.

Based on drawings C101 to C103 and C201 to C203 prepared by McIntosh Perry and dated September 25, 2023, the Phase I development will include the construction of two (2) buildings in the south portion of the site. These buildings will include a twenty-four (24) storey mixed-use apartment building and a four (4) storey podium, each with a design finished floor elevation (FFE) of Elevation 84.55 m. Based on email correspondence, it is understood that four (4) storeys of underground parking are to be constructed with the lowest floor slab at a minimum 12.0 m depth (Elevation 72.50 m) and footings founded approximately 1.0 m below the lowest floor slab; approximately 13.0 m depth (Elevation 71.50 m).

The drawings also indicate that an existing sanitary sewer located within the footprint of the proposed buildings is to be removed (including the manhole structures) and relocated south of the proposed buildings, within the City of Ottawa easement. The City of Ottawa easement will have a minimum width of 6.0 m. The drawings indicate the proposed replacement sanitary sewer will be a 375 mm diameter PVC pipe with inverts ranging from Elevation 80.94 m to Elevation 80.64 m.

It is understood that no significant grade raise is proposed at the site.

The borehole fieldwork for the entire site consists of six (6) boreholes (Borehole Nos. 1 to 6) undertaken from April 11 to 18, 2022. The boreholes were advanced to auger refusal and termination depths ranging from 12.2 m to 19.2 m below the existing grade. The locations and geodetic elevations of the boreholes were established by a survey crew from EXP and are shown on the borehole location plan, Figure 2. The borehole locations were selected based on the extent of the proposed development at the time of the geotechnical investigation. Borehole Nos. 5 and 6 are within the proposed footprint of the Phase I development located in the south portion of the site.

Based on the borehole information, the subsurface conditions at the entire site consist of fill underlain by clay, silty clay and silt followed by glacial till and limestone bedrock. The bedrock was contacted at 10.8 m to 15.7 m depths (Elevation 73.3 m to Elevation 68.7 m). The groundwater level ranges from 3.4 m to 5.4 m depths (Elevation 81.0 m to Elevation 78.7 m).

The seismic shear wave velocity sounding survey report is shown in Appendix A. The results of the survey indicate that the average seismic shear wave velocity is 1510.9 m/s for footings or for a mat foundation founded on the sound limestone bedrock as discussed in Section 8 of this report. This will result in a Class A site class for seismic site response in accordance with Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC), as amended January 1,2022.

Since the construction of the four (4) level underground parking garage would require the excavation and removal of all soils down to the bedrock, the presence of liquefiable soils at the site is not an issue for the proposed development.

The 2023 hydrogeological report titled Groundwater Impact Assessment (GIA) was completed for the entire site. It is recommended that the hydrogeological report be updated to include only the south portion of the site for the Phase I development. Based on the findings from the updated GIA report, the comments and recommendations provided in this geotechnical report may need to be revised.

If the updated hydrogeological report confirms that the short-term and long-term groundwater lowering of the south portion of the site will negatively impact the adjacent existing buildings (such as the commercial plaza and residential buildings) and existing infrastructure (including underground utilities and services), then the proposed buildings should be designed as a water-tight structure and the foundation will consist of a mat foundation. If the updated hydrogeological report confirms that the short-term and long-term groundwater lowering of the south portion of the site will not negatively impact the adjacent existing buildings and existing infrastructure, then the proposed buildings may be designed as a drained structure and the proposed buildings may be supported by footings with the lowest floor slab designed as a slab-on-grade with permanent perimeter and underfloor drainage systems. In either case, the excavation for the proposed buildings undertaken within the confines of a secant pile wall with tie-backs consisting of grouted rock anchors.

For a drained structure, the proposed buildings may be supported by strip and spread footings founded on the competent sound limestone bedrock, free of weathered zones, loose material, soft seams, fractures and voids may be designed for a factored geotechnical resistance at ultimate limit state (ULS) of 4,000 kPa. Similarly, for a water-tight structure, the proposed buildings may be supported by a mat foundation founded on the competent sound limestone bedrock, free of weathered zones, loose material, soft seams, fractures and voids may be designed for a factored geotechnical resistance at ultimate limit state (ULS) of 4,000 kPa. The factored geotechnical resistance at ULS in both cases includes a geotechnical resistance factor of 0.50. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement will be much larger than the recommended values for the factored geotechnical resistance at ULS. Therefore, for footings founded on sound bedrock, the factored geotechnical resistance at ULS will govern the design.

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

Post-tensioned rock anchors installed in the bedrock may be required as part of the footing design to resist uplift forces.

The lowest floor level of the parking garage for the proposed buildings is anticipated to be at a 12.0 m depth (Elevation 72.5 m) below existing grade. For the drained structure, the lowest floor slab of the parking garage may be designed as a slab-on-grade and the surface of the floor slab may consist of a concrete surface or a paved surface. Based on the borehole information, the lowest floor slab of the buildings for a drained structure will be founded on the dense to very dense glacial till. Additional comment will be provided should water-tight structure be selected for the proposed buildings.

An existing 375 mm sanitary sewer currently crosses the middle of the Phase I development in an east-west direction and is to be relocated within the City of Ottawa easement to the south of the proposed buildings, as shown in Figure 2. The relocated sanitary sewer will be set at invert elevations ranging from Elevation 80.94 m to Elevation 80.64 m, i.e below the ground surface and will be situated at a distance of 2.5 m to 3.5 from the North side of the residences located just south of the proposed buildings. The City of Ottawa easement will have a minimum width of 6.0 m, and the width may be increased to for allow future access for repairs and maintenance.

Excavations for the installation of the relocated sanitary sewer must be undertaken within an engineered support system, such as a trench box or a sheet pile support system. The engineered support system is to be designed by a professional engineer retained by the contractor specifically for this project and designed in accordance with OHSA 213/91 and the recommendations in this report. The shoring system must be designed in such a way to eliminate any movement of soil behind the support system/trench box which will prevent any negative impact on the existing residences or infrastructures situated within the zone of influence of the proposed work. Alternatively, consideration can be given to explore the feasibility of the installation using directional drilling. For this purpose, a specialized contractor should be consulted to establish the feasibility of this option.

It is recommended that additional boreholes be undertaken within the footprint of the proposed buildings in the area of the Phase I development to better delineate the bedrock depth (elevation) and the geotechnical engineering properties of the bedrock. It is also recommended that the GIA report be updated to include only the south portion of the site for the Phase I development. Based on the findings from the updated GIA report, the comments and recommendations provided in this update geotechnical report may need to be revised.

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 updated geotechnical investigation report completed for the proposed development to be located at 780 Baseline Road, Ottawa, Ontario (Figure 1). Terms and conditions of this assignment were outlined in EXP's proposal number OTT-22005690-AB dated March 15, 2022. Authorization to proceed with this geotechnical investigation was provided by 780 Baseline Inc.

As part of this assignment, from 2021 to 2023 EXP has completed Phase One and Two Environmental Site Assessments (ESAs) and a hydrogeological investigation titled Groundwater Impact Assessment (GIA) for the entire site. The results of these assessments are provided in separate reports.

It is our understanding that current plans call for only the south portion of the site to be developed and this stage is identified as the Phase I development. The rest of the site is occupied by the existing single-storey commercial plaza building and will remain unchanged during Phase 1.

Based on drawings C101 to C103 and C201 to C203 prepared by McIntosh Perry and dated September 25, 2023, the Phase I development will include the construction of two (2) buildings in the south portion of the site. These buildings will include a twenty-four (24) storey mixed-use apartment building and a four (4) storey podium, each with a design finished floor elevation (FFE) of Elevation 84.55 m. Based on email correspondence, it is understood that four (4) storeys of underground parking are to be constructed with the lowest floor slab at a minimum 12.0 m depth (Elevation 72.50 m) and footings founded approximately 1.0 m below the lowest floor slab; approximately 13.0 m depth (Elevation 71.50 m).

The drawings also indicate that an existing sanitary sewer located within the footprint of the proposed buildings is to be removed (including the manhole structures) and relocated south of the proposed buildings, within the City of Ottawa easement. The City of Ottawa easement will have a minimum width of 6.0 m. The drawings indicate the proposed replacement sanitary sewer will be a 375 mm diameter PVC pipe with inverts ranging from Elevation 80.94 m to Elevation 80.64 m.

It is understood that no significant grade raise is proposed at the site.

This updated geotechnical report provides the borehole information for the entire site and geotechnical engineering comments and recommendations only for the Phase I development located in the south portion of the site.

This geotechnical investigation was undertaken to:

a) Establish the subsurface soil, bedrock and groundwater conditions at the six (6) boreholes located on the entire site,

b) Classify the site for seismic site response in accordance with the requirements of the 2012 Ontario Building Code (as amended January 1, 2022) and assess the potential for liquefaction of the subsurface soils during a seismic event,

c) Comment on grade-raise restrictions,

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

e) Provide lateral earth pressure (force) against subsurface (basement) walls for the static and seismic (dynamic) conditions,

f) Slab on grade construction,

g) Anticipated excavation conditions and de-watering requirements during construction and potential impact on neighbouring properties and infrastructure,

h) Discuss excavation for the relocation of the 375 mm sanitary sewer,

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

j) Subsurface concrete requirements; and

k) Discuss Tree Planting Requirement

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

2. Site Description

The entire site is an L-shaped corner property located in the southwest corner of the Fisher Avenue and Baseline Road intersection in Ottawa, Ontario. The site is bounded by Baseline Road and the Central Experimental Farm to the north, residential dwellings to the west and south and by Fisher Avenue and residential dwellings to the east.

The property is approximately 14,290 square metres (m^2) in size and at the time of this updated geotechnical report is occupied by a single-storey commercial plaza surrounded by an outdoor paved parking lot.

The topography of the site is relatively flat with the elevation of the ground surface at the boreholes located on the site ranging from Elevation 84.42 m to Elevation 83.99 m.

3. Geology of the Site

3.1 Surficial Geology

The surficial geology map (Map 1506A – Surficial Geology, Ontario-Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1982) indicates that beneath any fill, the site is underlain by native clay and silt overlying erosional terraces. The upper part of the marine deposit has been removed to various depths by fluvial erosion. The unit includes lenses, bars and sand-filled channels and pockets of non-marine silt that were formed during channel cutting.

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

4. Procedure

4.1 Fieldwork

The borehole fieldwork for the entire site consists of six (6) boreholes (Borehole Nos. 1 to 6) undertaken from April 11 to 18, 2022. The boreholes were advanced to auger refusal and termination depths ranging from 12.2 m to 19.2 m below the existing grade. The borehole fieldwork was supervised on a full-time basis by EXP.

The locations and geodetic elevations of the boreholes were established by a survey crew from EXP and are shown on the borehole location plan, Figure 2. The borehole locations were selected based on the extent of the proposed development at the time of the geotechnical investigation. Borehole Nos. 5 and 6 are within the proposed footprint of the Phase I development located in the south portion of the site.

Prior to the fieldwork, the locations of the boreholes were cleared of any public and private underground services. The boreholes were drilled using a CME-55 truck-mounted drill rig equipped with continuous flight hollow-stem auger equipment and bedrock coring capabilities and operated by a drilling contractor subcontracted to EXP. Standard penetration tests (SPTs) were performed in all the boreholes on a continuous basis (at localized depths) to a 1.5 m depth interval and the soil samples were retrieved by the split-spoon sampler. An auger sample was obtained in the Borehole No. 1 from just below the asphaltic concrete to 0.7 m depth. The undrained shear strength of the cohesive soil was measured by conducting in-situ vane test at selected depths. The presence of the bedrock was proven in four (4) boreholes by conventional coring techniques using the NQ size core barrel. A field record of wash water return, colour of wash water and any sudden drops of the core barrel were kept during coring operations.

Monitoring wells (38 mm or 50 mm diameters) were installed in all six (6) boreholes for long-term monitoring of the groundwater level and for the sampling of the groundwater as part of the Phase Two ESAs and the GIA. The monitoring wells were installed in accordance with EXP standard practice, and the installation configuration is documented on the respective borehole logs. The boreholes were backfilled upon completion of the field work and the installation of the monitoring wells.

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

4.2 Laboratory Testing Program

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

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

Table I: Summary of Laboratory Testing Program

Type of Test	Number of Tests Completed
Soil Samples	
Moisture Content Determination	58
Unit Weight Determination	6
Grain Size Analysis	8
Atterberg Limit Determination	5
Chemical Test for Corrosion Potential (pH, sulphate, chloride and resistivity)	1
Bedrock Cores	
Unit Weight Determination and Unconfined Compressive Strength Test	4
Chemical test for Corrosion Potential (pH, sulphate, chloride and resistivity)	1

4.3 Seismic Shear Wave Velocity Sounding Survey

A seismic shear wave velocity sounding survey was conducted at the site on May 19, 2022, by Geophysics GPR International Inc. (GPR). The survey line is located along the north side of the site. The survey was undertaken using the multi-channel analysis of surface waves (MASW), spatial auto correlation (SPAC) and seismic refraction methods. The results of the survey are provided in the June 8,2022 GPR report shown in Appendix A.

5. Subsurface Conditions and Groundwater Levels

The location of the boreholes for the entire site are shown in Figure 2. A cross-section (profile) of the subsurface conditions and groundwater level measurements for the entire site is shown in Figure 3 (Section A-A') with the location of the section shown on the borehole location plan in Figure 2.

A detailed description of the subsurface conditions and groundwater levels from the boreholes for the entire site are given on the attached Borehole Logs, Figures 4 to 9. 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 a representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions. Reference is made to the Phase One and Two ESAs and the GIA reports regarding potential environmental conditions of the entire site.

It should be noted that the soil and rock 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 updated geotechnical report.

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

Borehole Nos, 5 and 6 are situated within the footprint of the proposed buildings of the Phase I development located in the south portion of the site.

5.1 Asphaltic Concrete

The boreholes are located within paved areas. A 60 mm to 80 mm thick asphaltic concrete layer was contacted at ground surface of all six (6) boreholes.

5.2 Fill

The asphaltic concrete is underlain by fill that extends to depths of 0.8 m to 1.4 m below the existing ground surface (Elevation 83.5 m to Elevation 82.8 m). The fill consists of sand and gravel with a variable amount of silt. The standard penetration test (SPT) N-values range from 7 to 32 indicating the fill is in a loose to dense state. The moisture content of the fill ranges from 4 percent to 10 percent.

The results from the grain-size analysis conducted on one (1) sample of the fill is summarized in Table II. The grain-size distribution curve is shown in Figure 10.

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

Borehole No. (BH) – Sample No. (AS)	Depth (m)	Grain-Size Analysis (%)			Soil Classification (USCS)
		Gravel	Sand	Fines (Silt and Clay)	
BH 1-AS1	0.1-0.7	45	40	15	Silty Gravel with Sand (GM)

Based on a review of the results from the grain size analysis of one (1) sample, the fill may be classified as silty gravel with sand (GM) in accordance with the USCS.

5.3 Clay

Native clay was encountered below the fill in all the boreholes. The clay extends to depths of 2.7 m to 4.3 m (Elevation 81.7 m to Elevation 79.9 m). The undrained shear strength of the clay ranges from 110 kPa to greater than 250 kPa indicating the clay has a very stiff to hard consistency. The natural moisture content and unit weight of the clay ranges from 25 percent to 44 percent and 17.7 kN/m^3 to 20.7 kN/m^3, respectively.

The results from the grain-size analysis and Atterberg limit determination conducted on one (1) sample of the clay are summarized in Table III. The grain-size distribution curve is shown in Figure No. 11.

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

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)				Atterberg Limits (%)				Soil Classification (USCS)
		Gravel	**Sand**	**Silt**	**Clay**	**Moisture Content**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	
BH 2-SS3	1.5-2.1	0	3	30	67	44	42	21	21	Clay of High Plasticity (CH)

Based on a review of the results of the grain-size analysis and Atterberg limits, the soil may be classified as a clay of high plasticity (CH) in accordance with the USCS.

5.4 Silty Clay

Underlying the clay, silty clay was encountered in all the boreholes. The silty clay extends to depths of 7.3 m to 7.9 m (Elevation 77.1 m to Elevation 76.4 m). The undrained shear strength of the silty clay ranges from 34 kPa to 96 kPa indicating the silty clay has a firm to stiff consistency. The natural moisture content of the silty clay ranges from 53 percent to 70 percent.

The results from the grain-size analysis and Atterberg limit determination conducted on three (3) samples of the silty clay are summarized in Table IV. The grain-size distribution curves are shown in Figures 12 to 14.

Table IV: 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 (%)				Soil Classification (USCS)
		Gravel	**Sand**	**Silt**	**Clay**	**Moisture Content**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	
BH 1-SS5	4.6-5.2	0	0	46	54	70	47	22	25	Silty Clay of Low Plasticity (CL)
BH 3-SS4	3.0-3.7	0	20	35	45	36	42	17	25	Silty Clay with Sand of Low Plasticity (CL)
BH 4-SS6	6.1-6.7	0	1	49	50	56	61	27	34	Silty Clay of Low Plasticity (CL)

Based on a review of the results of the grain-size analysis and Atterberg limits the soil may be classified as a silty clay of low plasticity (CL) with varying amounts of sand in accordance with the USCS.

5.5 Silt

The silty clay is underlain by silt that extends to depths of 8.7 m to 10.2 m (Elevation 75.7 m to Elevation 73.8 m). In Borehole Nos. 1 to 4 and 6, the silt exhibits a slight plasticity and has undrained shear strengths ranging from 53 kPa to 139 kPa indicating the silt has a stiff to very stiff consistency. The silt in Borehole No. 5 is non-plastic and based on SPT N-values of zero (hammer weight) and 1, the silt is in a very loose state. The natural moisture content of the silt ranges from 13 percent to 43 percent.

The results from the grain-size analysis and Atterberg limit determination conducted on one (1) sample of the silt are summarized in Table V. The grain-size distribution curve is shown in Figure 15.

Table V: Summary of Results from Grain-Size Analysis and Atterberg Limit Determination – Silt Sample

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)				Atterberg Limits (%)				Soil Classification (USCS)
		Gravel	**Sand**	**Silt**	**Clay**	**Moisture Content**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	
BH 5-SS7	7.6-8.2	0	2	80	18	35	N.P.			Silt (ML)

- N.P. = Non-plastic

Based on a review of the results of the grain-size analysis and Atterberg limits, the soil may be classified as a non-plastic silt (ML) in accordance with the USCS.

5.6 Glacial Till

The silt is underlain by a glacial till contacted at 8.1 m to 10.2 m depths (Elevation 75.7 m to Elevation 73.8 m) in all the boreholes. The glacial till consists of silty sand with gravel and contains shale fragments, cobbles and boulders. Based on the SPT N-values that range from 8 to 79, the glacial till is in a loose to very dense state. High SPT N-values for low sampler penetration, such as 50 for 125 mm sampler penetration were recorded and may be a result of the sampler resting on a cobble or boulder within the glacial till. Based on the observation of augers grinding and that coring had to be used to advance Borehole Nos. 1, 4 and 6 through the glacial till, it appears the glacial till from 9.1 m to 10.7 m depths (Elevation 75.1 m to Elevation 73.7 m) contains numerous cobbles and boulders. The natural moisture content of the glacial till ranges from 6 percent to 13 percent.

The results from the grain-size analysis conducted on two (2) sample of the glacial till are summarized in Table VI. The grain-size distribution curves are shown in Figures 16 and 17.

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

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)					Soil Classification (USCS)
		Gravel	Sand	**Silt**	**Clay**	Moisture Content	
BH 6-SS9	10.7-11.3	19	52	20	9	6	Silty Sand with Gravel (SM)
BH 1-SS10	12.2-12.8	27	53	14	6	7	Silty Sand with Gravel (SM)

Based on a review of the results of the grain-size analysis of the two (2) samples, the glacial till may be classified as a silty sand with gravel (SM) in accordance with the USCS. As previously mentioned, the glacial till contains shale fragments, cobbles and boulders.

5.7 Limestone Bedrock

Auger refusal was encountered in Borehole Nos. 2 and 5 at 13.7 m (Elevation 70.5 m) and 12.2 m depths (Elevation 71.8 m), respectively, and may possibly represent cobbles or boulders within the glacial till or the bedrock surface.

The presence of the bedrock was proven in Borehole Nos. 1, 3, 4 and 6 by coring the bedrock. Based on a review of the bedrock cores, the bedrock is considered to be limestone with shaley partings. Photographs of the bedrock cores are shown in Appendix B. A summary of the possible and actual bedrock depths (elevations) is shown in Table VII.

Table VII: Summary of Bedrock Depths (Elevations)

Borehole (BH) No.	Ground Surface Elevation (m)	Bedrock Depth (Elevation) m
BH-1	84.42	15.7 (68.7)
BH-2	84.21	13.7 (70.5) – Possible Bedrock
BH-3	84.05	10.8 (73.2)
BH-4	84.33	14.2 (70.1)
BH-5	83.99	12.2 (71.8) – Possible Bedrock
BH-6	84.18	13.7 (70.5)

Based on the bedrock coring results, the total core recovery (TCR) ranges from 80 percent to 100 percent. The rock quality designation (RQD) ranges from 0 percent to 86 percent indicating the bedrock quality is very poor to good. The test results are presented below in Table VIII.

Table VIII: Summary of RQD and TCR Values of Bedrock Cores

Run No.	**Depth (m)**	**Rock Quality Designation RQD (%)**	**Total Core Recovery TCR (%)**
Borehole No. 1			
2	15.7 - 16.3	0	100
3	16.3 - 17.7	27	100
4	17.7 - 19.2	61	100
Borehole No. 3			
1	10.8 – 11.6	47	100
2	11.6 – 13.2	42	80
Borehole No. 4			
4	14.2 - 14.6	86	100
5	14.6 - 16.2	29	85
Borehole No. 6			
2	13.7 - 15.2	23	80
3	15.2 - 16.6	44	91

Unit weight determination and unconfined compressive strength tests were conducted on four (4) rock core sections and the results are summarized in Table IX.

Table IX: 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-1	16.7 - 16.8	26.6	209	R5
BH-3	11.8 - 11.9	26.3	195	R5
BH-4	14.2 - 14.3	26.6	197	R5
BH-6	14.0 - 14.1	27.0	226	R5

A review of the test results in Table IX indicates the strength of the rock may be classified as very strong (R5) in accordance with the Canadian Foundation Engineering Manual (CFEM), Fourth Edition, 2006.

Groundwater Level Measurements

A total of six (6) monitoring wells were installed at the site. A summary of the groundwater level measurements taken in the monitoring wells are shown in Table X.

Table X: 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	Date of Measurement (Elapsed Time in Days from Date of Installation)	Groundwater Depth Below Ground Surface (Elevation), m	Date of Measurement (Elapsed Time in Days from Date of Installation)	Groundwater Depth Below Ground Surface (Elevation), m
BH-1	84.42	April 27, 2022 (12)	3.4 (81.0)	June 23, 2022 (70)	3.6 (80.8)	Sept 8, 2022 (147)	3.8 (80.6)
BH-2	84.21	April 27, 2022 (15)	5.0 (79.2)	June 23, 2022 (73)	5.2 (79.0)	Sept 8, 2022 (150)	5.3 (78.9)
BH-3	84.05	April 27, 2022 (13)	4.9 (79.1)	June 23, 2022 (71)	5.1 (78.9)	Sept 8, 2022 (148)	5.3 (78.7)
BH-4	84.33	April 27, 2022 (14)	5.0 (79.3)	June 23, 2022 (72)	5.2 (79.1)	Sept 8, 2022 (149)	5.4 (78.9)
BH-5	83.99	April 27, 2022 (14)	4.7 (79.3)	June 23, 2022 (72)	4.9 (79.1)	Sept 8, 2022 (149)	5.1 (78.9)
BH-6	84.18	April 27, 2022 (8)	4.6 (79.6)	June 23, 2022 (66)	4.8 (79.4)	Sept 8, 2022 (143)	5.0 (79.2)

The groundwater level ranges from 3.4 m to 5.4 m depths (Elevation 81.0 m to Elevation 78.7 m).

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

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

6.1 Site Classification for Seismic Site Response

The seismic shear wave velocity sounding survey report is shown in Appendix A. The results of the survey indicate that the average seismic shear wave velocity is 1510.9 m/s for footings or for a mat foundation founded on the sound limestone bedrock as discussed in Section 8 of this report. This will result in a Class A site class for seismic site response in accordance with Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC), as amended January 1,2022.

6.2 Liquefaction Potential of Soils

Since the construction of the four (4) level underground parking garage would require the excavation and removal of all soils down to the bedrock, the presence of liquefiable soils at the site is not an issue for the proposed development.

7. Grade Raise Restrictions

Since the site is located in a well-established developed area of the city of Ottawa and the current grades of the site are near those of the adjacent roadways, major grade raise is not anticipated at the site as part of the proposed development. However, for purposes of this geotechnical investigation, a maximum permissible grade raise of 0.5 m may be used for design purposes.

8. Foundation Considerations

As previously mentioned, the geotechnical engineering comments and recommendations for the Phase I development located in the south potion of the site are provided in this section and in the following sections of this updated geotechnical report.

The 2023 hydrogeological report titled Groundwater Impact Assessment (GIA) was completed for the entire site. It is recommended that the hydrogeological report be updated to include only the south portion of the site for the Phase I development. Based on the findings from the updated GIA report, the comments and recommendations provided in this geotechnical report may need to be revised.

If the updated hydrogeological report confirms that the short-term and long-term groundwater lowering of the south portion of the site will negatively impact the adjacent existing buildings (such as the commercial plaza and residential buildings) and existing infrastructure (including underground utilities and services), then the proposed buildings should be designed as a water-tight structure and the excavation for the proposed buildings undertaken within the confines of a secant pile wall with tie-backs consisting of grouted rock anchors. In this case, the foundation will consist of a mat foundation.

If the updated hydrogeological report confirms that the short-term and long-term groundwater lowering of the south portion of the site will not negatively impact the adjacent existing buildings and existing infrastructure, then the proposed buildings may be designed as a drained structure and the proposed buildings may be supported by footings with the lowest floor slab designed as a slab-on-grade. The proposed buildings should have permanent perimeter and underfloor drainage systems. In this case, it is still recommended that the excavation for the proposed buildings be undertaken within the confines of a secant pile wall with tie-backs to support the walls of the excavation and to cut-off the groundwater flows into the excavation.

The foundations and other geotechnical aspects for water-tight and drained structures are discussed in the following sections of this updated report.

8.1 Drained Structure

The borehole information within the proposed Phase I development indicates that bedrock was encountered at 13.7 m depth (Elevation 70.5 m) in Borehole No. 6. Auger refusal was encountered at 12.2 m depth (Elevation 71.8 m) in Borehole No. 5 and the auger refusal may have occurred the bedrock surface or cobbles/boulders within the glacial till. It is understood that the four (4) storeys of underground parking are to be constructed with the lowest floor slab at a minimum 12.0 m depth (Elevation 72.50 m) and footings founded approximately 1.0 m below the lowest floor slab; approximately 13.0 m depth (Elevation 71.50 m).

For a drained structure, the proposed buildings may be supported by strip and spread footings founded on the competent sound limestone bedrock, free of weathered zones, loose material, soft seams, fractures and voids may be designed for a factored geotechnical resistance at ultimate limit state (ULS) of 4,000 kPa. The factored geotechnical resistance at ULS includes a geotechnical resistance factor of 0.50. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement will be much larger than the recommended values for the factored geotechnical resistance at ULS. Therefore, for footings founded on sound bedrock, the factored geotechnical resistance at ULS will govern the design.

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

8.1.1 Sliding Resistance

The factored sliding resistance at ULS between the underside of concrete footing and the top of the un-weathered sound bedrock is 0.56 and includes a resistance factor of 0.8.

8.2 Water-Tight Structure

For a water-tight structure, the proposed buildings may be supported by a mat foundation founded on the competent sound limestone bedrock, free of weathered zones, loose material, soft seams, fractures and voids may be designed for a factored geotechnical resistance at ultimate limit state (ULS) of 4,000 kPa. The factored geotechnical resistance at ULS includes a geotechnical resistance factor of 0.50. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement will be much larger than the recommended values for the factored geotechnical resistance at ULS. Therefore, for footings founded on bedrock, the factored geotechnical resistance at ULS will govern the design.

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

8.3 Additional Comments for Foundations

All footing beds or subgrade for the mat foundation should be examined by a geotechnical engineer/technician to ensure that the founding surfaces are capable of supporting the recommended factored geotechnical resistance at ULS and that the footing beds or subgrade for the mat foundation have been properly prepared. Where fractured bedrock is encountered, sub-excavation will be required down to the competent sound bedrock and the footings will need to be stepped down to the competent sound bedrock. Alternatively, the sub-excavated area may be raised by the placement of 15 MPa lean mix concrete. Also, if the surface of the excavated bedrock is not level, the bedrock surface may be levelled by the placement of concrete.

A minimum of 1.5 m of earth cover should be provided to the exterior foundations 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 their vicinity and to 2.4 m if snow will be removed from the vicinity of the structure. When earth cover is less than the required cover, an equivalent thermal combination of earth cover and rigid insulation or rigid insulation alone should be provided. EXP can provide additional comments in this regard, if required. For the proposed buildings, the footings or mat foundation will have the required earth cover since the foundations are anticipated to be at depths greater than 1.5 m below final grade.

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

8.4 Rock Anchors

Post-tensioned rock anchors installed in the bedrock may be required as part of the footing design to resist uplift forces.

Post-tensioned rock anchors may fail in one or more of the following manners:

a) Failure of the grout/tendon bond,

b) Failure of the steel tendon or top anchorage,

c) Failure of the rock/grout bond; or

d) Pull-out failure of the cone-shaped rock mass.

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

Failure of the rock/grout bond:

- The unfactored ultimate limit state (ULS) bond stress between the sound limestone bedrock and the grout may be taken as 2000 kPa (2.0 MPa). Based on the 2020 National Building Code of Canada (NBCC), for semi-empirical analysis, using a resistance factor of 0.3, the factored ULS bond stress is 600 kPa. The factored ULS bond stress may be taken as 800 kPa and includes a resistance factor of 0.4 based on conducting proof test on all anchors. The unconfined compressive strength of the grout is assumed to be 35 MPa.
- Weathered zones of the bedrock should not be included in the bond length. The depth and presence of the weathered and highly fractured zones of the bedrock may vary at locations away from the boreholes.
- The minimum bonded length should be 3.0 m.
- The unbonded length may be taken as equal to the height of the theoretical rock cone minus half of the bonded length.

Pull-out failure of the cone-shaped rock mass:

- The pull-out failure of the embedment cone-shaped rock mass is defined by a 60 or 90-degree cone in the bedrock with the apex located at the midpoint of the bonded length of the anchor. For the limestone bedrock, the apex angle of the rock failure cone should be taken as 60 degrees.
- The factored uplift resistance of the anchor should be determined by the submerged weight of the cone-shaped rock mass around the anchor. The submerged weight of the rock cone mass should not be less than the ultimate capacity of the anchor. The submerged unit weight of the limestone bedrock equal to 16.8 kN/m^3 should be used in the calculations.
- For the case where the centre to centre spacing of the adjacent rock anchors is less than 1.2 times the height of the rock cone, the anchor group resistance for rock mass failure should be reduced to reflect the rock cone overlap.
- Where the embedment rock cones for a group of anchors overlap with each other, the combined embedment cones for the group of anchors should be used to determine the anchor group resistance to the rock mass pull-out failure.

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 the Ontario Provincial Standard Specification (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.

8.5 Additional Boreholes

It is recommended that additional boreholes be undertaken within the footprint of the proposed buildings in the area of the Phase I development to better delineate the bedrock depth (elevation) and the geotechnical engineering properties of the bedrock.

9. Floor Slab and Drainage Requirements

The lowest floor level of the parking garage for the proposed buildings is anticipated to be at a 12.0 m depth (Elevation 72.5 m) below existing grade.

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

9.1 Drained Structure

For the drained structure, the lowest floor slab of the parking garage may be designed as a slab-on-grade and the surface of the floor slab may consist of a concrete surface or a paved surface. Based on the borehole information, the lowest floor slab of the buildings for a drained structure will be founded on the dense to very dense glacial till. 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.

9.1.1 Lowest Floor Level as a Concrete Surface

The subgrade is anticipated to consist of glacial till or the limestone bedrock. The exposed glacial till should be proofrolled in the presence of EXP and any identified loose/soft areas should be excavated, removed and replaced with Ontario Provincial Standard Specification (OPSS) Granular B Type II material compacted to 100 percent standard Proctor maximum dry density (SPMDD). The limestone bedrock should be examined by EXP and any loose/soft zones of the bedrock should be excavated and removed.

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

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

The concrete slab should be reinforced and adequate saw cuts should be provided in the floor slab to control cracking.

9.1.2 Lowest Floor Level as a Paved Surface

The subgrade is anticipated to consist of glacial till or limestone bedrock. The exposed glacial till should be proofrolled in the presence of EXP and any identified loose/soft areas should be excavated, removed and replaced with Ontario Provincial Standard Specification (OPSS) Granular B Type II material compacted to 100 percent standard Proctor maximum dry density (SPMDD). The limestone bedrock should be examined by EXP and any loose/soft zones of the bedrock should be excavated and removed.

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

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

9.2 **Water-Tight Structure**

Additional comment will be provided should water-tight structure be selected for the proposed buildings.

10. Lateral Earth Pressure Against Subsurface Walls

10.1 Water-Tight Structure

For a water-tight structure, the foundation and subsurface basement should be designed to withstand lateral earth pressure as well as full hydrostatic pressure. For this purpose, the highest groundwater table at the site should be assumed to coincide with the ground surface.

The lateral thrust on the subsurface walls due to earth and water pressures may be computed from the expression:

$$p = ½k\,\gamma' H^2 + kqH + ½\gamma_w H^2$$

where p = lateral thrust due to earth and water pressure, kN/m
k = lateral earth pressure coefficient at rest, assumed to be 0.5
γ' = 12 kN/m^3 is the estimated submerged unit weight of the soil
q = is an allowance for surcharge, kPa
H = height of subsurface wall, m
γ_w = unit weight of water (9.81 kN/m^3)

In addition to the static earth and water pressures, subsurface walls would be subjected to dynamic thrust from the soil and hydrodynamic thrust during a seismic event. The soil dynamic thrust (Δ_{Pe}) and the hydrodynamic thrust (P_w) may be computed from the equations given below:

$$\Delta_{Pe} = \gamma H^2 \frac{a_h}{g} F_b$$

where Δ_{Pe} = dynamic thrust in kN/m of wall
H = height of wall of the tank/basement, m
γ = unit weight of soil = 22 kN/m^3
$\frac{a_h}{g}$ = seismic coefficient = 0.32 for the Ottawa area
F_b = thrust factor = 1.0

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

$$P_w = \frac{7}{12} \frac{a_h}{g} \gamma_w H^2$$

where P_w = hydrodynamic thrust in kN/m of wall
H = depth of water in tank, m
γ_w = unit weight of water (9.81 kN/m^3)
$\frac{a_h}{g}$ = seismic coefficient = 0.32 for Ottawa area

The hydrodynamic thrust acts at P_w should be assumed to act at $0.6H_w$ from the top of the water level.

The total lateral thrust due to the water on the face of the wall is the sum of the hydrostatic and hydrodynamic thrusts.

All subsurface walls should be properly waterproofed.

10.2 Drained Structure

For drained structures, the subsurface basement walls are designed not to support hydrostatic pressure behind the wall. In this case, the subsurface basement walls should be backfilled with free draining material, such as OPSS Granular B Type II compacted to 95 percent SPMDD and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the walls. The walls will be subjected to lateral static and dynamic (seismic) earth forces. The expressions below assume free draining backfill material, a perimeter drainage system, level backfill surface behind the wall and vertical face on the back side of the wall.

For design purposes, the lateral static earth thrust against the subsurface walls may be computed from the following equation:

$$P = K_0\, h\, (½\, \gamma h + q)$$

where P = lateral earth thrust acting on the subsurface wall, kN/m

K_0 = lateral earth pressure at rest coefficient, assumed to be 0.5 for Granular B Type II backfill material

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

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

q = surcharge load stress, kPa

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

$$\Delta_{Pe} = \gamma H^2 \frac{a_h}{g} F_b$$

where Δ_{Pe} = dynamic thrust in kN/m of wall

H = height of wall, m

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

$\frac{a_h}{g}$ = seismic coefficient = 0.32 (Ottawa Area)

F_b = thrust factor = 1.0

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

All subsurface walls should be properly waterproofed.

11. Excavations and De-Watering Requirements

11.1 Excess Soil Management

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

Reference should be made to the Phase One And Two ESAs for the environmental aspects of the project.

11.2 Excavations

11.2.1 Overburden Soil Excavation

Excavations for the construction of the proposed Phase I development is expected to extend to a minimum of 13.0 m depth below the existing ground surface. These excavations will extend through the fill, native overburden soils and to or possibly into the limestone bedrock. The excavations are anticipated to be below the groundwater level.

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

All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils on site are considered to be Type 3 and as such must be cut back at 1H:1V from the bottom of the excavation above the groundwater level. Within zones of persistent seepage and below the groundwater level, the excavation side slopes are expected to slough and eventually stabilize at a slope of 2H:1V to 3H:1V.

It is anticipated that due to the significant depth of the excavation and the proximity of the excavation to existing buildings and infrastructure, the excavations will likely have to be undertaken within the confines of a shoring system. The shoring system may consist of steel H soldier pile and timber lagging system, interlocking sheeting system and/or a secant pile wall 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 wall system can be considered. The steel interlocking sheeting and secant pile system also provide a cut-off to groundwater flows into the excavation. In areas where the potential of settlement of the nearby structures is low, soldier pile and timber lagging system may be used. The shoring system will require lateral restraint provided by tiebacks consisting of rock anchors. Due to the presence of cobbles and boulders in the subsurface soils, pre-drilling may be required for the installation of the soldier piles and a thickened section may be required for the interlocking steel sheeting system.

Since the excavation for the proposed buildings will be deep and below the groundwater level and located near existing buildings and infrastructure, it is recommended that the excavation be undertaken within the confines of a secant pile wall system that will support the walls of the excavation and will provide a cut-off to ground water flows.

Open cut trench method excavation for the installation of the new sanitary sewer to be located south of the proposed buildings will likely need to be undertaken within a shoring system that will support the walls of the excavation and prevent settlement of adjacent buildings and infrastructure. Alternatively, the new sanitary sewer may be installed by directional drilling. A contractor specializing in directional drilling should be consulted to assess the feasibility of installing the new sanitary sewer by directional drilling.

Confirmation of the need for a shoring system, the appropriate type of shoring system and the design and installation of the shoring system should be conducted by a professional engineer experienced in shoring design and by a contractor experienced in the installation of shoring systems. The shoring system should be designed and installed in accordance with OHSA and the 2023 CFEM (Canadian Foundation Engineering Manual (Fifth Edition)).

Additional comments regarding shoring systems are discussed below.

Soldier Pile and Timber Lagging System

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

P = k (γh + q)

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

k = applicable earth pressure coefficient; active lateral earth pressure coefficient = 0.33

‘at rest’ lateral earth pressure coefficient = 0.50

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

h = the depth, in metres, at which pressure, P, is being computed

q = the equivalent surcharge acting on the ground surface adjacent to the shoring system

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

The shoring should be designed using appropriate ‘k’ values depending on the location of any settlement-sensitive infrastructure (roadways sidewalks 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.

Secant Pile Wall System

The secant pile shoring system should be designed to resist ‘at rest’ lateral earth thrust in addition to the hydrostatic thrust as given by the expression below:

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

where:

P_0 = at rest’ earth and water thrusts acting against secant pile wall (kN/m)
K_0 = ‘at rest’ lateral earth pressure coefficient = 0.50
q = surcharge acting adjacent to the excavation (kPa)
h_1 = height of shoring from the ground surface to groundwater table (m)
h_2 = height of shoring from groundwater table to the bottom of excavation (m)
γ = unit weight of the soil = 22 kN/m^3

γ' = submerged unit weight of soil = 11.2 kN/m^3

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

If the secant pile wall system is incorporated into the design of the proposed buildings, they should be designed to resist soil dynamic thrust and hydrodynamic thrust during a seismic event.

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

The shoring systems should be tied back by rock anchors grouted into the sound bedrock. The factored ULS grout to rock bond of 600 kPa may be used for design of the anchors. This value assumes a grout with a minimum strength of 30 MPa is used and that the sides of the drilled holes are cleaned prior to the grouting operation. It is anticipated that the bedrock may contain near vertical seams and some horizontal fractures and therefore some grout loss when grouting anchors in the bedrock should be anticipated. The grout loss is expected to be higher in the fractured bedrock and lower in the sound bedrock. Difficulties may be encountered during the installation of the rock anchors due to the presence of boulders/cobbles within the till.

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

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

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

The shoring system should be monitored for movement (including deflection) on a periodic basis during construction operations.

It is recommended that the adjacent sensitive structures and infrastructure should be monitored for movement (including deflection), settlement and vibration on a periodic basis during construction operations.

11.2.2 Rock Excavation

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

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

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

Rock Support

Zones of the weathered and fractured rock faces for the portion of the excavation within the bedrock may require support in the form of rock bolts to maintain the integrity of the rock face in conjunction with a wire mesh system and/or shotcrete. Excavations 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

It is anticipated that blasting will not be required at this site. However, should blasting be carried out then the vibration limits for blasting should be in accordance with City of Ottawa Special Provisions (SP No. 1201).

Prior to the commencement of any blasting operation, the contractor must retain a blasting specialist to prepare a detailed blast plan and a methodology which will prevent damage to the nearby structures and infrastructures situated within the zone of influence of the blasting. .

As previously indicated, it is recommended that a pre-construction condition survey of adjacent building(s) and infrastructure (roadways, sidewalks, municipal services) be undertaken prior to any earth (soil) and rock excavation work. Vibration monitoring and monitoring of the shoring system as well as adjacent settlement sensitive structures for movement (deflection) should be carried out on a periodic basis during construction operations. If blasting is being considered then additional vibration monitoring during excavation, blasting and construction operations will be required.

In addition, instrumentation should be installed along the newly relocated watermain to ensure that it is not negatively impacted by any blasting and rock removal.

General Comment

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

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

Reference is made to the 2023 GIA report for additional comments regarding short-term and long-term lowering of the groundwater level. It is recommended that the GIA report be updated to include only the south portion of the site for the Phase I development. Based on the findings from the updated GIA report, the comments and recommendations provided in this update geotechnical report may need to be revised.

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

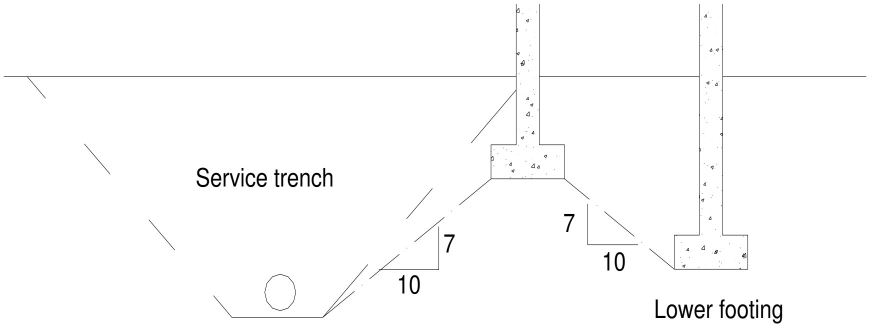
For construction dewatering, an Environmental Activity and Sector Registry (EASR) approval may be obtained for water takings greater than 50 m^3 and less than 400 m^3. If more than 400 m^3 per day of groundwater are generated per day for dewatering purposes, then a Permit to Take Water (PTTW) must be obtained from the MECP. The GIA should be updated based on the plans for the Phase 1 development, in support a PTTW application.

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. Relocation of the 375 mm Sanitary Sewer

An existing 375 mm sanitary sewer currently crosses the middle of the Phase I development in an east-west direction and is to be relocated within the City of Ottawa easement to the south of the proposed buildings, as shown in Figure 2. The relocated sanitary sewer will be set at invert elevations ranging from Elevation 80.94 m to Elevation 80.64 m, i.e below the ground surface and will be situated at a distance of 2.5 m to 3.5 from the North side of the residences located just south of the proposed buildings. The City of Ottawa easement will have a minimum width of 6.0 m, and the width may be increased to for allow future access for repairs and maintenance.

The sanitary sewer should be constructed so that the pipe invert is above a line drawn at 10 horizontal to 7 vertical (10H:7V) from the near edge of the footings of the existing residences. If the invert is located below this line and cannot be relocated, then underpinning of the existing footings may be required.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

Excavations for the installation of the relocated sanitary sewer must be undertaken within an engineered support system, such as a trench box or a sheet pile support system. The engineered support system is to be designed by a professional engineer retained by the contractor specifically for this project and designed in accordance with OHSA 213/91 and the recommendations in Section 11 of this report. The shoring system must be designed in such a way to eliminate any movement of soil behind the support system/trench box which will prevent any negative impact on the existing residences or infrastructures situated within the zone of influence of the proposed work. A work plan must be prepared by the contractor for this purpose and submitted for review prior to the start of any work or implementation. An instrumentation and monitoring program should also be outlined in the work plan for both the installation of the new sewer and following the installation of the sewer, i.e. during the excavation for the proposed buildings.

Alternatively, consideration can be given to explore the feasibility of the installation of the relocated 375 mm sewer using directional drilling with access and exist pits on the east and west side of the easement. For this purpose, a specialized contractor should be consulted to establish the feasibility of this option.

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

The soils to be excavated from the site will comprise of fill, clay, silty clay, silt and glacial till and limestone bedrock. From a geotechnical perspective, the soils and the limestone bedrock are not considered suitable for reuse as backfill material in the interior or exterior of the building and should be discarded. It may be possible to use portions of the fill as OPSS Select Subgrade Material (SSM), subject to further examination and testing at time of construction. However, these soils are subject to moisture absorption due to precipitation and must be protected at all times from the elements.

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

- Engineered fill under the floor slab (including backfilling in service trenches inside the building) - OPSS Granular B Type II (50 mm minus) placed in 300 mm thick lifts with each lift compacted to 100 percent SPMDD beneath the floor slab,
- Backfill against exterior subsurface walls - OPSS Granular B Type II placed in 300 mm thick lifts and compacted to 95 percent SPMDD,
- Trench backfill outside building area, and fill placement to design subgrade level for pavement - OPSS Select Subgrade Material (SSM), free of organics, debris and with a natural moisture content within 2 percent of the optimum moisture content. It should be placed in 300 mm thick lifts compacted to minimum 95 percent SPMDD; and
- Landscaped areas - Clean fill that is free of organics and deleterious material and is placed in 300 mm thick lifts with each lift compacted to 92 percent of the SPMDD.

14. Tree Planting Restrictions

Preliminary plans indicate the new trees will be planted within the footprint of the excavation for the proposed buildings. Since the existing native clay and silty clay will be excavated and removed from within the excavation for the proposed buildings, the new trees will not be planted in the clay and silty clay. Therefore, there are no tree planting restrictions from a sensitive marine clay perspective for this project.

15. Corrosion Potential

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

Table XI: Chemical Test Results

Borehole – Run No.	Depth (m)	Soil/Bedrock Type	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
BH No.1 - SS11	13.7 - 14.3	Glacial Till	8.04	0.013	0.005	3130
BH No. 6-Run 2	14.9 - 15.2	Limestone Bedrock	8.70	0.010	0.002	2910

The test results indicate the glacial till sample and limestone bedrock core section have 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 glacial till sample and limestone bedrock core section are mildly corrosive to bare steel as per the National Association of Corrosion Engineers (NACE). Appropriate measures should be undertaken to protect buried steel elements from corrosion.

16. Additional Work

The following additional work is recommended for the Phase I development of the site:

- It is recommended that additional boreholes be undertaken within the footprint of the proposed buildings in the area of the Phase I development to better delineate the bedrock depth (elevation) and the geotechnical engineering properties of the bedrock.
- It is recommended that the GIA report be updated to include only the south portion of the site for the Phase I development. Based on the findings from the updated GIA report, the comments and recommendations provided in this update geotechnical report may need to be revised.

17. 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 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. Reference is made to the Phase One and Two ESAs and the GIA reports prepared by EXP regarding the environmental aspects of the site.

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

Sincerely,

Daniel Wall

Daniel Wall, M. Eng., P.Eng.
Geotechnical Engineer
Earth and Environment

Susan M. Potyondy, P.Eng.
Senior Project Manager
Earth and Environment

LICENSED PROFESSIONAL ENGINEER
May 24, 2024
D C WALL
100159490
Daniel Wall
PROVINCE OF ONTARIO

Figures

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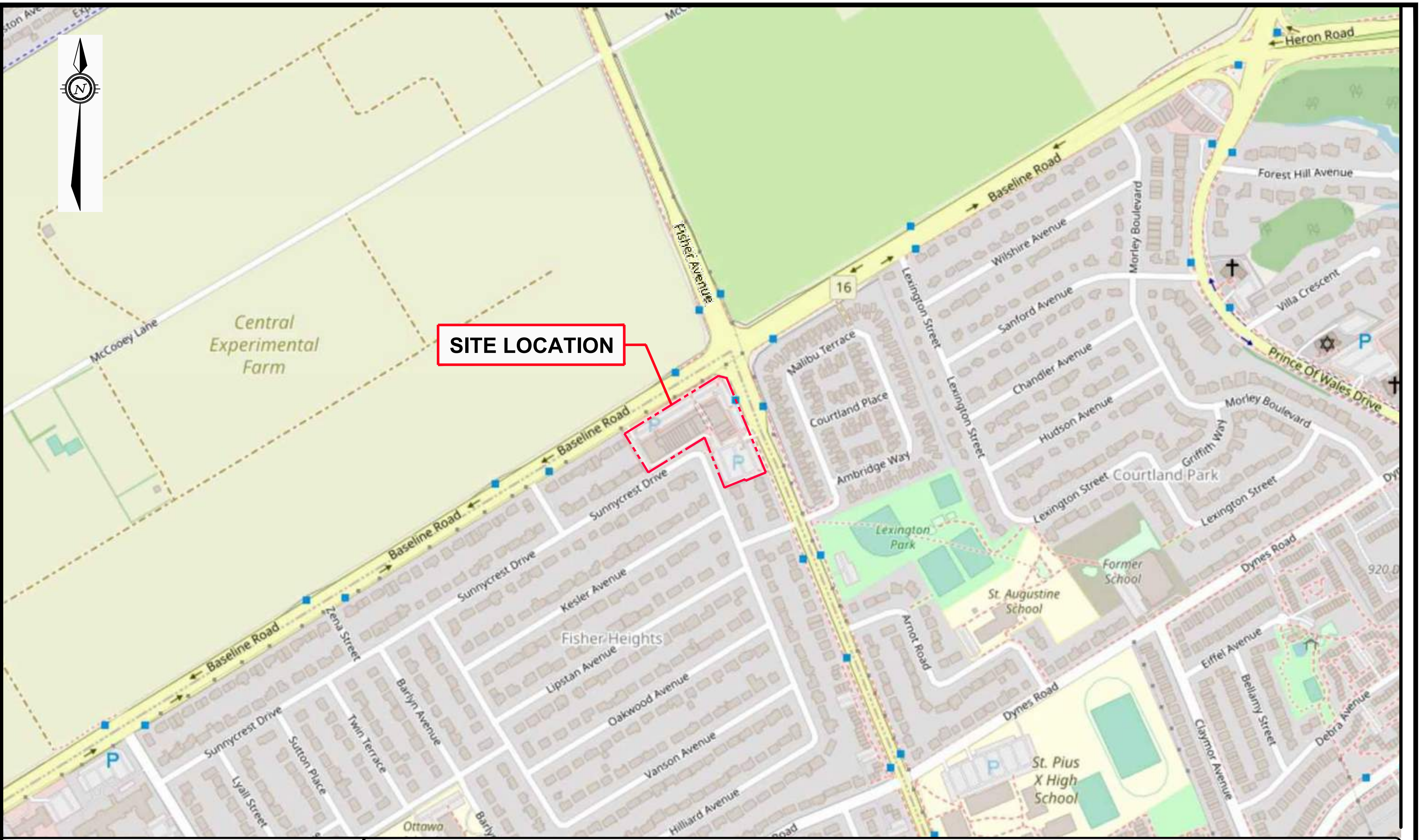

0 50m 100m 300m

HORIZONTAL 1:7,500

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

DATE
MAY 2022

DESIGN
DW

CHECKED
IT

DRAWN BY
AS

PROPOSED MULTI-USE TOWERS

780 BASELINE ROAD, OTTAWA, ONTARIO

SITE LOCATION PLAN

project no.
OTT-21011499-C0

scale
1:7,500

FIG 1

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GENERAL NOTES:

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

LEGEND

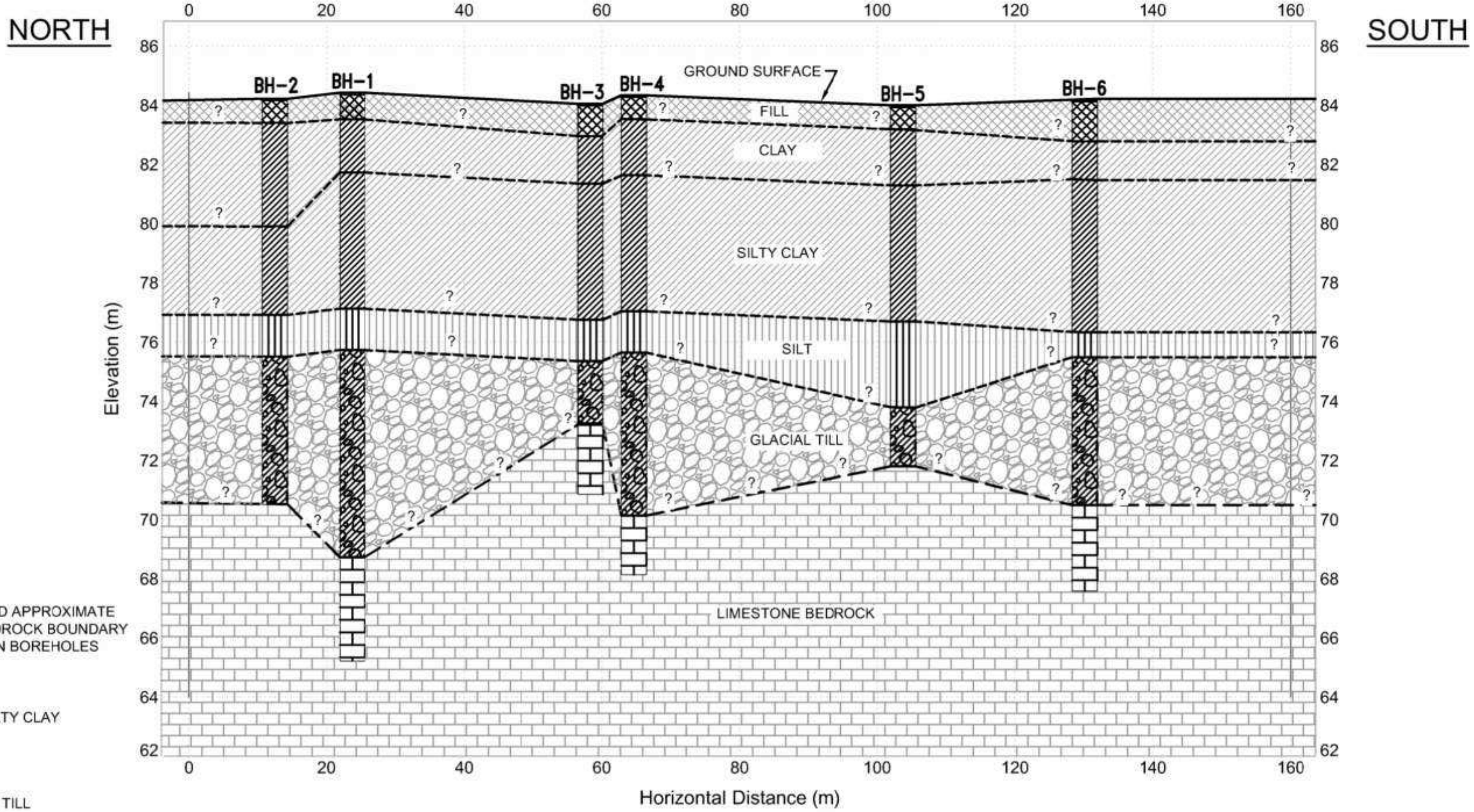
SITE BOUNDARIES
PHASE 1 DEVELOPMENT BOUNDARY
BH-1 BOREHOLE NUMBER AND LOCATION
GS = GROUND SURFACE ELEVATION (m)
R = BEDROCK ELEVATION (m)
AR = AUGER REFUSAL ON INFERRED BEDROCK (m)
A SECTION MARK
EXISTING SANITARY SEWER (TO BE REMOVED)
NEW PROPOSED SANITARY SEWER

ORIGINAL SHEET SIZE = 17" X 11"

0 5m 10m 30m
HORIZONTAL 1:750

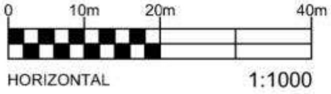
EXP Services Inc. www.exp.com
t: +1.613.688.1899 | f: +1.613.225.7337
2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

DATE MAY 2024		PROPOSED MULTI-USE TOWERS 780 BASELINE ROAD, OTTAWA, ONTARIO		project no. OTT-21011499-C0
DESIGN DW / SP	CHECKED IT			scale 1:750
DRAWN BY AS		BOREHOLE LOCATION PLAN		FIG 2



LEGEND

- ? ------ INFERRED APPROXIMATE SOIL/BEDROCK BOUNDARY BETWEEN BOREHOLES
- ▨ FILL
- ▨ CLAY/SILTY CLAY
- ▥ SILT
- ▨ GLACIAL TILL
- ▦ LIMESTONE BEDROCK



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Ottawa, ON K2B 8H6, Canada

DATE JUNE 2022		PROPOSED MULTI-USE TOWERS 780 BASELINE ROAD, OTTAWA, ONTARIO	project no. OTT-21011499-C0
DESIGN DW	CHECKED IT		scale 1:1,000
DRAWN BY AS		CROSS-SECTION A-A'	FIG 3

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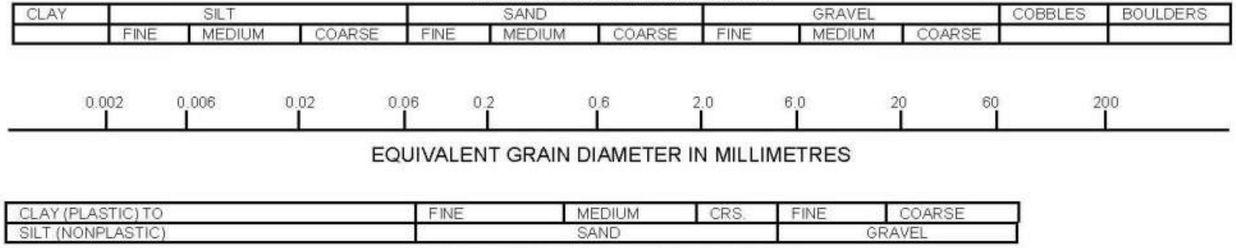
Notes On Sample Descriptions

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

ISSMFE SOIL CLASSIFICATION



UNIFIED SOIL CLASSIFICATION

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

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

Log of Borehole BH-1

Project No: OTT-21011499-C0

Figure No. 4

Project: Proposed Multi-Use Towers

Page. 1 of 2

Location: 780 Baseline Road, Ottawa, Ontario

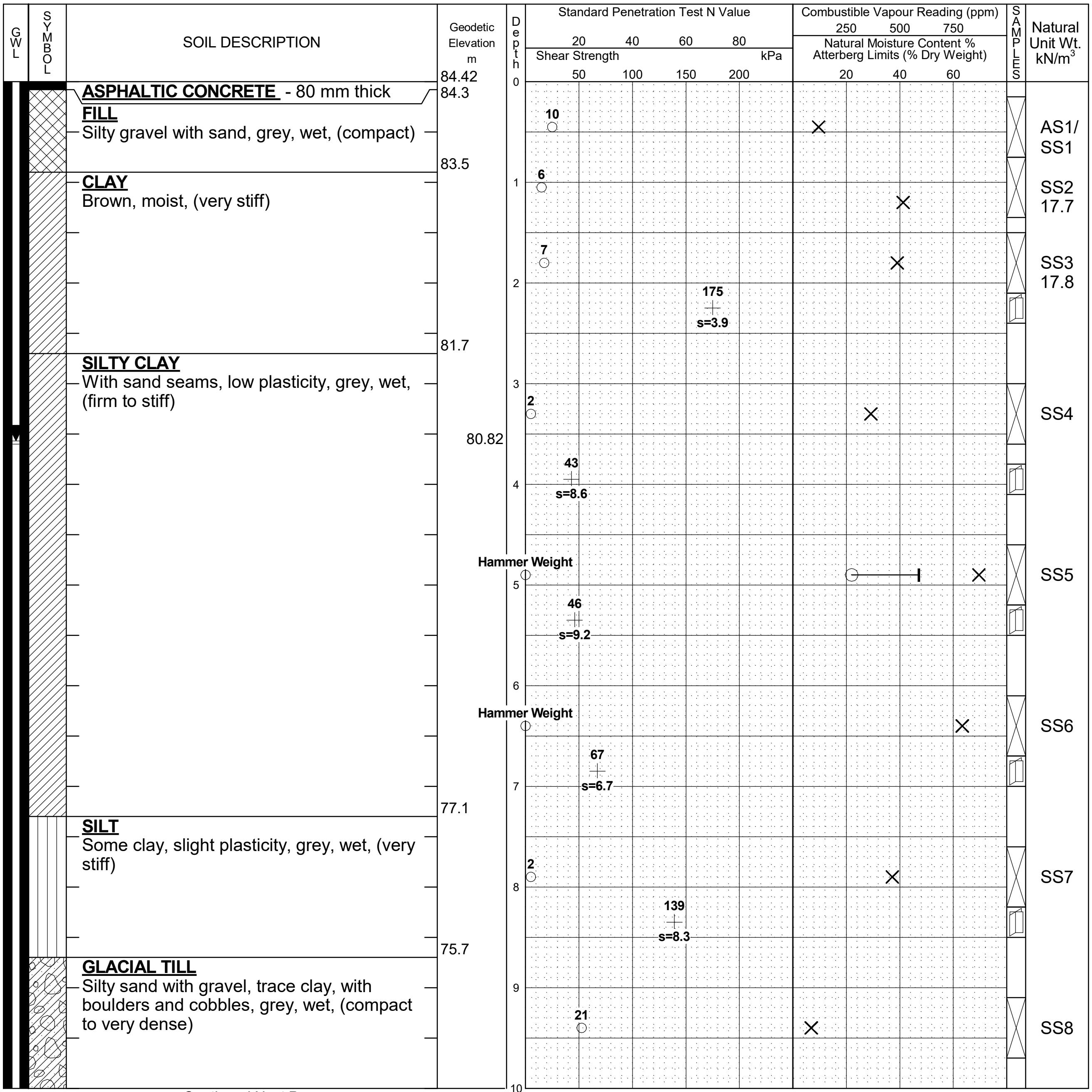
Date Drilled: 'April 14, 2022

Drill Type: CME 55 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: MZ   Checked by: DW

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
84.42	
84.3	**ASPHALTIC CONCRETE** - 80 mm thick
	FILL Silty gravel with sand, grey, wet, (compact)
83.5	**CLAY** Brown, moist, (very stiff)
81.7	**SILTY CLAY** With sand seams, low plasticity, grey, wet, (firm to stiff)
80.82	
77.1	**SILT** Some clay, slight plasticity, grey, wet, (very stiff)
75.7	**GLACIAL TILL** Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (compact to very dense)

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	3.6	
April 28, 2022	3.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	14.3 - 15.7	29	0
2	15.7 - 16.3	100	0
3	16.3 - 17.7	100	27
4	17.7 - 19.2	100	61

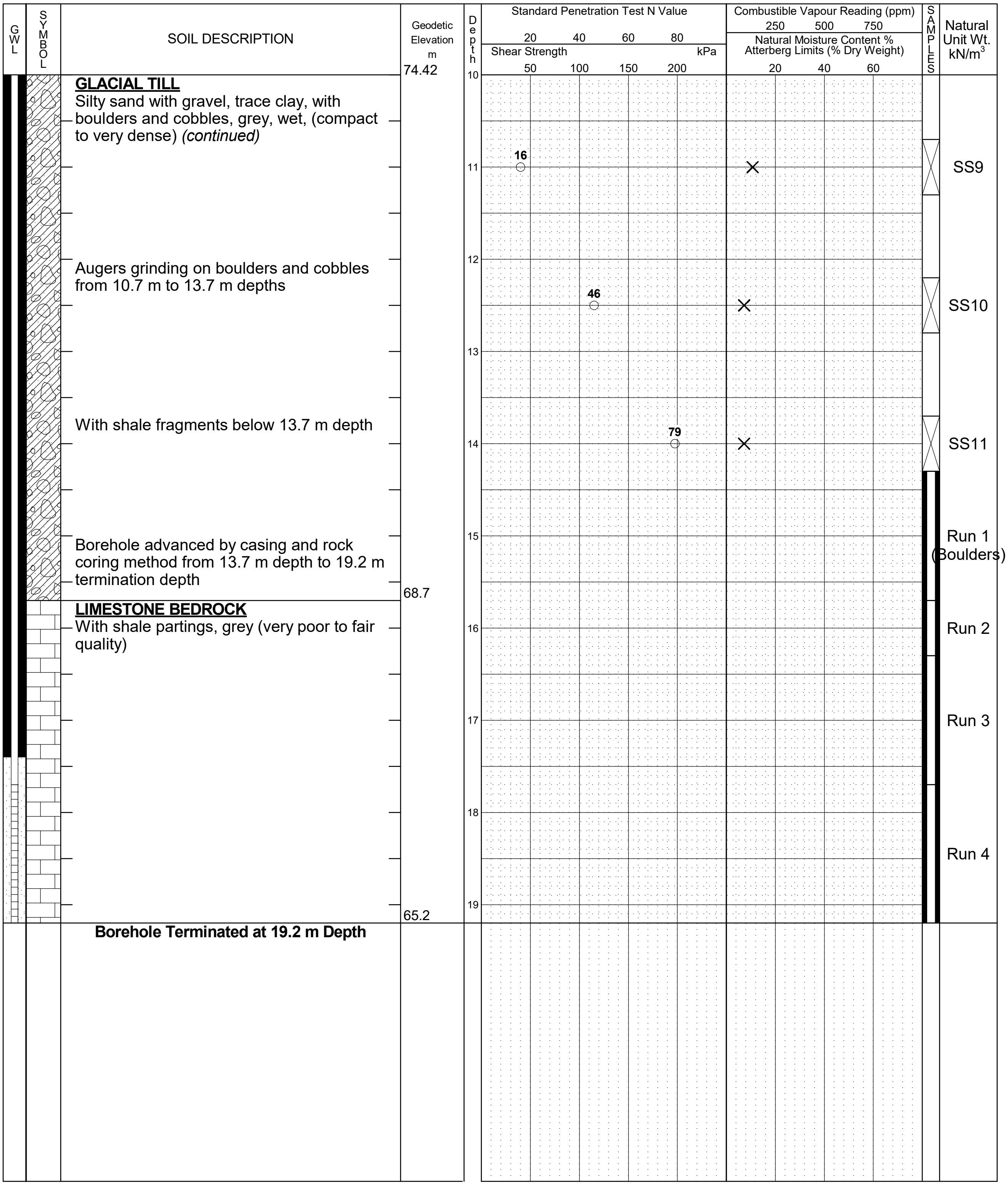
Log of Borehole BH-1

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Figure No. 4

Page. 2 of 2



GLACIAL TILL
Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (compact to very dense) *(continued)*

Augers grinding on boulders and cobbles from 10.7 m to 13.7 m depths

With shale fragments below 13.7 m depth

Borehole advanced by casing and rock coring method from 13.7 m depth to 19.2 m termination depth

LIMESTONE BEDROCK
With shale partings, grey (very poor to fair quality)

Borehole Terminated at 19.2 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	3.6	
April 28, 2022	3.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	14.3 - 15.7	29	0
2	15.7 - 16.3	100	0
3	16.3 - 17.7	100	27
4	17.7 - 19.2	100	61

Log of Borehole BH-2

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Location: 780 Baseline Road, Ottawa, Ontario

Figure No. 5

Page. 1 of 2

Date Drilled: 'April 11, 2022

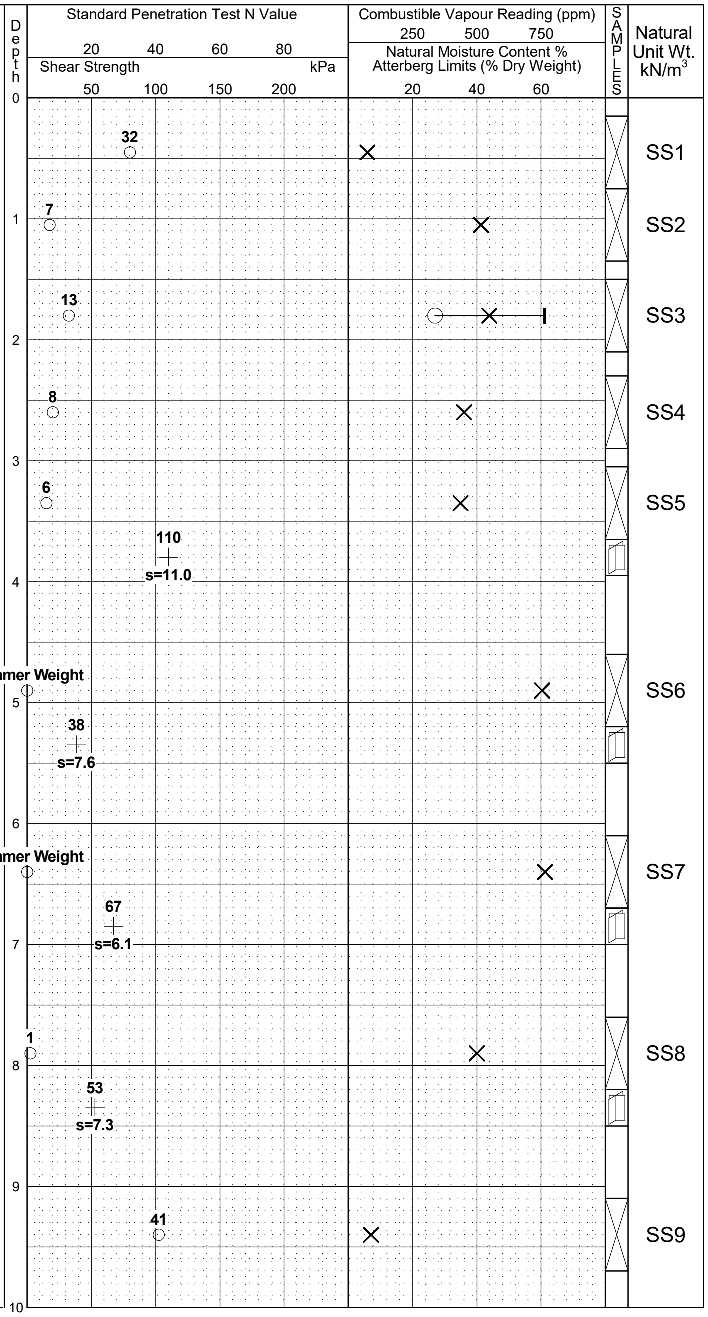
Drill Type: CME 55 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: MZ Checked by: DW

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
84.21	
84.2	**ASPHALTIC CONCRETE** - 60 mm thick
	FILL Silty sand and crushed gravel, brown, moist, (dense)
83.4	**CLAY** Trace sand, high plasticity, brown, moist (very stiff)
79.9	**SILTY CLAY** With sand seams, grey, wet, (firm to stiff)
79.01	
76.9	**SILT** Some clay, slight plasticity, grey, wet, (stiff)
75.5	**GLACIAL TILL** Silty sand with gravel, trace clay, with boulders, cobbles, and shale fragments grey, wet, (dense)



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NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 50 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	5.2	
April 28, 2022	5.0	

CORE DRILLING RECORD

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

Log of Borehole BH-2

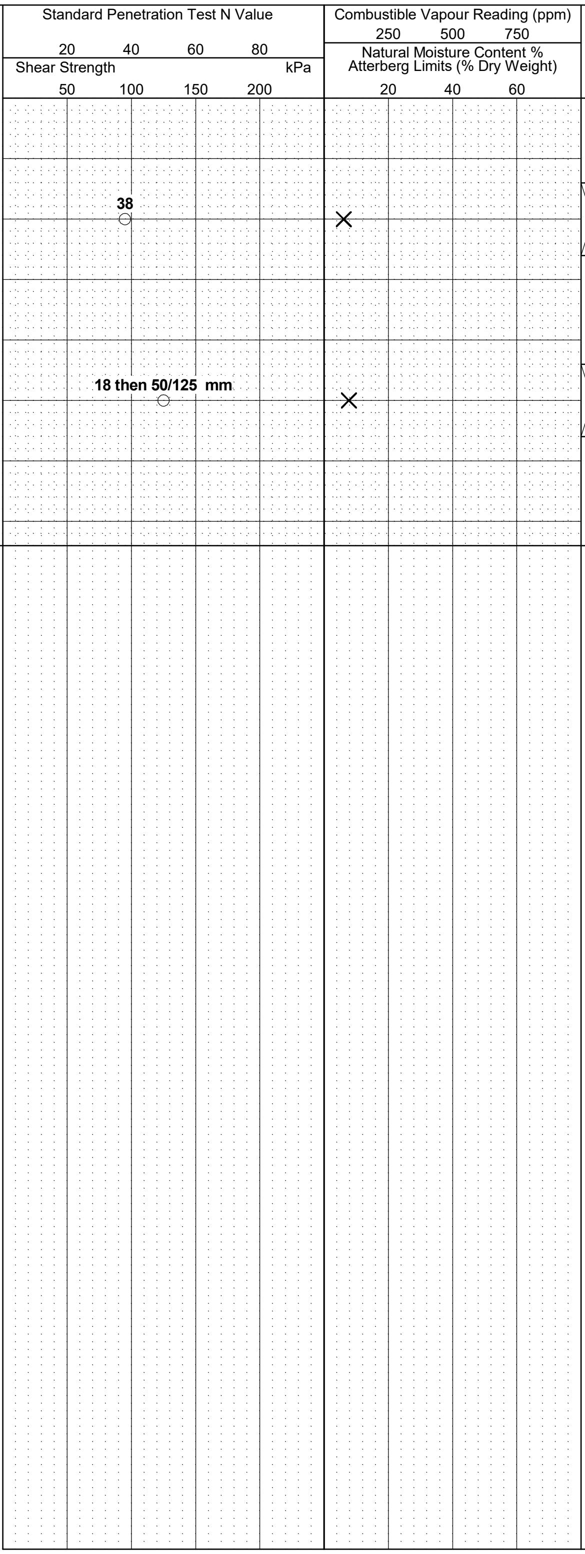
Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Figure No. 5

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	Standard Penetration Test N Value / Shear Strength kPa	Combustible Vapour Reading (ppm) / Natural Moisture Content % Atterberg Limits (% Dry Weight)	SAMPLES	Natural Unit Wt. kN/m³
		GLACIAL TILL Silty sand with gravel, trace clay, with boulders, cobbles, and shale fragments grey, wet, (dense) *(continued)*	74.21	10				
				11	38	×	SS10	
		Augers grinding on boulders and cobbles from 9.1 m depth to 13.7 m auger refusal depth.		12	18 then 50/125 mm	×	SS11	
				13				
			70.5					
		Auger Refusal at 13.7 m Depth.						



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 50 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	5.2	
April 28, 2022	5.0	

CORE DRILLING RECORD

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

Log of Borehole BH-3

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Location: 780 Baseline Road, Ottawa, Ontario

Figure No. 6

Page. 1 of 2

Date Drilled: 'April 13, 2022

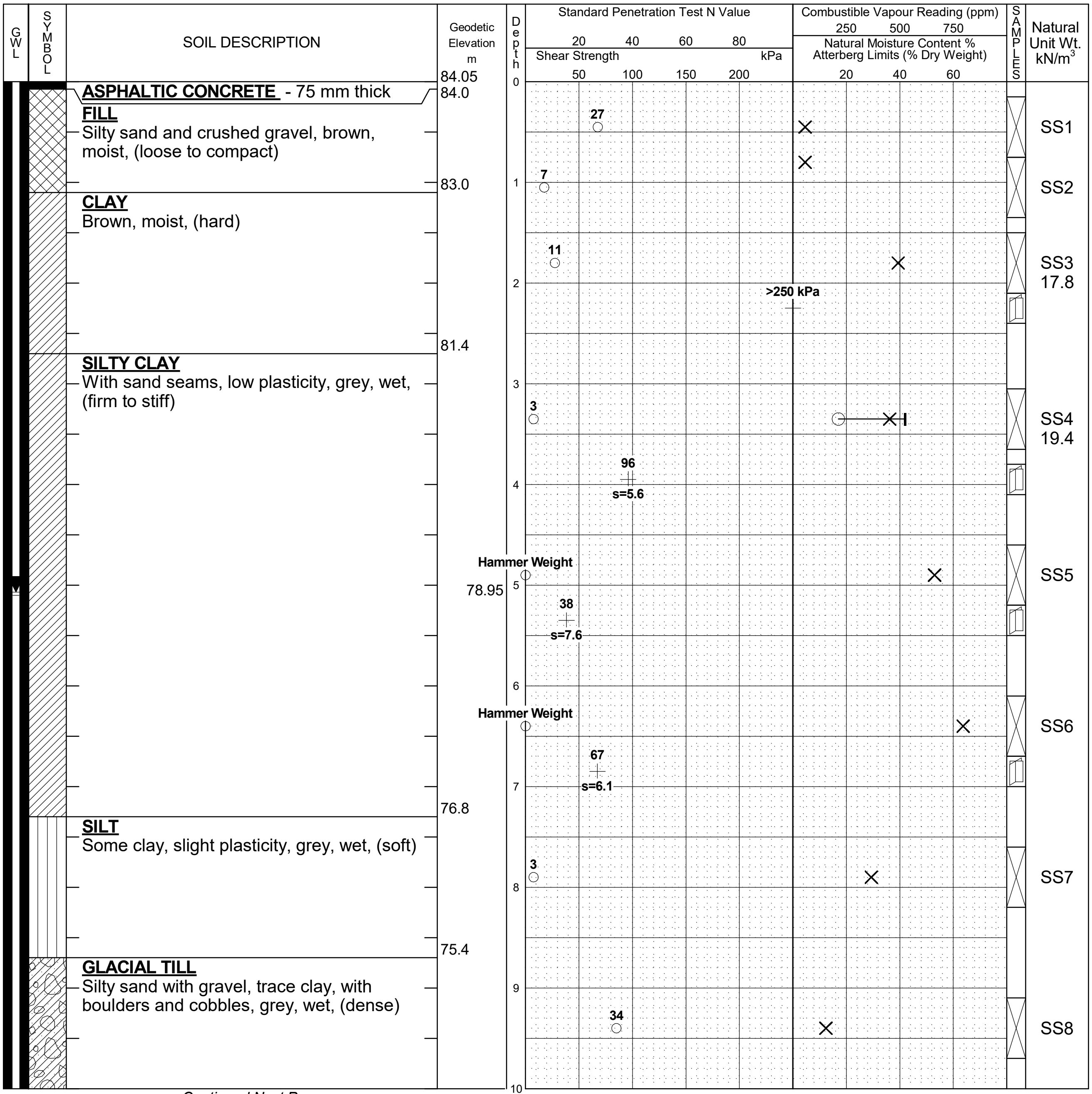
Drill Type: CME 55 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: MZ   Checked by: DW

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
84.05 84.0	**ASPHALTIC CONCRETE** - 75 mm thick
	FILL Silty sand and crushed gravel, brown, moist, (loose to compact)
83.0	**CLAY** Brown, moist, (hard)
81.4	**SILTY CLAY** With sand seams, low plasticity, grey, wet, (firm to stiff)
78.95	
76.8	**SILT** Some clay, slight plasticity, grey, wet, (soft)
75.4	**GLACIAL TILL** Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (dense)



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NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	5.1	
April 28, 2022	4.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	10.8 - 11.6	100	47
2	11.6 - 13.2	80	42

Log of Borehole BH-3

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Figure No. 6

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	Standard Penetration Test N Value (20, 40, 60, 80) / Shear Strength kPa (50, 100, 150, 200)	Combustible Vapour Reading (ppm) (250, 500, 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20, 40, 60)	SAMPLES	Natural Unit Wt. kN/m³
			74.05	10				
		GLACIAL TILL Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (dense) *(continued)*						
		LIMESTONE BEDROCK With shale partings, grey, (poor quality)	73.3		46/bouncing ○	×	☒	SS9
				11				Run 1
				12				Run 2
			70.9	13				

Borehole Terminated at 13.2 m Depth

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	5.1	
April 28, 2022	4.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	10.8 - 11.6	100	47
2	11.6 - 13.2	80	42

Log of Borehole BH-4

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Location: 780 Baseline Road, Ottawa, Ontario

Figure No. 7

Page. 1 of 2

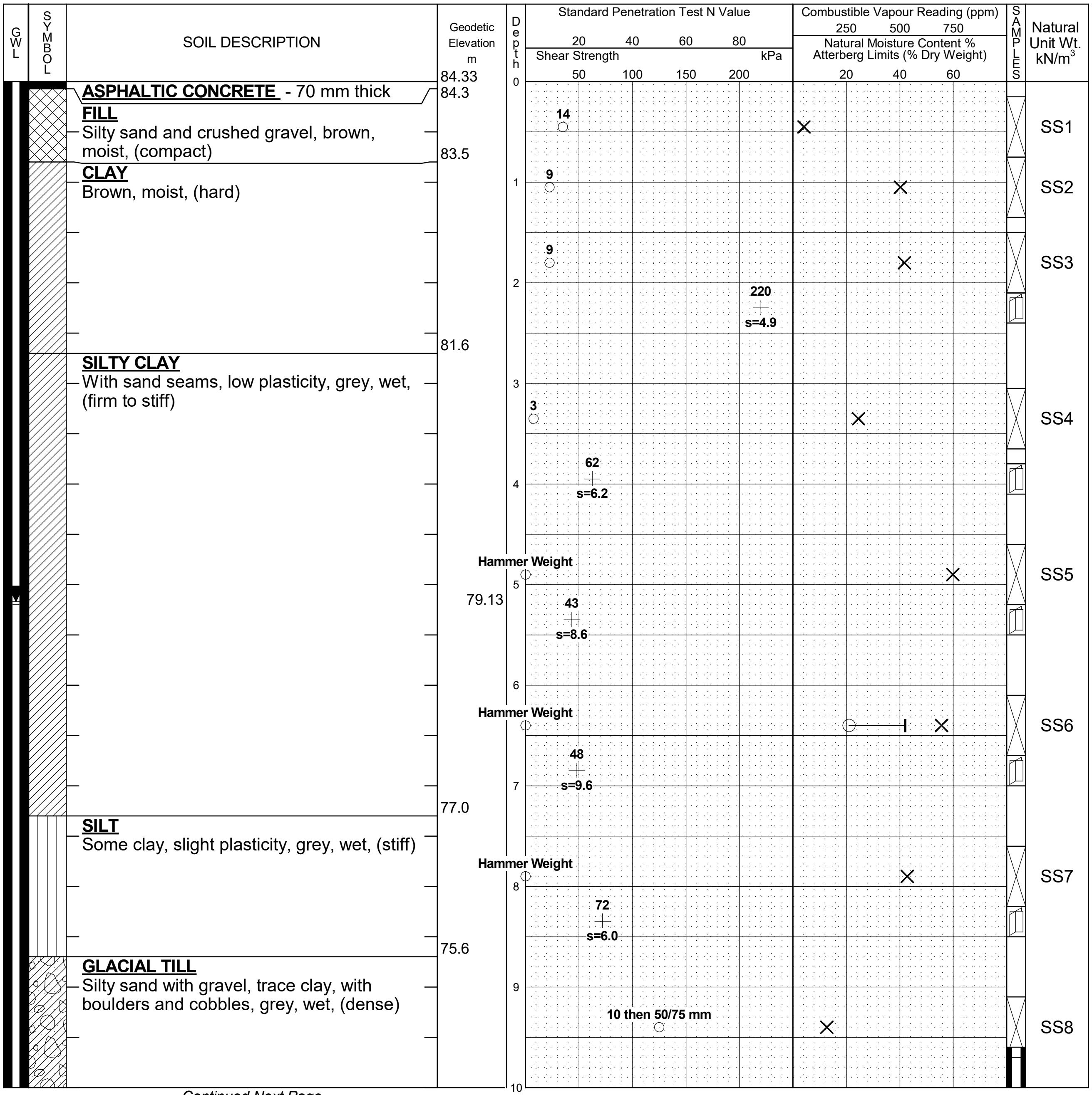
Date Drilled: 'April 12, 2022

Drill Type: CME 55 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: MZ Checked by: DW

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
	84.33
ASPHALTIC CONCRETE - 70 mm thick	84.3
FILL Silty sand and crushed gravel, brown, moist, (compact)	83.5
CLAY Brown, moist, (hard)	81.6
SILTY CLAY With sand seams, low plasticity, grey, wet, (firm to stiff)	77.0
SILT Some clay, slight plasticity, grey, wet, (stiff)	75.6
GLACIAL TILL Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (dense)	

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	5.2	
April 28, 2022	5.0	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	9.6 - 10.7	61	18
2	10.7 - 12.2	28	17
3	12.2 - 14.2	15	0
4	14.2 - 14.6	100	86
5	14.6 - 16.2	85	29

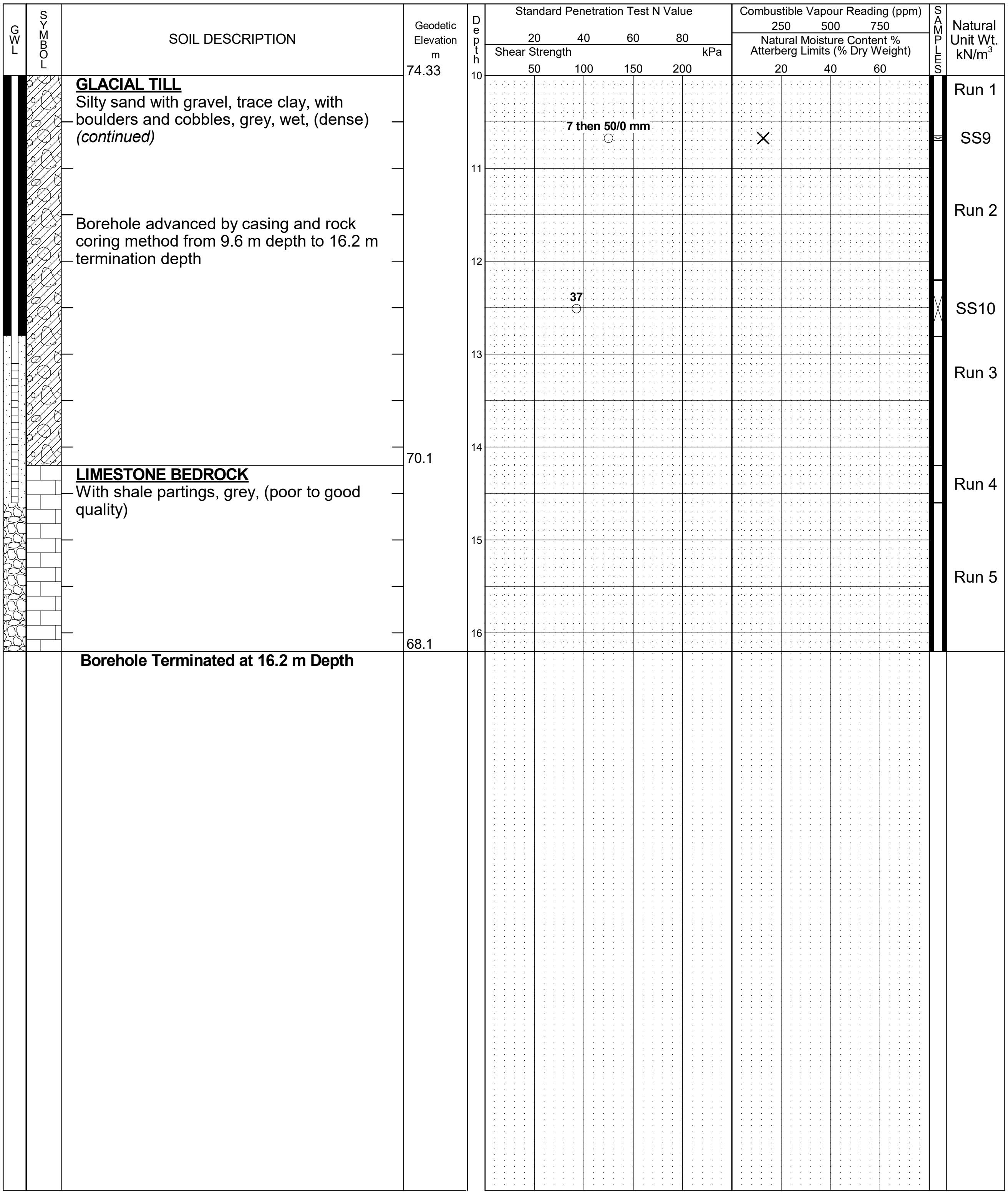
Log of Borehole BH-4

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Figure No. 7

Page. 2 of 2



GLACIAL TILL
Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (dense) *(continued)*

Borehole advanced by casing and rock coring method from 9.6 m depth to 16.2 m termination depth

LIMESTONE BEDROCK
With shale partings, grey, (poor to good quality)

Borehole Terminated at 16.2 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	5.2	
April 28, 2022	5.0	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	9.6 - 10.7	61	18
2	10.7 - 12.2	28	17
3	12.2 - 14.2	15	0
4	14.2 - 14.6	100	86
5	14.6 - 16.2	85	29

Log of Borehole BH-5

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Location: 780 Baseline Road, Ottawa, Ontario

Figure No. 8

Page. 1 of 2

Date Drilled: 'April 12, 2022

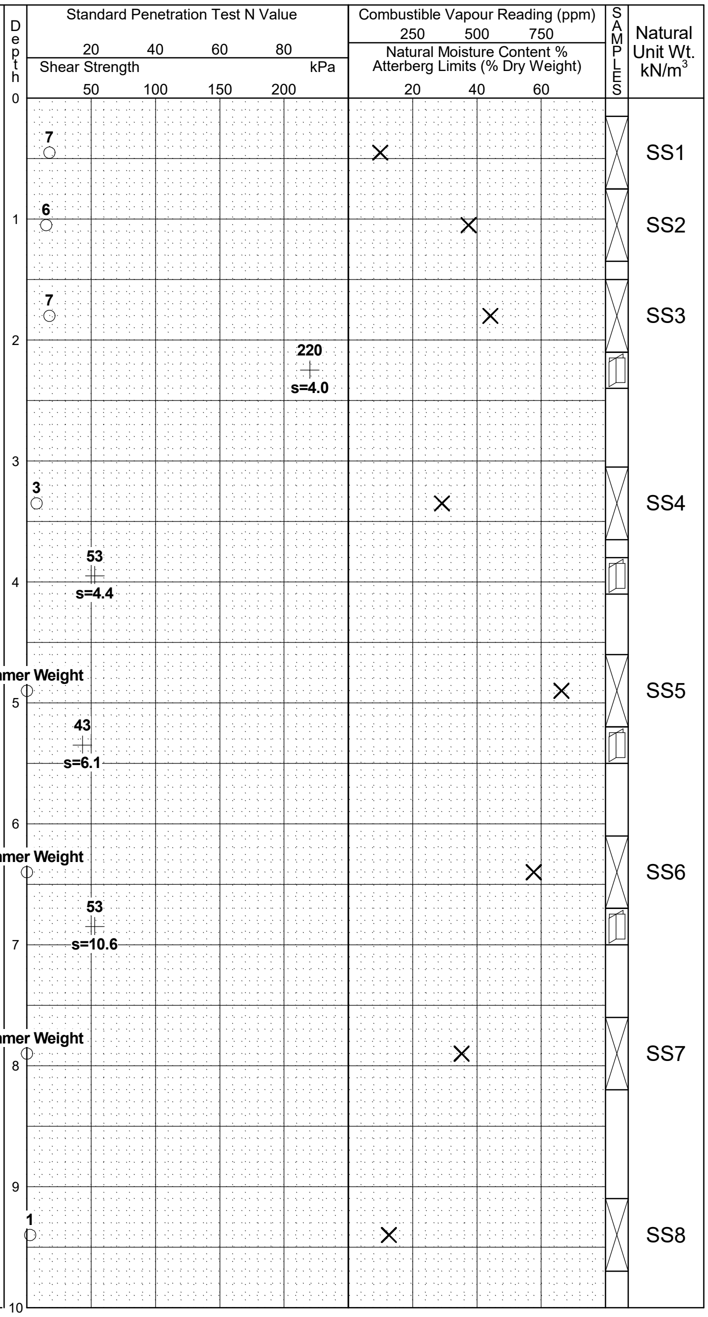
Drill Type: CME 55 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: MZ Checked by: DW

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	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m
			83.99
		ASPHALTIC CONCRETE - 80 mm thick	83.9
		FILL Silty sand and crushed gravel, grey, moist, (loose)	83.2
		CLAY Brown, moist, (hard)	81.3
		SILTY CLAY With sand seams, grey, wet, (firm to stiff)	
▼			79.09
			76.7
		SILT Some clay, non-plastic, grey, wet, (very loose)	



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NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	4.9	
April 28, 2022	4.7	

CORE DRILLING RECORD

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

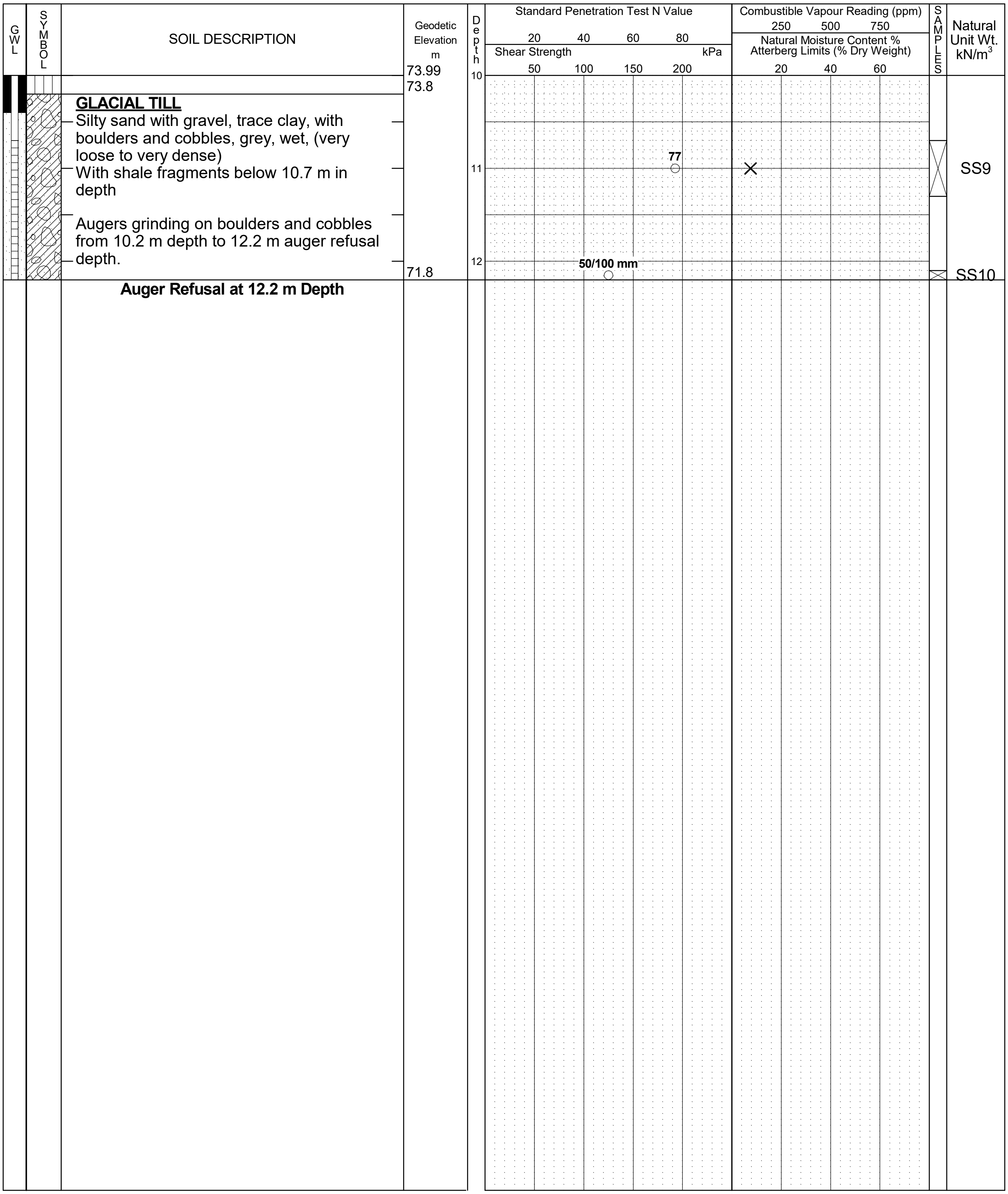
Log of Borehole BH-5

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Figure No. 8

Page. 2 of 2



GLACIAL TILL

Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (very loose to very dense)

With shale fragments below 10.7 m in depth

Augers grinding on boulders and cobbles from 10.2 m depth to 12.2 m auger refusal depth.

Auger Refusal at 12.2 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	4.9	
April 28, 2022	4.7	

CORE DRILLING RECORD

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

Log of Borehole BH-6

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Location: 780 Baseline Road, Ottawa, Ontario

Figure No. 9

Page. 1 of 2

Date Drilled: 'April 18, 2022

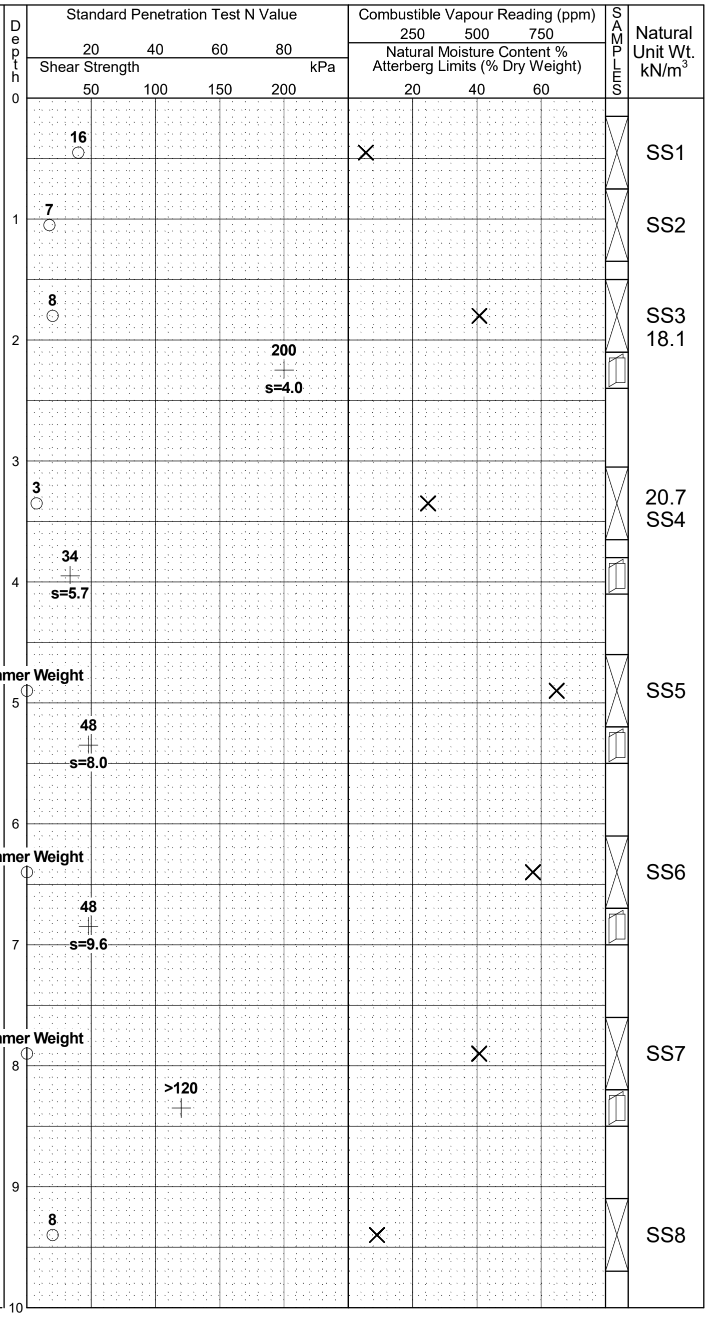
Drill Type: CME 55 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: MZ   Checked by: DW

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

SOIL DESCRIPTION	Geodetic Elevation m
	84.18
ASPHALTIC CONCRETE - 70 mm thick	84.1
FILL Silty sand and crushed gravel, grey, wet, (compact)	
CLAY Brown, moist, (hard)	82.8
SILTY CLAY With sand seams, grey, wet, (firm)	81.5
SILT Some clay, slight plasticity, grey, wet, (very stiff)	76.3
GLACIAL TILL Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (loose to very dense)	75.5



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NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
June 23, 2022	4.8	
April 28, 2022	4.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	12.2 - 13.7	37	0
2	13.7 - 15.2	80	23
3	15.2 - 16.6	91	44

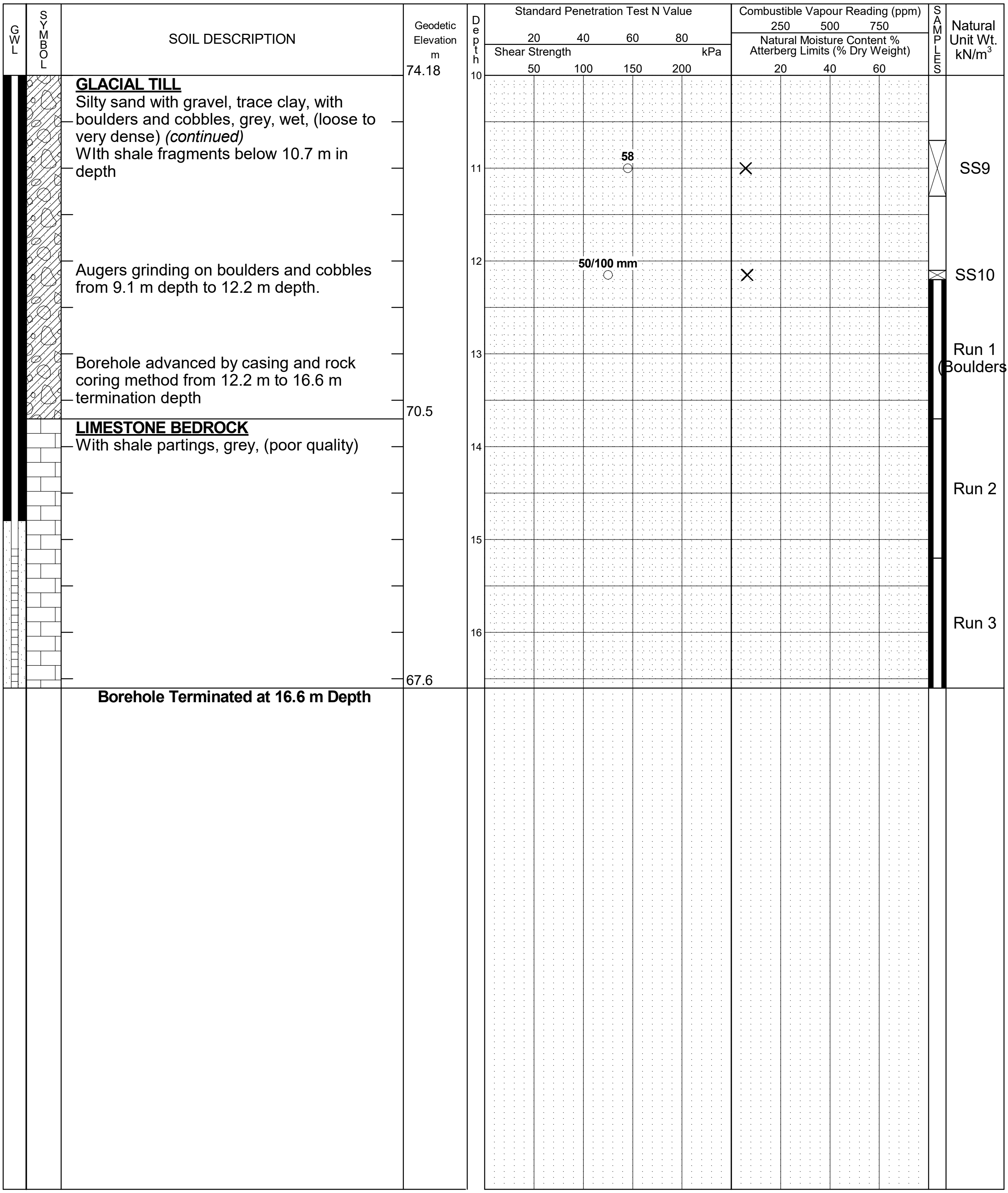
Log of Borehole BH-6

Project No: OTT-21011499-C0

Project: Proposed Multi-Use Towers

Figure No. 9

Page. 2 of 2



GLACIAL TILL
Silty sand with gravel, trace clay, with boulders and cobbles, grey, wet, (loose to very dense) *(continued)*
WIth shale fragments below 10.7 m in depth

Augers grinding on boulders and cobbles from 9.1 m depth to 12.2 m depth.

Borehole advanced by casing and rock coring method from 12.2 m to 16.6 m termination depth

LIMESTONE BEDROCK
With shale partings, grey, (poor quality)

Borehole Terminated at 16.6 m Depth

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 38 mm diameter monitoring well installed as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21011499-C0

WATER LEVEL RECORDS

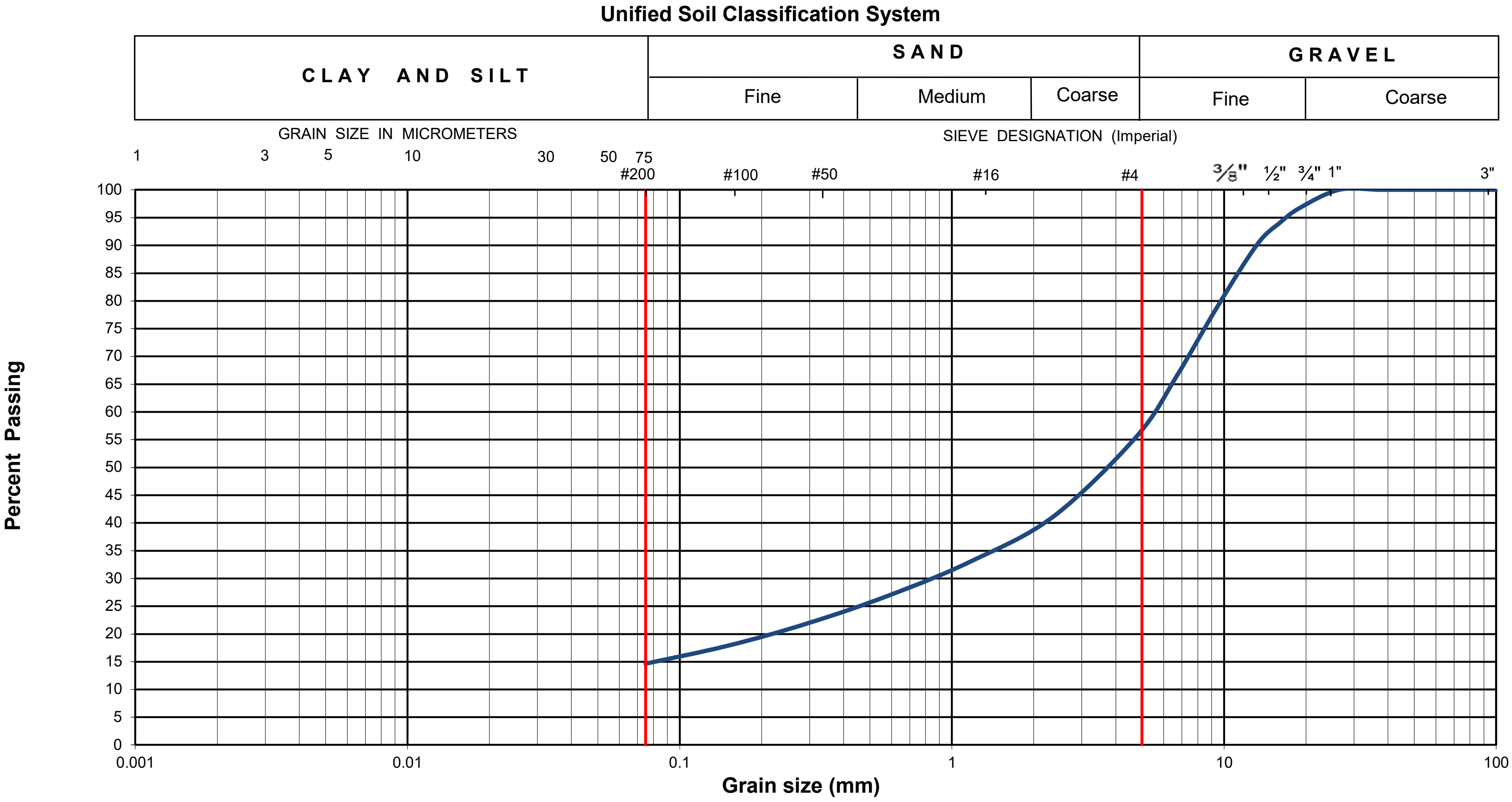
Date	Water Level (m)	Hole Open To (m)
June 23, 2022	4.8	
April 28, 2022	4.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	12.2 - 13.7	37	0
2	13.7 - 15.2	80	23
3	15.2 - 16.6	91	44

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

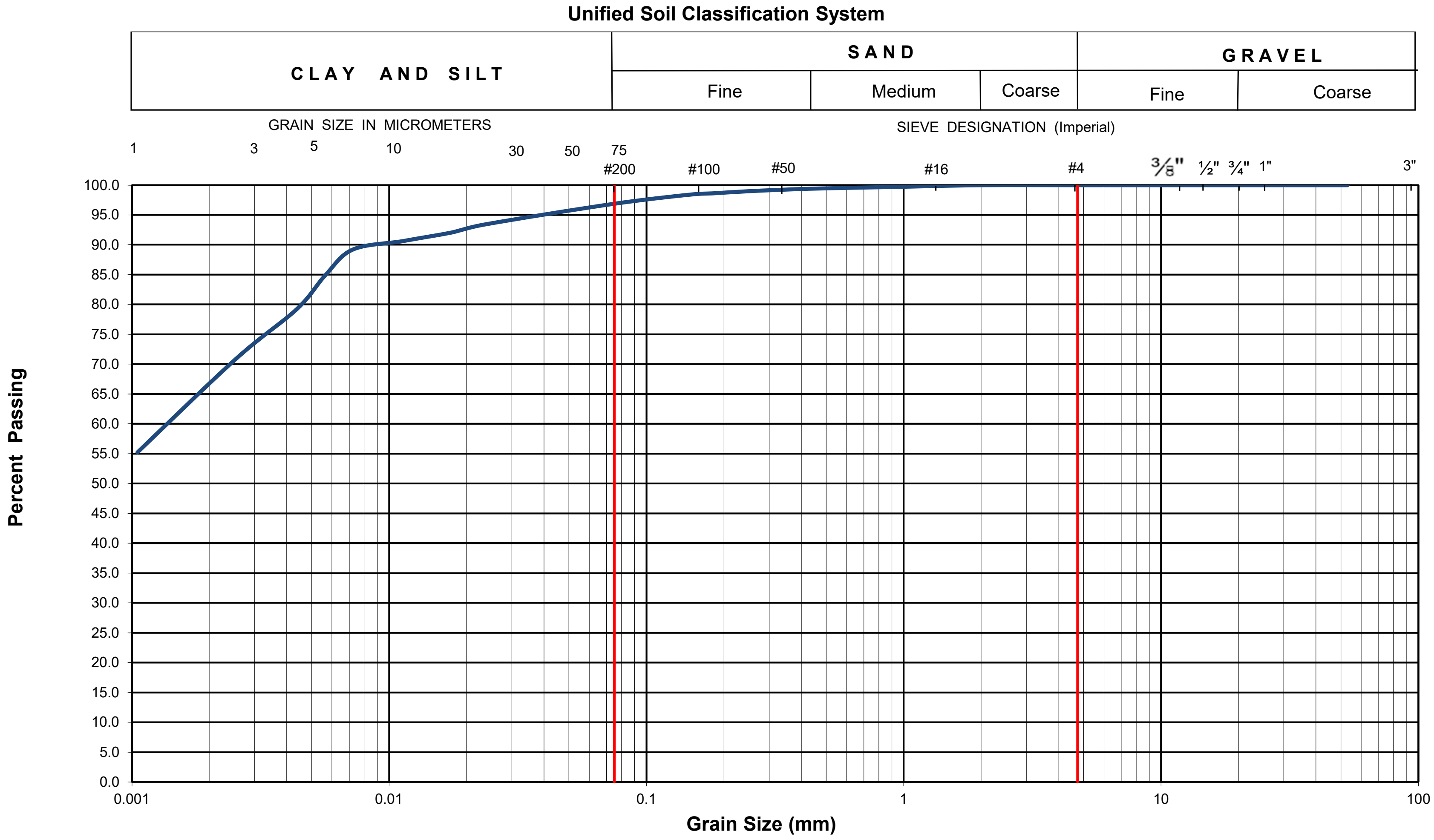
Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136



EXP Project No.:	OTT-21011499-C0	**Project Name :**	Proposed Multi-Use Towers				
Client :	780 Baseline Inc.	**Project Location :**	780 Baseline Road, Ottawa, ON				
Date Sampled :	April 14, 2022	**Borehole No:**	BH1	**Sample:**	AS1	**Depth (m) :**	0.1 - 0.7
Sample Composition :		**Gravel (%)**	45	**Sand (%)**	40	**Silt & Clay (%)**	15
Sample Description :	FILL: Silty Gravel with Sand (GM)					**Figure :**	10

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-21011499-C0**	**Project Name :**	**Proposed Multi-Use Towers**						
Client :	**780 Baseline Inc.**	**Project Location :**	**780 Baseline Road, Ottawa, ON**						
Date Sampled :	**April 11, 2022**	**Borehole No:**	**BH 2**	**Sample No.:**	**SS3**			**Depth (m) :**	**1.5-2.1**
Sample Description :		**% Silt and Clay**	**97**	**% Sand**	**3**	**% Gravel**	**0**	**Figure :**	**11**
Sample Description :	**Clay of High Plasticity (CH)**								

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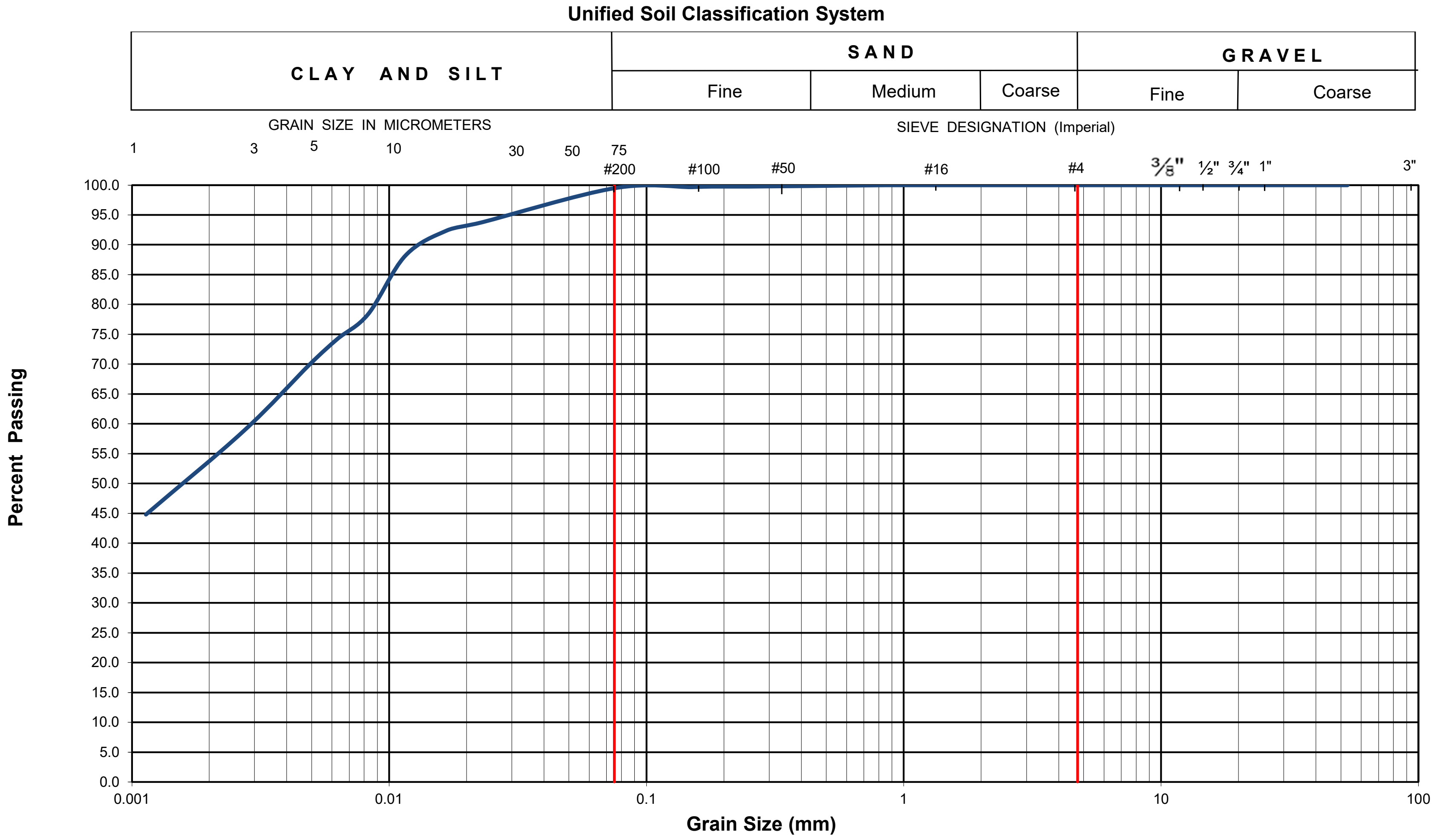
Ottawa, ON K2B 8H6

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



EXP Project No.:	**OTT-21011499-C0**	**Project Name :**	**Proposed Multi-Use Towers**				
Client :	**780 Baseline Inc.**	**Project Location :**	**780 Baseline Road, Ottawa, ON**				
Date Sampled :	**April 14, 2022**	**Borehole No:**	**BH 1**	**Sample No.:**	**SS5**	**Depth (m) :**	**4.6-5.2**
Sample Description :		**% Silt and Clay**	**100**	**% Sand**	**0**	**% Gravel**	**0**
Sample Description :	**Silty Clay of Low Plasticity (CL)**					**Figure :**	**12**

EXP Services Inc.
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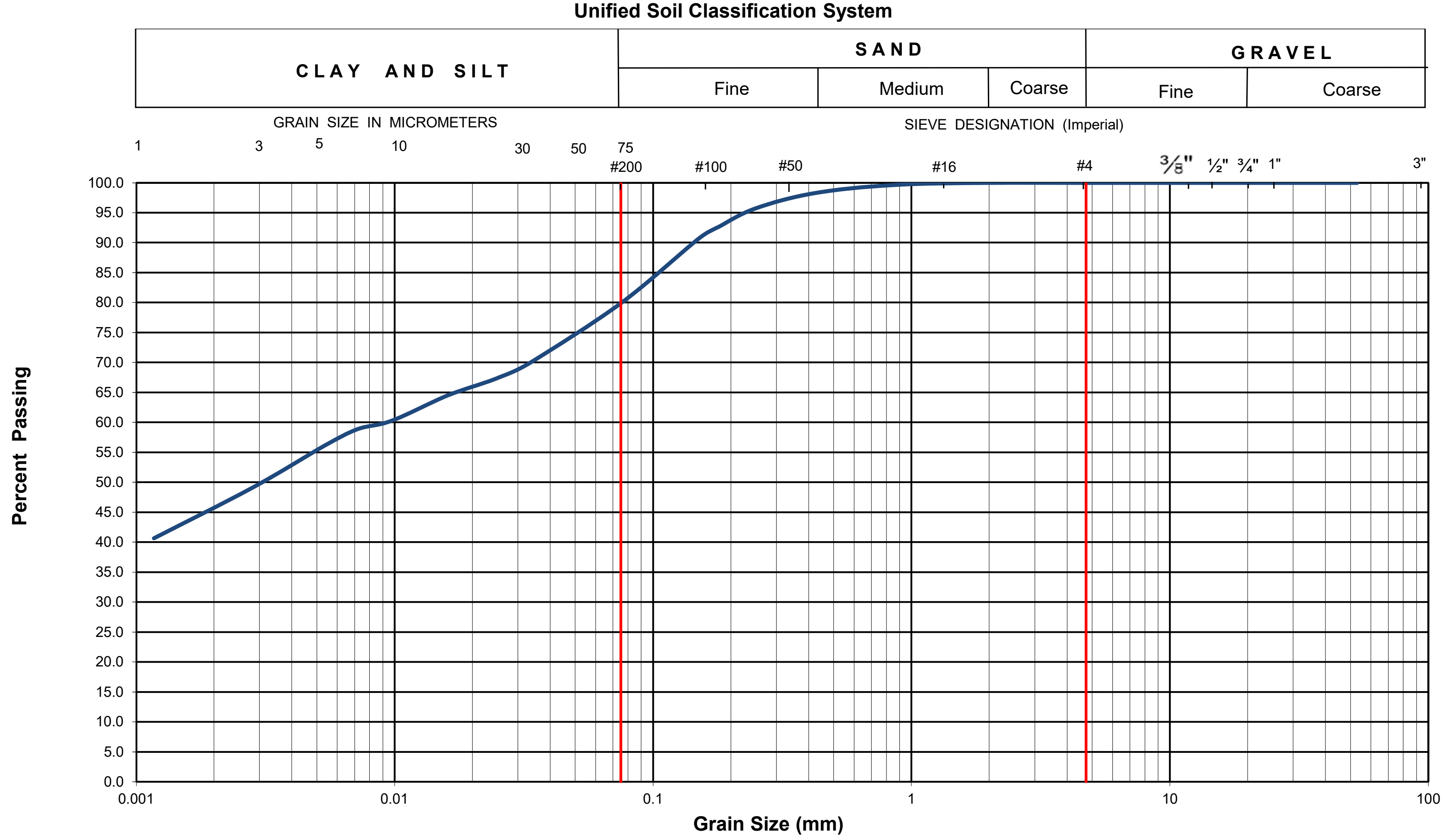
Ottawa, ON K2B 8H6

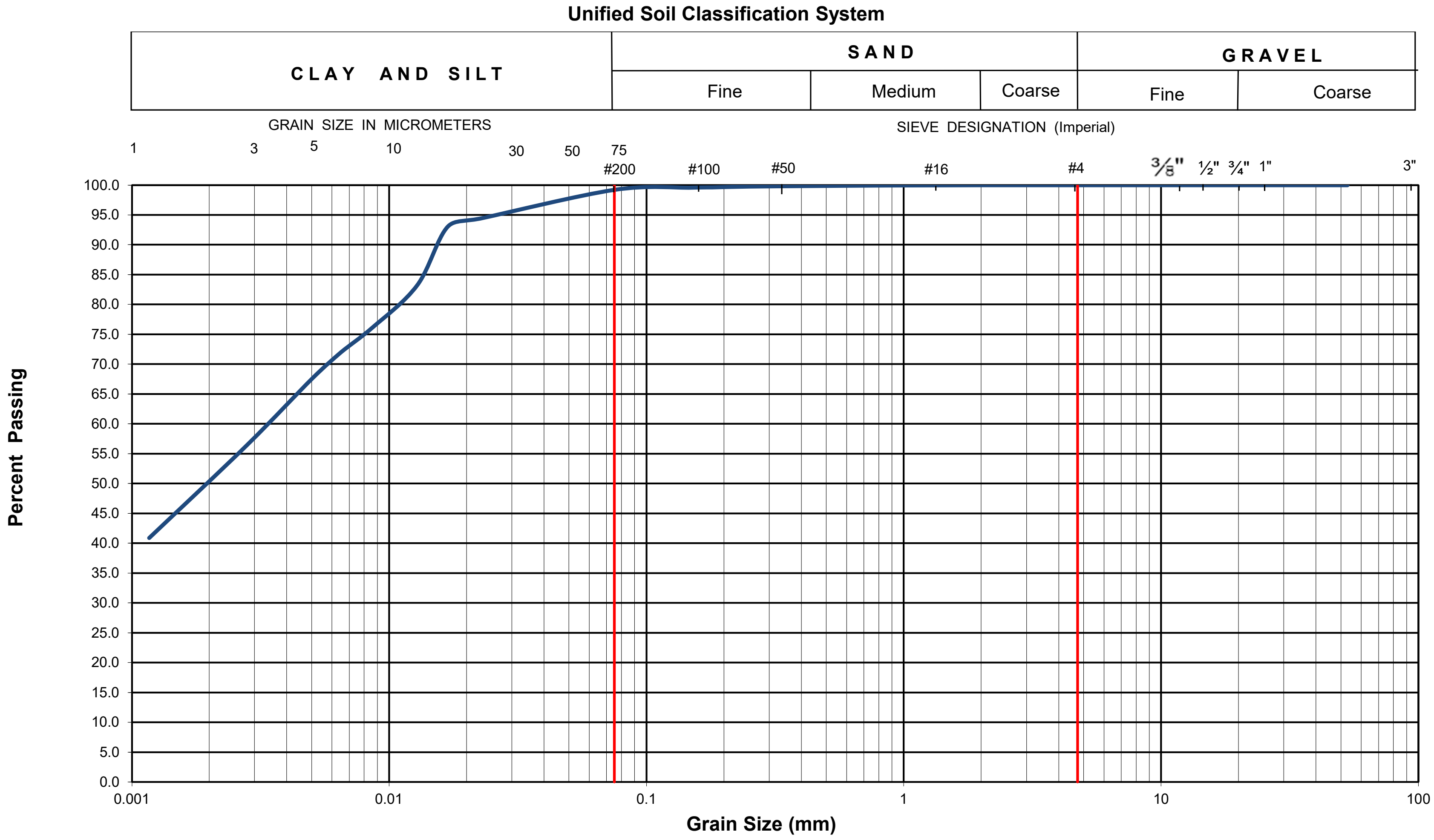
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-21011499-C0**	**Project Name :**	**Proposed Multi-Use Towers**				
Client :	**780 Baseline Inc.**	**Project Location :**	**780 Baseline Road, Ottawa, ON**				
Date Sampled :	**April 13, 2022**	**Borehole No:**	**BH 3**	**Sample No.:**	**SS4**	**Depth (m) :**	**3.0-3.6**
Sample Description :		**% Silt and Clay**	**80**	**% Sand**	**20**	**% Gravel**	**0**
Sample Description :	**Silty Clay of Low Plasticity with Sand (CL)**					**Figure :**	**13**

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-21011499-C0**	**Project Name :**	**Proposed Multi-Use Towers**				
Client :	**780 Baseline Inc.**	**Project Location :**	**780 Baseline Road, Ottawa, ON**				
Date Sampled :	**April 12, 2022**	**Borehole No:**	**BH 4**	**Sample No.:**	**SS6**	**Depth (m) :**	**6.1-6.7**
Sample Description :		**% Silt and Clay**	**99**	**% Sand**	**1**	**% Gravel**	**0**
Sample Description :		**Silty Clay of Low Plasticity (CL)**				**Figure :**	**14**

EXP Services Inc.
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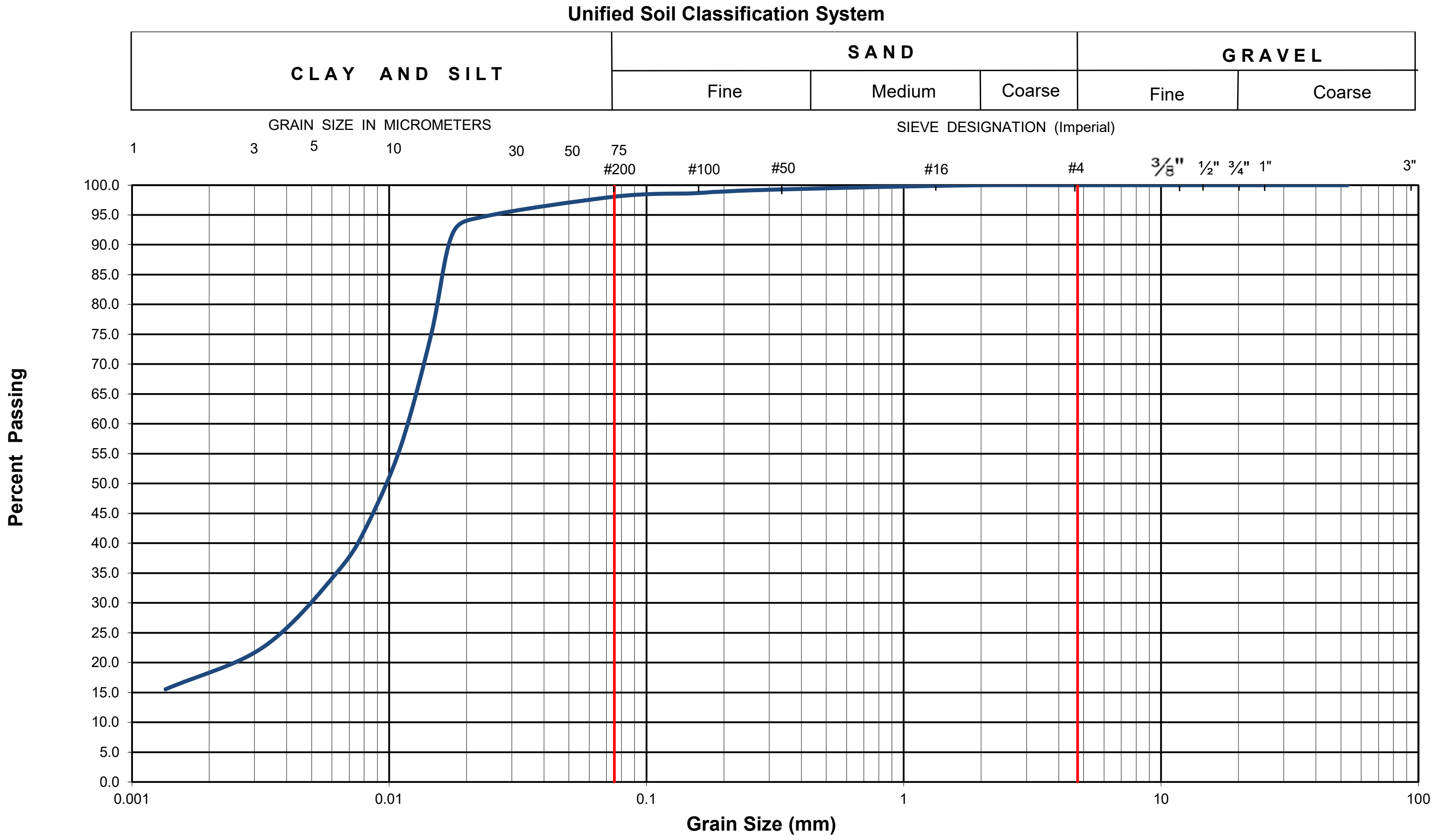
Ottawa, ON   K2B 8H6

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



EXP Project No.:	**OTT-21011499-C0**	**Project Name :**	**Proposed Multi-Use Towers**						
Client :	**780 Baseline Inc.**	**Project Location :**	**780 Baseline Road, Ottawa, ON**						
Date Sampled :	**April 12, 2022**	**Borehole No:**	**BH 5**	**Sample No.:**	**SS7**			**Depth (m) :**	**7.6-8.2**
Sample Description :		**% Silt and Clay**	**98**	**% Sand**	**2**	**% Gravel**	**0**	**Figure :**	**15**
Sample Description :	**Silt (ML)**								

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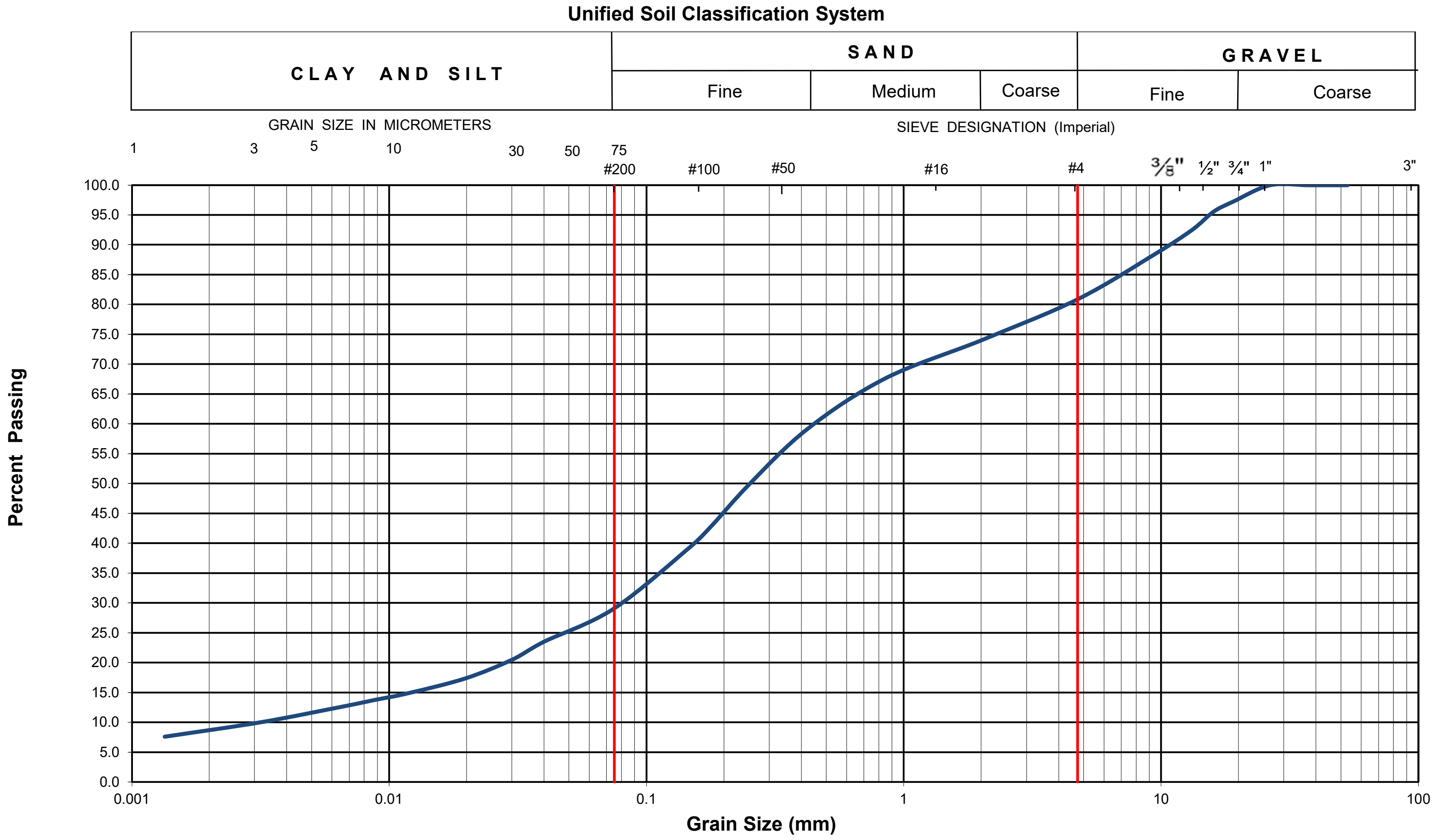
Ottawa, ON K2B 8H6

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



EXP Project No.:	**OTT-21011499-C0**	**Project Name :**	**Proposed Multi-Use Towers**					
Client :	**780 Baseline Inc.**	**Project Location :**	**780 Baseline Road, Ottawa, ON**					
Date Sampled :	**April 18, 2022**	**Borehole No:**	**BH 6**	**Sample No.:**	**SS9**		**Depth (m) :**	**10.7-11.3**
Sample Description :		**% Silt and Clay**	**29**	**% Sand**	**52**	**% Gravel** **19**	**Figure :**	**16**
Sample Description :		**Glacial Till: Silty Sand with Gravel (SM)**						

Appendix A – Seismic Shear Wave Velocity Sounding Survey Report by GPR

GEOPHYSICS GPR INTERNATIONAL INC.

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

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

June 8th, 2022

Transmitted by email: Ismail.Taki@exp.com
Our Ref.: GPR-22-03837b-01

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

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
780 Baseline Road, Ottawa (ON)

[Project: OTT-21011499-B0]

Dear Sir,

Geophysics GPR International inc. has been mandated by **exp** Services inc. to carry out seismic shear wave surveys at 780 Baseline Road, 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 May 19th, 2022, by Mr. Timothy Ward, tech., Louis-Emmanuel Warnock, tech. & Zak Castonguay, trainee. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the main seismic spread. 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 longer seismic acquisition spread was laid on a grassed strip, with a geophone spacing of 3.0 metres, 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 were produced with a seismograph Terraloc Pro 2 (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.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and 40 µs for the seismic refraction. The records included a pre-trigged portion of 10 ms. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

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

RESULTS

From seismic refraction (V_P), the rock depth was calculated at 12.5 metres (± 10 %). Its calculated seismic velocity (V_S) was 2095 m/s for its shallow portion.

The MASW calculated V_S results are illustrated at Figure 5. Some low seismic velocities were calculated between 1 and 5 to 7 metres deep.

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

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

(N: number of layers; H_i : thickness of layer "i" ; V_i : $\mathrm{V_S}$ of layer "i")

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

The calculated $\overline{V}_{S30}$ value of the actual site is 441.3 m/s (Table 1), corresponding to the Site Class "C". In the case there would be less than 3 metres between the rock and the bottom of the foundation, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" (Table 2).

CONCLUSION

Geophysical surveys were carried out to identify the Site Class at 780 Baseline Road, in Ottawa (ON). The seismic surveys used the MASW and the SPAC analysis, and 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 441 m/s, corresponding to the Site Class "C" ($360 < \overline{V}_{S30} \leq 760$ 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.

Some low seismic velocities were calculated between 1 and 5 to 7 metres deep. A geotechnical assessment of the corresponding materials could be required for the potential of liquefaction, the clay degree of sensitivity and other critical parameters.

In the case there would be less than 3 metres of unconsolidated material between the rock surface and the bottom of the foundation, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" ($\overline{V}_{S30} > 1500$ m/s).

It must 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

LICENSED PROFESSIONAL ENGINEER
J.-L. ARSENAULT
100148122
PROVINCE OF ONTARIO
2022-06-08

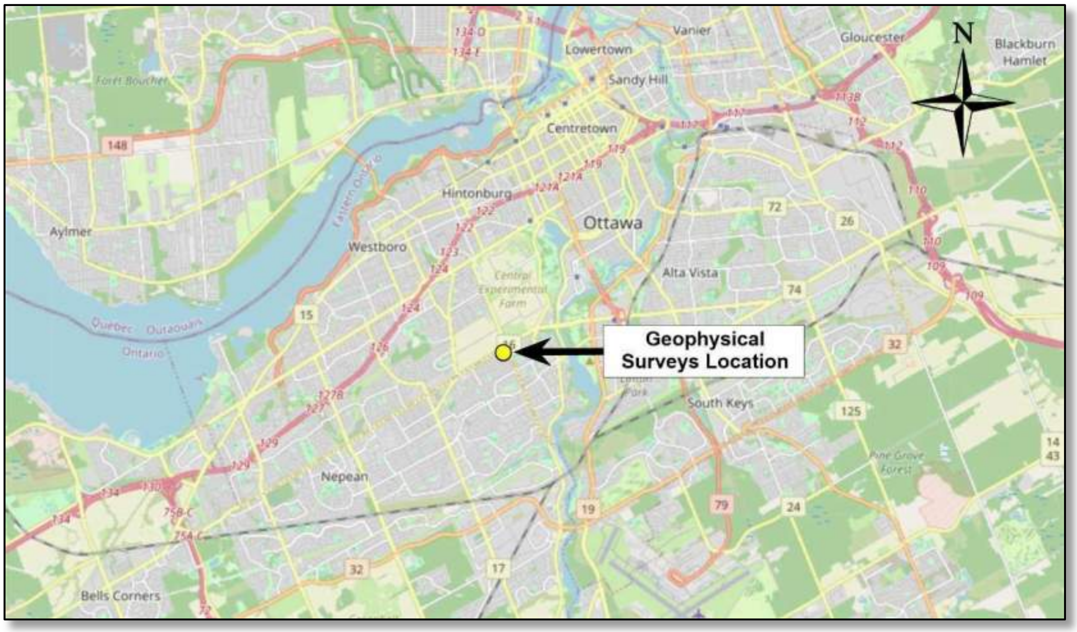


Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

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

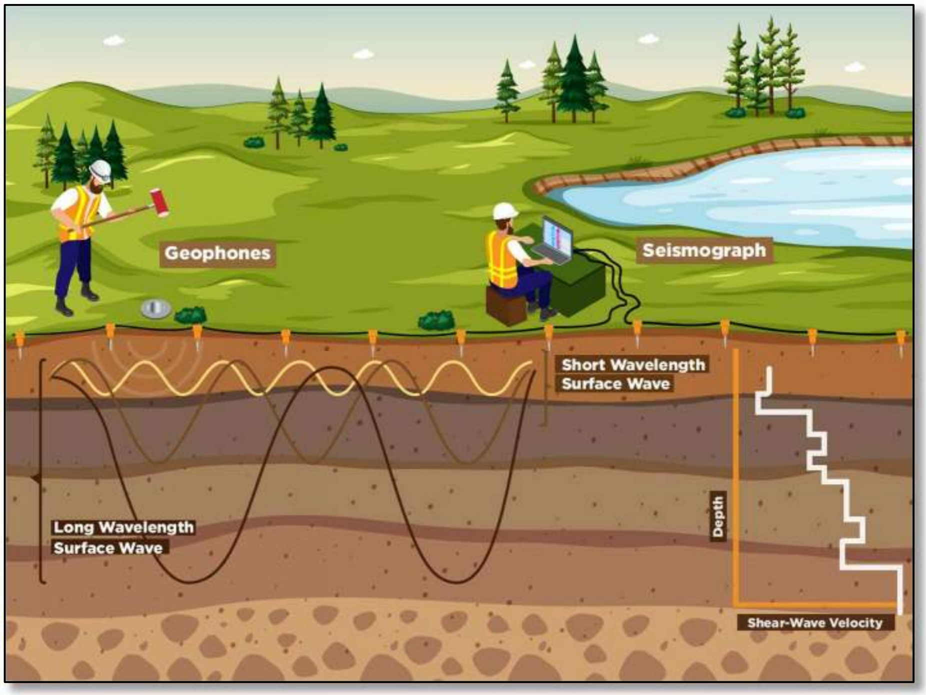
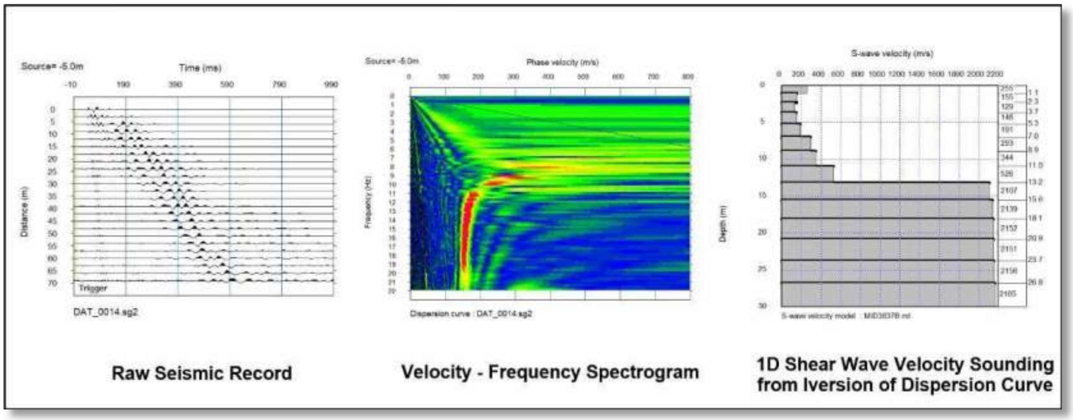


Figure 3: MASW Operating Principle



Raw Seismic Record

Velocity - Frequency Spectrogram

1D Shear Wave Velocity Sounding from Iversion of Dispersion Curve

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

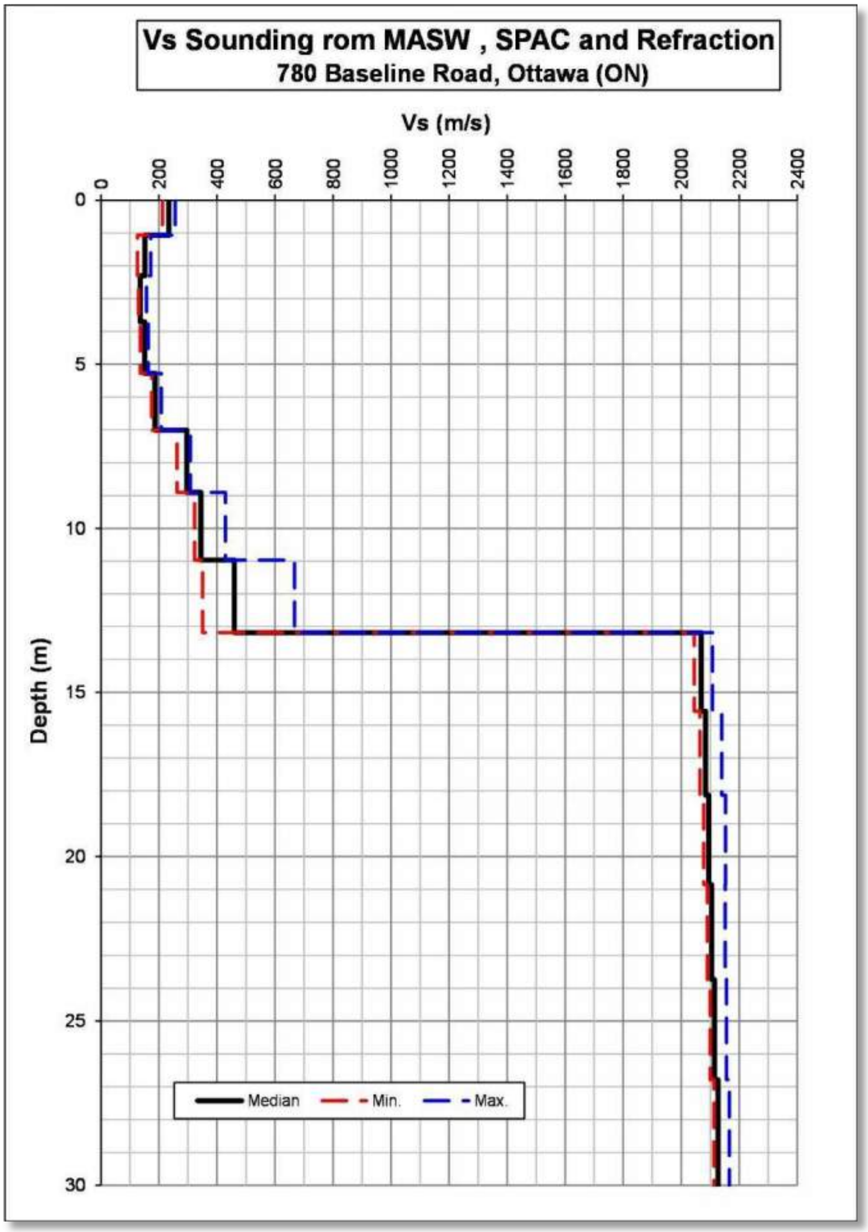


Figure 5: MASW Shear-Wave Velocity Sounding

TABLE 1

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

Depth	Vs			Thickness	Cumulative Thickness	Delay for Med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	212.7	234.3	255.9	**Grade Level (May 19, 2022)**				
1.07	**126.4**	**151.5**	**171.6**	1.07	1.07	0.004572	0.004572	234.3
2.31	**130.0**	**135.7**	**157.1**	1.24	2.31	0.008162	0.012734	181.2
3.71	**135.3**	**150.5**	**162.9**	1.40	3.71	0.010327	0.023060	160.8
5.27	**175.1**	186.3	208.3	1.57	5.27	0.010407	0.033468	157.6
7.01	262.8	296.2	308.2	1.73	7.01	0.009289	0.042757	163.8
8.90	322.6	344.4	429.5	1.90	8.90	0.006399	0.049156	181.1
10.96	350.5	460.0	667.1	2.06	10.96	0.005983	0.055139	198.8
13.19	2045.2	2068.6	2107.8	2.23	13.19	0.004837	0.059976	219.9
15.58	2064.3	2084.2	2139.9	2.39	15.58	0.001155	0.061132	254.8
18.13	2077.5	2094.9	2152.9	2.55	18.13	0.001226	0.062357	290.8
20.85	2089.5	2105.0	2151.4	2.72	20.85	0.001298	0.063656	327.6
23.74	2100.3	2114.5	2156.1	2.88	23.74	0.001370	0.065026	365.0
26.79	2113.3	2126.8	2165.9	3.05	26.79	0.001442	0.066468	403.0
30				3.21	30.00	0.001511	0.067980	441.3

V_{S30} (m/s)	**441.3**
Class	**C (1)**

(1) A geotechnical assessment could be required for the materials between 1 and 5 to 7 metres deep, for the potential of liquefaction, the degree of clay sensitivity and other critical parameters.

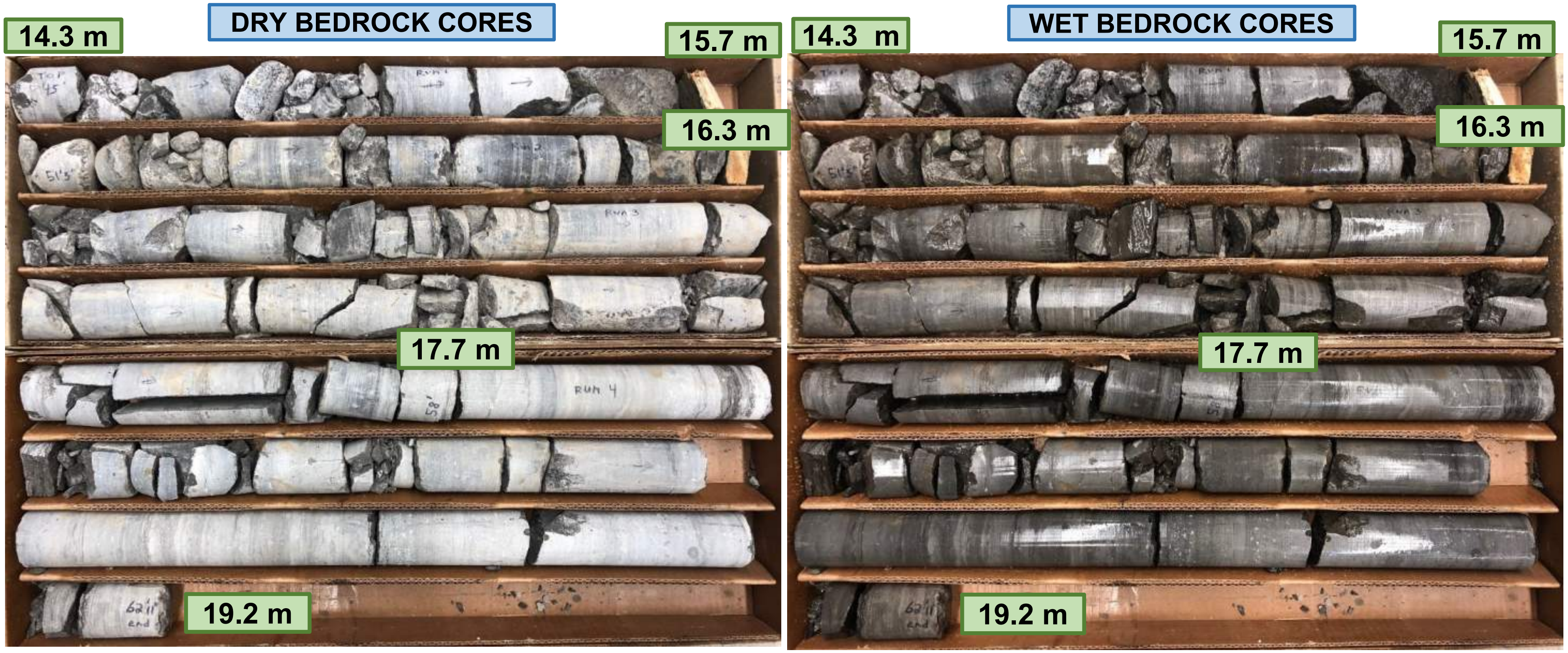
TABLE 2

V_{S30}* Calculation for less than 3 metres of soil below the foundations

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	212.7	234.3	255.9	**Less than 3 metres of soil**				
1.07	126.4	151.5	171.6					
2.31	130.0	135.7	157.1					
3.71	135.3	150.5	162.9					
5.27	175.1	186.3	208.3					
7.01	262.8	296.2	308.2					
8.90	322.6	344.4	429.5					
10.20	322.6	344.4	429.5					
10.96	350.5	460.0	667.1	0.76	0.76	0.002221	0.002221	344.4
13.19	2045.2	2068.6	2107.8	2.23	2.99	0.004837	0.007058	423.6
15.58	2064.3	2084.2	2139.9	2.39	5.38	0.001155	0.008213	655.0
18.13	2077.5	2094.9	2152.9	2.55	7.94	0.001226	0.009439	840.7
20.85	2089.5	2105.0	2151.4	2.72	10.65	0.001298	0.010737	992.3
23.74	2100.3	2114.5	2156.1	2.88	13.54	0.001370	0.012108	1118.2
26.79	2113.3	2126.8	2165.9	3.05	16.59	0.001442	0.013550	1224.3
40.20				13.41	30.00	0.006306	0.019856	1510.9

V_{S30}* (m/s)	**1510.9**
Class	**A**

Appendix B – Bedrock Core Photographs



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

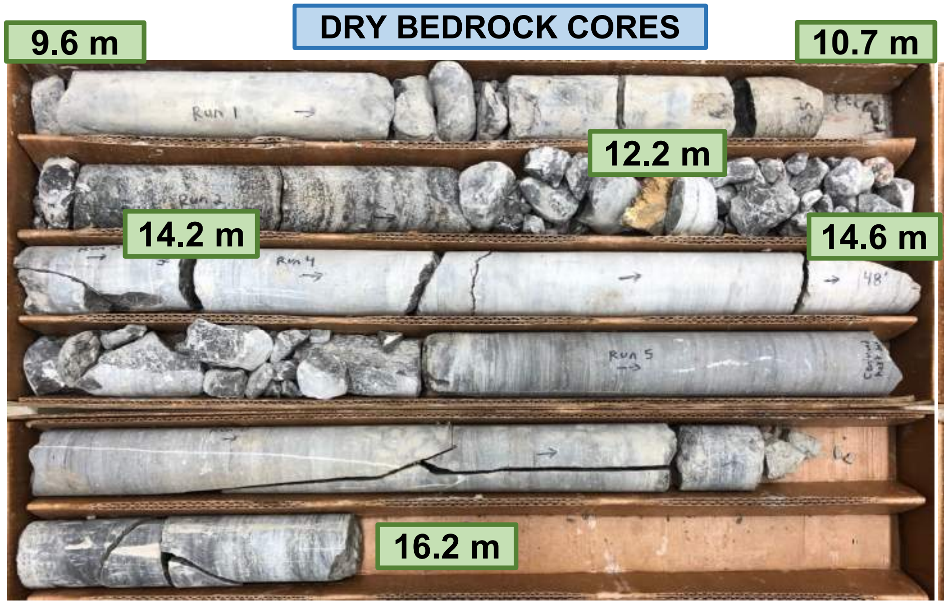
Borehole No: **BH1**	Core Runs Run 1: 14.3 m - 15.7 m Run 2: 15.7 m - 16.3m Run 3: 16.3 m - 17.7 m Run 4: 17.7 m - 19.2 m	project Geotechnical Investigation - Proposed Multi-Use Towers. 780 Baseline Road, Ottawa, Ontario.	Project N0: OTT-21011499-C0
Date Cored Apr 14, 2022		Rock Core Photographs	FIG B-1

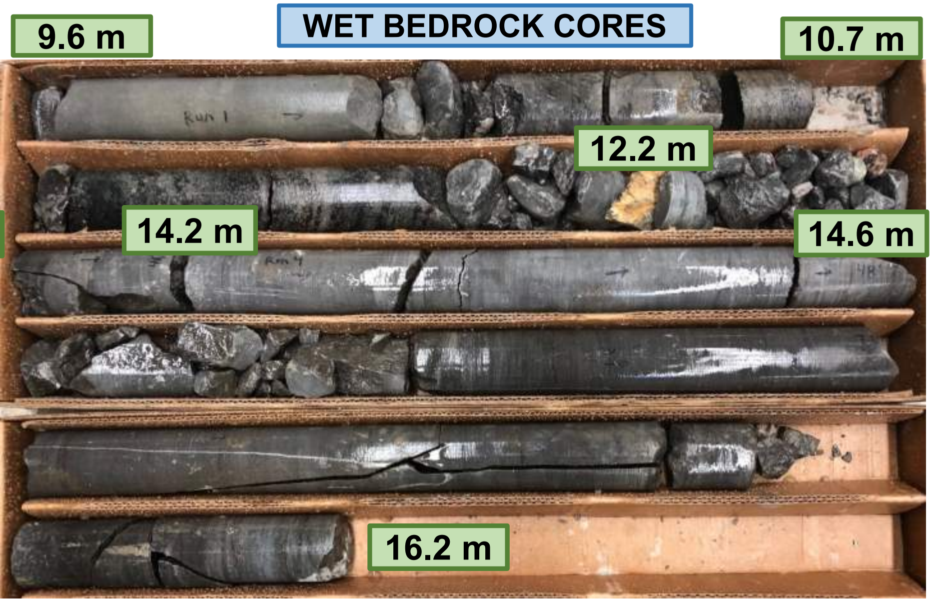
DRY BEDROCK CORES

WET BEDROCK CORES

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

Borehole No: **BH3**	Core Runs Run 1: 10.8 m - 11.6 m Run 2: 11.6 m - 13.2 m	project Geotechnical Investigation - Proposed Multi-Use Towers. 780 Baseline Road, Ottawa, Ontario.	Project N0: OTT-21011499-C0
Date Cored Apr 13, 2022		Rock Core Photographs	FIG B-2





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

Borehole No:
BH4

Date Cored
Apr 12, 2022

Core Runs
Run 1: 9.6 m - 10.7 m
Run 2: 10.7 m - 12.2 m
Run 3: 12.2 m - 14.2 m
Run 4: 14.2 m - 14.6 m
Run 5: 14.6 m - 16.2 m

project
Geotechnical Investigation - Proposed Multi-Use Towers.
780 Baseline Road, Ottawa, Ontario.

Rock Core Photographs

Project N0:
OTT-21011499-C0

FIG B-3

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

Borehole No:
BH6

Core Runs
Run 1: 12.2 m - 13.7 m
Run 2: 13.7 m - 15.2 m
Run 3: 15.2 m - 16.6 m

project
Geotechnical Investigation - Proposed Multi-Use Towers.
780 Baseline Road, Ottawa, Ontario.

Project N0:
OTT-21011499-C0

Date Cored
Apr 18, 2022

Rock Core Photographs

FIG B-4

Appendix C – Laboratory Certificate of Analysis

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: Daniel Wall

PROJECT: OTT-21011499-CO

AGAT WORK ORDER: 22Z888170

SOIL ANALYSIS REVIEWED BY: Jacky Zhu, Spectroscopy Technician

DATE REPORTED: May 03, 2022

PAGES (INCLUDING COVER): 5

VERSION*: 1

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

Notes

Disclaimer:

- *All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.*
- *All samples will be disposed of within 30 days after receipt unless a Long Term Storage Agreement is signed and returned. Some specialty analysis may be exempt, please contact your Client Project Manager for details.*
- *AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.*
- *This Certificate shall not be reproduced except in full, without the written approval of the laboratory.*
- *The test results reported herewith relate only to the samples as received by the laboratory.*
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Western Enviro-Agricultural Laboratory Association (WEALA)
Environmental Services Association of Alberta (ESAA)

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Certificate of Analysis

AGAT WORK ORDER: 22Z888170

PROJECT: OTT-21011499-CO

CLIENT NAME: EXP SERVICES INC

SAMPLING SITE:780 Baseline Rd., Ottawa

ATTENTION TO: Daniel Wall

SAMPLED BY:EXP

Inorganic Chemistry (Soil)

DATE RECEIVED: 2022-04-26

DATE REPORTED: 2022-05-03

			SAMPLE DESCRIPTION:	BH#1 SS11 45'-47'	BH#6 run 2 48'10"-49'4"
			SAMPLE TYPE:	Soil	Soil
			DATE SAMPLED:	2022-04-14	2022-04-18
Parameter	**Unit**	**G / S**	**RDL**	**3789955**	**3789956**
Chloride (2:1)	µg/g		2	49	19
Sulphate (2:1)	µg/g		2	125	101
pH (2:1)	pH Units		NA	8.04	8.70
Resistivity (2:1) (Calculated)	ohm.cm		1	3130	2910

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

Analysis performed at AGAT Toronto (unless marked by *)

Certified By: Niandong Zhu

CHARTERED NIANDONG ZHU CHEMIST

Quality Assurance

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-21011499-CO
SAMPLING SITE:780 Baseline Rd., Ottawa

AGAT WORK ORDER: 22Z888170
ATTENTION TO: Daniel Wall
SAMPLED BY:EXP

Soil Analysis

RPT Date: May 03, 2022			**DUPLICATE**			**Method Blank**	**REFERENCE MATERIAL**			**METHOD BLANK SPIKE**			**MATRIX SPIKE**		
PARAMETER	**Batch**	**Sample Id**	**Dup #1**	**Dup #2**	**RPD**		**Measured Value**	**Acceptable Limits**		**Recovery**	**Acceptable Limits**		**Recovery**	**Acceptable Limits**	
								Lower	**Upper**		**Lower**	**Upper**		**Lower**	**Upper**
Inorganic Chemistry (Soil)															
Chloride (2:1)	3798056		180	179	0.6%	< 2	97%	70%	130%	99%	80%	120%	102%	70%	130%
Sulphate (2:1)	3798056		857	864	0.8%	< 2	103%	70%	130%	100%	80%	120%	NA	70%	130%
pH (2:1)	3801168		6.21	6.49	4.4%	NA	99%	80%	120%						

Comments: NA Signifies Not Applicable.
Duplicate NA: results are less than 5X the RDL and RPD will not be calculated.
Matrix spike: Spike level < native concentration. Matrix spike acceptance limits do not apply.

Certified By: Xiandong Zhu

Method Summary

CLIENT NAME: EXP SERVICES INC

PROJECT: OTT-21011499-CO

SAMPLING SITE:780 Baseline Rd., Ottawa

AGAT WORK ORDER: 22Z888170

ATTENTION TO: Daniel Wall

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	modified from EPA 9045D and MCKEAGUE 3.11	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

Laboratory Use Only

Work Order #: 22Z888170

Cooler Quantity: 1 bag

Arrival Temperatures: 24.0 24.1 | 24.0
4.9 | 4.1 | 4.7

Custody Seal Intact: ☐ Yes ☐ No ☐ N/A

Notes: Ice Packs

Chain of Custody Record

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

Report Information:

Company: EXP Services Inc. Ottawa
Contact: Daniel Wall
Address: 2650 Queensview dr. Unit 100
Ottawa, ON K2B 8H6
Phone: 613-688-1899 Fax:
Reports to be sent to:
1. Email: daniel.wall@exp.com
2. Email:

Regulatory Requirements:

(Please check all applicable boxes)

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

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

☐ Sewer Use
☐ Sanitary ☐ Storm
___ *Region*
☐ Prov. Water Quality Objectives (PWQO)
☐ Other
___ *Indicate One*

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

Report Guideline on Certificate of Analysis
☐ Yes ☐ No

Turnaround Time (TAT) Required:

Regular TAT (Most Analysis) ☑ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

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

OR Date Required (Rush Surcharges May Apply):

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

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

Project Information:

Project: OTT-21011499-CO
Site Location: 780 Baseline rd. Ottawa
Sampled By: EXP
AGAT ID #: PO:

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

Invoice Information:

Bill To Same: Yes ☑ No ☐

Company:
Contact:
Address:
Email:

Sample Matrix Legend

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

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI, DOC Y / N	O. Reg 153 Metals & Inorganics	O. Reg 153 Metals - ☐ CrVI, ☐ Hg, ☐ HWSB	O. Reg 153 BTEX, F1-F4 PHCs Analyze F4G if required ☐ Yes ☐ No	PAHs	Total PCBs ☐ Aroclor	VOC	O. Reg 558 Landfill Disposal Characterization TCLP: TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	O. Reg 406 Excess Soils SPLP Rainwater Leach SPLP: ☐ Metals ☐ VOCs ☐ SVOCs	O. Reg 406 Excess Soils Characterization Package pH, ICPMS Metals, BTEX, F1-F4	Salt - EC/SAR	pH	sulphate	electrical resistivity	chloride				Potentially Hazardous or High Concentration (Y/N)
BH #1 SS11 45'-47'	Apr.14/22	/ AM PM	1	S													✓	✓	✓	✓				
BH #6 run 2 48'10"-49'4"	Apr.18/22	/ AM PM	1	rock													✓	✓	✓	✓				
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Jeff MacMillan	Apr.26/22	1:30	C.	26/04/22	13h35
CC to Burlington	26/04/22	16h00	Armando Rivas		

Page 1 of 1

Nº: T114962

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