Geotechnical Investigation

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Ottawa, Ontario

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Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed high-rise development to be located at 116 Beech Street in Ottawa, Ontario. Terms and conditions of this assignment were outlined in EXP Services Inc. (EXP) proposal OTT-23005943-A0 dated May 9, 2023. Authorization to proceed with this work was provided by Katasa Groupe on May 16, 2023 under Purchase Order (PO) No. 25-100523.

A Phase II Environmental Site Assessment (ESA) was completed in parallel with the geotechnical investigation and is reported under a separated cover.

The subject site is currently occupied by a three-storey residential building with surface parking which will be demolished to allow the construction of a 25-storey building with 407 rental units. The building will have two basement parking levels.

The fieldwork for this geotechnical investigation was undertaken between June 6 and June 7, 2023 and consists of five (5) boreholes (Borehole Nos. 1 to 5) advanced to auger refusal and termination depths ranging from 3.1 m to 9.2 m (Elevation 62.3 m to Elevation 56.3 m) below the existing grade. Thirty-two (32) mm monitoring wells, with screened sections, were installed in selected boreholes for long-term monitoring of the groundwater levels. A seismic shear wave velocity sounding survey was completed to establish the seismic class of the proposed high-rise development.

The boreholes indicate the site is surficially underlain by an asphaltic concrete/topsoil and in turn underlain by fill, silty sand/sandy silt and/or silty clay. Glacial till was contacted in Borehole Nos. 2 to 5 at depths of 0.7 m to 3.5 m (Elevation 64.7 m to Elevation 63.2 m). Auger refusal was met in all the boreholes at 1.7 m to 6.0 m depths (Elevation 63.1 m to Elevation 61.8 m). Limestone bedrock was confirmed by core drilling techniques at 1.7 m to 6.0 m (Elevation 63.1 m to Elevation 618 m) in Borehole Nos.1, 2 and 4. The groundwater was established to range between 1.8 m and 3.5 m depth (Elevation 64.7 m to Elevation 61.8 m).

It is our understanding that two (2) levels of underground parking will be provided to the proposed high-rise structure and therefore, and it has been assumed that footings of the proposed building will be set at approximately 6.0 m (or deeper) below the existing ground surface. Based on a review of the borehole logs the footings will be founded in the limestone bedrock.

Spread and strip footings founded on the sound limestone bedrock, competent and free of soil filled seams may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 3000 kPa. The factored ULS value includes a resistance factor of 0.5. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement of the structure will be much larger than the recommended value for factored geotechnical resistance at ULS. Therefore, the factored geotechnical resistance at ULS will govern the design. Settlements of footing designed for the above recommended factored geotechnical resistance at ULS and properly constructed are expected to be less than 10 mm.

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

A seismic site response of Class A can be used for the proposed high rise building to be founded within the sound limestone bedrock.

The lowest floor level for the parking garages is anticipated to be located below the groundwater level. Therefore, underfloor and perimeter drainage systems will be required for the proposed below grade parking garages. All subsurface structures should be waterproofed.

Excavation of the soils may be undertaken using heavy equipment capable of removing debris as well as cobbles, boulders and within fill or the glacial till. All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils on site are considered to be Type 3 and as such must be cut back at 1H:1V from the bottom of the excavation above the groundwater level. Within zones of persistent seepage and below the groundwater level in the soils, the excavation side slopes are expected to slough and eventually stabilize at a slope of 2H:1V to 3H:1V. The excavations will extend into the limestone bedrock. The excavation side slopes in the upper depths of with weathered/highly fractured zones (if encountered) of the bedrock may be cut back at a 1H:1V gradient.

The excavation side slopes in the sound bedrock may be undertaken with near vertical sides subject to examination by a geotechnical engineer.

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

The excavation of the sound limestone bedrock to extensive depths below the bedrock surface is anticipated to require line drilling and blasting. Hoe ramming can be used for small quantities of rock but is a slow process. 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.

The overburden soils to be excavated from the site are not considered suitable for reuse as backfill material in the interior or exterior of the buildings and it is anticipated that the majority of the material required for backfilling purposes will need to be imported and should preferably conform to OPSS 1010 (as amended by SSP110S13) for Granular B Type II.

A hydrogeological study must be completed as part of the final design to establish the quantity of water to be pumped as well as any potential influence of the construction on neighboring properties so appropriate design steps can be implemented as part of the construction and design, i.e. shoring, drainage, etc.

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

1. Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed high-rise development to be located at 116 Beech Street in Ottawa, Ontario. Terms and conditions of this assignment were outlined in EXP Services Inc. (EXP) proposal OTT-23005943-A0 dated May 9, 2023. Authorization to proceed with this work was provided by Katasa Groupe on May 16, 2023, under Purchase Order (PO) No. 25-100523.

A Phase II Environmental Site Assessment (ESA) was completed in parallel with the geotechnical investigation and is reported under a separate cover.

The subject site is currently occupied by a three-storey residential building with surface parking which will be demolished to allow the construction of a 25-storey building with 407 Rental Units. The building will have two basement parking levels.

This geotechnical investigation was undertaken to:

a) Establish the subsurface soil, bedrock and groundwater conditions at the five (5) borehole locations;

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

c) Comment on grade-raise restrictions;

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

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

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

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

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

2. Site Description

The Site is located on the southeast corner of the intersection of Beech Street and Loretta Avenue South. The Site is rectangular in shape with an approximate area of 0.56 hectares (1.4 acres) and is currently occupied by a three-storey apartment. The existing building is in an L shape. To the east of the existing building is a surface parking lot. To the west of the building is a landscaped area. A Site Location Plan is provided as Figure 1.

The topography of the site generally slopes downwards from the west to the east. The ground surface elevations at the borehole locations ranges from 67.75 m to 64.83 m.

3. Geology of the Site

3.1 Surficial Geology

The surficial geology was reviewed via the Google Earth applications published by the Ontario Ministry of Energy, Northern Development and Mines available via www.mndm.gov.on.ca/en/mines-and-minerals/applications/ogsearth/surficial-geology and was last modified on May 23, 2017. The map indicates that beneath any fill the site is underlain by stone-poor, sandy silt to silty sand-textured glacial till or Paleozoic bedrock. The surficial deposits are shown in Image 1 below.

Image 1 – Surficial Geology

3.2 Bedrock Geology

The bedrock geology was reviewed via the Google Earth applications published by the Ontario Ministry of Energy, Northern Development and Mines available via http://www.geologyontario.mndm.gov.on.ca/mines/data/google/MRD219/geology/doc.kml, published in 2007. The map indicates the bedrock is either limestone of the Lindsay or Bobcaygeon formation with minor shale upper part of the Bobcaygeon formation.

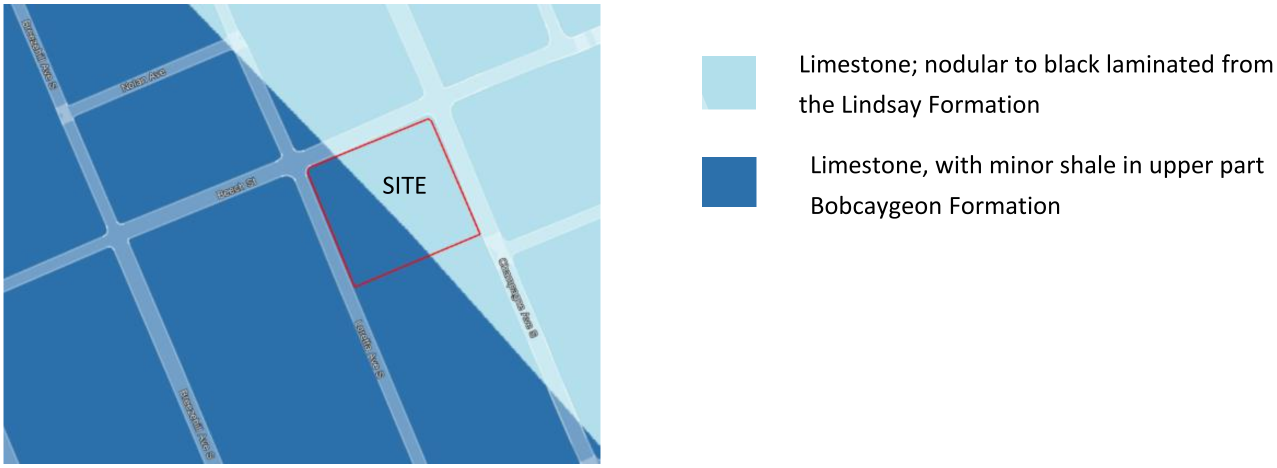

Image 2 – Bedrock Geology

4. Procedure

4.1 Fieldwork

The fieldwork for this geotechnical investigation was undertaken between June 6 and June 7, 2023 and consists of five (5) boreholes (Borehole Nos. 1 to 5) advanced to auger refusal and termination depths ranging from 3.1 m to 9.2 m (Elevation 62.3 m to Elevation 56.3 m) below the existing grade.

The locations and geodetic elevations of the boreholes were surveyed by EXP. 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-75 truck mounted drill rig equipped with continuous flight hollow stem augers and the capability to sample soil and bedrock. Standard penetration tests (SPTs) were performed in the boreholes at 0.75 m depth intervals with soil samples retrieved by the split-barrel sampler. Auger samples were also taken from below the asphaltic concrete surface. The undrained shear strength of the clayey soils were measured by conducting pocket penetrometer tests. The bedrock was cored in Borehole Nos. 1, 2 and 4 by conventional rock coring methods. A careful record of any sudden drops of the core barrel, colour of the wash water and wash water return were recorded during the rock coring operation.

The subsurface soil conditions in each borehole were logged with each soil sample placed in a labelled plastic bag. The bedrock cores were also logged and stored in core boxes and identified.

Thirty-two (32) mm monitoring wells, with screened sections, were installed in selected boreholes for long-term monitoring of the groundwater levels. The monitoring wells were installed in accordance with EXP standard practice, and the installation configuration is documented on the respective borehole log. The boreholes were backfilled upon completion of the field work and the installation of the monitoring wells.

On completion of the fieldwork, the soil and rock samples were transported to the EXP laboratory in Ottawa.

4.2 Laboratory Testing Program

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

A summary of the soil 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	40
Unit Weight Determination	2
Grain Size Analysis	4
Atterberg Limit Determination	1
Bedrock Cores	
Unit Weight Determination	3
Unconfined Compressive Strength Test	3

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

A seismic shear wave survey of the site was undertaken by Geophysics GPR International Inc. (GPR) on June 14, 2023. The purpose of the survey is to determine the seismic shear wave velocity of the site from the existing ground surface to 30.0 m depth and based on the results of the survey, provide the classification of the site for seismic response. The location of the seismic survey line is shown in Figure No. 2. The seismic shear wave survey report is attached in Appendix A.

5. Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface conditions and groundwater levels from this geotechnical investigation are given on the attached Borehole Logs, Figure Nos. 3 to 7 inclusive. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

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

It should be noted that the soil 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 report.

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

5.1 Asphaltic Concrete Pavement

The pavement structure encountered in Borehole Nos. 1 to 3, drilled in the existing parking lot, consists of a 50 mm to 100 mm thick surficial asphaltic concrete layer underlain by a 100 mm to 360 mm thick granular fill layer that extends to depths ranging from 0.2 m to 0.46 m below the existing grade (Elevation 65.1 m to Elevation 64.3 m). The granular fill layer generally consists of silty sand with gravel.

The results from the grain-size analysis conducted on one (1) sample of the granular fill are summarized in Table II. The grain-size distribution curve is shown in Figure 8.

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

Borehole No. (BH) – Sample Spoon No. (SS)	Depth (m)	Grain-Size Analysis (%)			
		Gravel	Sand	Fines	Soil Classification (USCS)
BH2 – SS1	0.1 - 0.8	18	62	20	Silty Sand with Gravel (SM)

Based on a review of the results of the grain-size analysis the granular fill may be classified Silty Sand with Gravel (SM) in accordance with the USCS.

5.2 Topsoil

A 225 mm to 250 mm thick topsoil layer was surficially contacted in Boreholes Nos. 4 to 5, drilled in the grass covered landscaped areas.

5.3 Fill

A layer of fill was encountered underlying either the asphaltic concrete pavement structure or the topsoil layer and extends to depths of 0.7 m to 1.8 m (Elevation 66.0 m to Elevation 63.5 m). The fill generally consists of a mixture of silt and sand. The SPT N-values of 5 to 21 indicates a loose to compact state. The moisture content of the fill ranges from 6 percent to 17 percent. The unit weight of one sample of the silty clay was 23.2 kN/m^3.

The results from the grain-size analysis conducted on one (1) sample of the fill are summarized in Table III. The grain-size distribution curve is shown in Figure 9.

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

Borehole No. (BH) – Sample Sample No. (SS)	Depth (m)	Grain-Size Analysis (%)			
		Gravel	Sand	Fines	Soil Classification (USCS)
BH2 SS2	0.8-1.4	3	38	59	Silt with Sand (ML)

Based on a review of the results of the grain-size analysis the fill may be classified a silt with sand (ML) in accordance with the USCS.

5.4 Silty Sand/Sandy Silt

A layer which ranged from silty sand to sandy silt was contacted below the fill in Borehole Nos. 1, 3 and 4 at depths of 1.4 m to 1.8 m (Elevation 66.0 m to Elevation 63.5m). The SPT N-values of with the silty sand/sandy silt ranges from 8 to 13 indicating a loose to compact condition. The natural moisture content of the silty sand/sandy silt ranges from 15 percent to 27 percent.

5.5 Silt Clay

A silty clay layer was encountered in Boreholes Nos. 4 and 5 underlying the fill or silty sand/sandy silt layer. The silty clay extends to depths of 2.6 m to 3.5 m (Elevation 64.7 m to Elevation 64.3 m). The undrained shear strength of the silty clay ranges from 24 kPa to 36 kPa indicating a soft to firm consistency. The natural moisture content of the silty clay ranges from 22 percent to 34 percent. The unit weight of one sample of the silty clay was 20.1 kN/m^3.

The results from the grain-size analysis and Atterberg limit determination conducted on one (1) samples of the silty clay are summarized in Table IV. The grain-size distribution curves is shown in Figure 10.

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

Borehole No. (BH) – Sample No. (SS)	Depth (m)	Grain-Size Analysis (%) and Atterberg Limits (%)							Soil Classification (USCS)
		Gravel	Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index	
BH5 – SS4	2.3-2.9	0	13	58	29	34	18	16	Silty Clay of Low Plasticity (CL)

Based on a review of the results of the grain-size analysis the silty clay till may be classified a Silty Clay of Low Plasticity (CL) in accordance with the USCS.

5.6 Glacial Till

Glacial till was contacted below the fill, silty sand/sandy silt or the silty clay in Borehole Nos. 2 to 5 at depths of 0.7 m to 3.5 m (Elevation 64.7 m to Elevation 63.2 m). The composition of the glacial till contains varying amounts of gravel, sand, silt and clay. The glacial till contains cobbles and boulders. The SPT N-values of with the glacial till ranges from 2 to 44 indicating a very loose to dense condition. The natural moisture content of the glacial till ranges from 4 percent to 34 percent.

The results from the grain-size analysis conducted on one (1) sample of the glacial till are summarized in Table V. The grain-size distribution curve is shown in Figure 11.

Table V: Summary of Results from Grain-Size Analysis – Glacial Till Samples

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)				Soil Classification (USCS)
		Gravel	Sand	**Silt**	Clay	
BH4 SS7	4.6 - 5.2	10	44	35	11	Silty Sand (SM)

Based on a review of the results of the grain-size analysis the glacial till may be classified a silty sand (SM) in accordance with the USCS.

5.7 Limestone Bedrock

Auger refusal was met in all the boreholes at 1.7 m to 6.0 m depths (Elevation 63.1 m to Elevation 61.8 m). The presence of the bedrock was proven in Borehole Nos. 1,2 and 4 by washboring and core drilling techniques. Based on a review of the bedrock cores, the bedrock is considered to be limestone with shale partings.

Photographs of the bedrock cores are shown in Appendix B. A summary of the auger refusal and confirmed bedrock depths (elevations) are shown in Table VI.

Table VI: Summary of Bedrock Depths (Elevations)

Borehole (BH) No.	**Ground Surface Elevation (m)**	**Bedrock or Refusal Elevation (m)**	**Bedrock Confirmed by Core drilling**
BH-1	65.31	61.9	Yes. 5.0 m length of bedrock cored below 3.4 m (61.9 m)
BH-2	64.83	63.1	Yes. 6.8 m length of bedrock cored below 1.7 m (63.1 m)
BH-3	65.41	62.3	No. Auger refusal at 3.1 m (62.3 m)
BH-4	67.75	61.8	Yes. 3.2 m length of bedrock cored below 6.0 m (61.8 m)
BH-5	67.28	62.1	No. Auger refusal at 5.2 (62.1 m)

A review of Table VI indicates that the presence of limestone bedrock was confirmed at 1.7 m to 6.0 m (Elevation 63.1 m to Elevation 61.8 m). Based on the bedrock coring results, the total core recovery (TCR) ranges from 90 percent to 100 percent. The rock quality designation (RQD) ranges from 57 percent to 100 percent indicating the bedrock quality is fair to excellent quality.

Unit weight determination and unconfined compressive strength tests were conducted on three (3) rock core sections and the results are summarized in Table VII.

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

Borehole (BH) No. – Run No.	Depth (m)	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)	Classification of Rock with respect to Strength
BH1 Run2	5.2 - 5.4	26.0	141.4	Very Strong (R5)
BH2 Run1	1.8 - 1.9	25.5	123.1	Very Strong (R5)
BH4 Run1	6.9 - 7.0	25.9	126.5	Very Strong (R5)

A review of the test results in Table VII 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.

5.8 Groundwater Level Measurements

A summary of the groundwater level measurements taken in the monitoring wells are shown in Table VIII.

Table VIII: Summary of Groundwater level Measurements

Borehole (BH) /Monitoring Well (MW) No.	Ground Surface Elevation (m)	Date of Measurement (Elapsed Time in Days from Date of Installation)	Screened Material	Groundwater Depth Below Ground Surface (Elevation), m
BH-01	65.31	June 22, 2023 (16 Days)	Limestone	3.5 (61.8)
BH-02	64.83	June 22, 2023 (15 Days)	Limestone	1.8 (63.0)
BH-04	67.75	June 22, 2023 (16 Days)	Limestone	3.1 (64.7)

The groundwater within the limestone was found to be 1.8 m to 3.5 m (Elevation 64.7 m to Elevation 61.8 m).

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

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

6.1 Site Classification for Seismic Site Response

The results of the seismic shear wave survey conducted at the site are provided in the report attached in Appendix A. The survey indicates that the seismic shear wave velocity for footings placed on bedrock would be greater than 1,500 m/s. Table 4.1.8.4.A of the 2012 Ontario Building Code (as amended May 2, 20219) indicates that a seismic shear wave velocity value greater than 1,500 m/s falls within the range of velocities for site class A (V_{s30} <1,500 m/s). Therefore, the site classification for seismic response is **Class A.**

6.2 Liquefaction Potential of Soils

All overburden soils will be removed as part of the proposed development and therefore there is no liquefaction potential of the soils at the site during a seismic event.

7. Grade Raise Restrictions

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

8. Foundation Considerations

It is our understanding that two (2) levels of underground parking will be provided for the proposed high-rise structure and therefore, and it has been assumed that footings of the proposed building will be set at approximately 6.0 m (or deeper) below the existing ground surface. Based on a review of the borehole logs the footings will be placed on limestone bedrock.

Spread and strip footings founded on the sound limestone bedrock, competent and free of soil filled seams may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 3000 kPa. The factored ULS value includes a resistance factor of 0.5. The Serviceability Limit State (SLS) bearing pressure of the bedrock, required to produce 25 mm settlement of the structure will be much larger than the recommended value for factored geotechnical resistance at ULS. Therefore, the factored geotechnical resistance at ULS will govern the design. Settlements of footing designed for the above recommended factored geotechnical resistance at ULS and properly constructed are expected to be less than 10 mm.

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

All the footing beds should be thoroughly examined by a geotechnical engineer to ensure that the bedrock area is capable of supporting the design ULS value. Where fractured rock is encountered, sub-excavation may be undertaken to the underlying more competent bedrock. Alternatively, the footings may be redesigned to a reduced factored geotechnical resistance at ULS.

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

9. Floor Slab and Drainage Requirements

The lowest floor level of the parking garage for the proposed building will be located at approximately 6.0 m depth below the existing grade. Based on the borehole information, the lowest floor slabs of the building will be founded on limestone bedrock and may be constructed as a concrete slab-on-grade or as a paved surface. The concrete and asphalt pavement structures indicated below are for light duty traffic only (cars). EXP can provide concrete and asphalt pavement structures for heavy duty traffic (cars and trucks), if required.

The lowest floor level for the parking garages is anticipated to be located below the groundwater level. Therefore, underfloor and perimeter drainage systems will be required for the proposed below grade parking garages. All subsurface structures should be waterproofed.

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

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

9.1 Lowest Floor Level as a Concrete Surface

The subgrade is anticipated to be limestone bedrock. The subgrade should be examined by a geotechnical engineer and any loose/soft zones of the bedrock should be excavated, removed and replaced with lean mix concrete. Upon approval, the bedrock subgrade should be prepared as noted above.

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

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

The concrete slab should be reinforced, and adequate saw cuts should be provided in the floor slab to control cracking. Additional recommendations can be provided once the final design of the lower floor level has been determined.

9.2 Lowest Floor Level as a Paved Surface

The subgrade is anticipated to consist of limestone bedrock. The exposed limestone bedrock should be examined by a geotechnical engineer and any loose/soft zones of the bedrock noted in the footing beds should be excavated and replaced with lean mix concrete.

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

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

- 450 mm thick layer of OPSS Granular B Type II compacted to 100 percent SPMDD.

10. Lateral Earth Pressures Against Basement Walls

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

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

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

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

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

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

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

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

q = surcharge load stress, kPa

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

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

Where

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

B = backfill width (m)

z = depth from top of wall (m)

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

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

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

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

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

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

H = height of wall, m

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

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

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

Where the upper portion of the subsurface basement wall is backfilled with granular material, the vertical drainage board should extend into the backfill to provide drainage of the backfill. The top of the drainage board should be covered with a fabric filter to prevent the loss of overlying soil into the drainage board.

The vertical drainage board should be connected to a solid discharge pipe that passes through the foundation wall and outlets to a solid pipe inside the building that leads to a sump. The solid pipe inside the building should be connected to a separate sump from the sumps used for the perimeter and underfloor drains, so that this system would be operational should one of the other drainage systems fail.

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.

The Phase II ESA can be consulted for further details regarding the environmental aspects of the project.

Overburden Soil Excavation

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

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

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

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

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

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

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

Soldier Pile and Timber Lagging System

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

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

Where:

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

k = applicable earth pressure coefficient;
active lateral earth pressure coefficient = 0.33
‘at rest’ lateral earth pressure coefficient = 0.50

γ = unit weight of soil to be retained, estimated at 22 kN/m^3

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

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

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

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

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

Secant Pile Shoring 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

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

11.1.1 Rock Excavation

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

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

Rock Support

The weathered and fractured rock face may require support in the form of rock bolts to maintain the integrity of the rock face in conjunction with a wire mesh system and the shotcrete mentioned above. Excavations that will extend a significant depth into the bedrock will have to be undertaken in a staged approach with the rock excavated in a pre-determined depth interval (for example every 3 m). The exposed rock face in each stage will have to be examined by a geotechnical engineer to determine the number of rock bolts required. The rock bolt system should be installed in this manner to the bottom of the excavation.

Vibration Control

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

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

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

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

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

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

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

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

12. Pipe Bedding Requirements

The invert depths of the underground services are not known at the time of this geotechnical investigation. It is anticipated that the subgrade for the proposed municipal services will be limestone bedrock.

The bedding for the underground services including material specifications, thickness of cover material and compaction requirements conform to the Ontario Provincial Standard Specification and Drawings (OPSS and OPSD) based on the selection of the pipe material and size.

It is recommended that the pipe bedding be 150 mm thick and consist of OPSS Granular A. The bedding material should be placed along the sides and on top of the pipe to provide a minimum cover of 300 mm. The bedding should be compacted to at least 98 percent of the standard Proctor maximum dry density (SPMDD).

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

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

The soils to be excavated from the site will comprise of fill, silty sand/sandy silt, silty clay and glacial till. From a geotechnical perspective, these soils are not considered suitable for reuse as backfill material in the interior or exterior of the building. Therefore, it is anticipated that the majority of the material required for backfilling purposes in the interior and exterior of the proposed building will need to be imported and should preferably conform to OPSS 1010 (as amended by SSP110S13) for Granular B Type II. The backfill should be placed in 300 mm thick lifts compacted to 95 percent standard Proctor maximum dry density (SPMDD) outside the building and to 98 percent SPMDD inside the building.

14. Corrosion Potential

Chemical tests limited to pH, sulphate, chloride and resistivity were undertaken on two (2) samples, one (1) sample of the glacial till overburden soil and one (1) sample of the limestone bedrock. A summary of the results is shown in Table IX. The laboratory certificate of analysis is shown in Appendix C.

Table IX: Chemical Test Results – Soil and Bedrock Samples

Borehole No. - Sample No. (SS)	Depth (m)	Soil/Bedrock Type	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
BH1 Run 1	3.4 - 3.5	Limestone Bedrock	8.88	0.003	0.007	4900
BH4 SS8	5.3 - 5.9	Glacial Till	8.98	0.010	0.007	3510

The results indicate the soil has a negligible sulphate attack on subsurface concrete. The concrete should be in accordance with the most recent CSA A.23.1.

The results of the resistivity test indicate the glacial till and limestone bedrock is 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.

15. Tree Planting Restrictions

It is anticipated that all subsurface soils on site including the overburden soils will be excavated down to the bedrock and removed from site for the construction of the proposed new building. Since all subsurface soils will be excavated and removed from the site and, where required, replaced with compacted granular fill, there are no tree planting restrictions from a geotechnical perspective.

16. Earthworks Quality Control During Construction

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

17. General Comments

The comments and recommendations given in this report are preliminary in nature as they are based on the assumption that the above-described design concepts will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations, or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint. This geotechnical report should be updated once final design for the proposed development is available.

The information contained in this report is not intended to reflect on environmental aspects of the soils and groundwater. The EXP Phase II ESA should be consulted for the environmental aspects of the project.

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

Daniel Wall

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

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

LICENSED PROFESSIONAL ENGINEER
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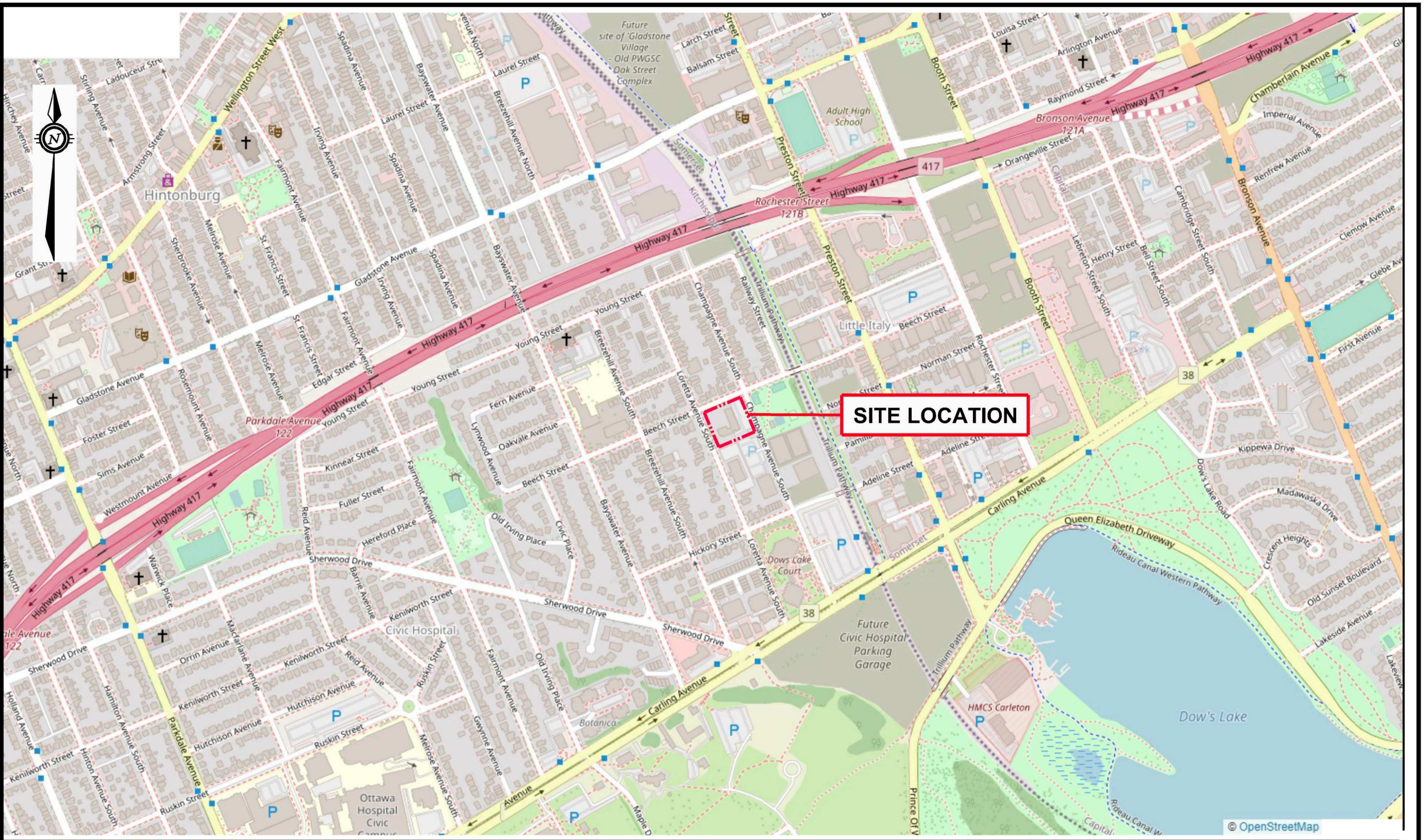
PROVINCE OF ONTARIO

Figures



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ORIGINAL SHEET SIZE = 11" x 8.5"

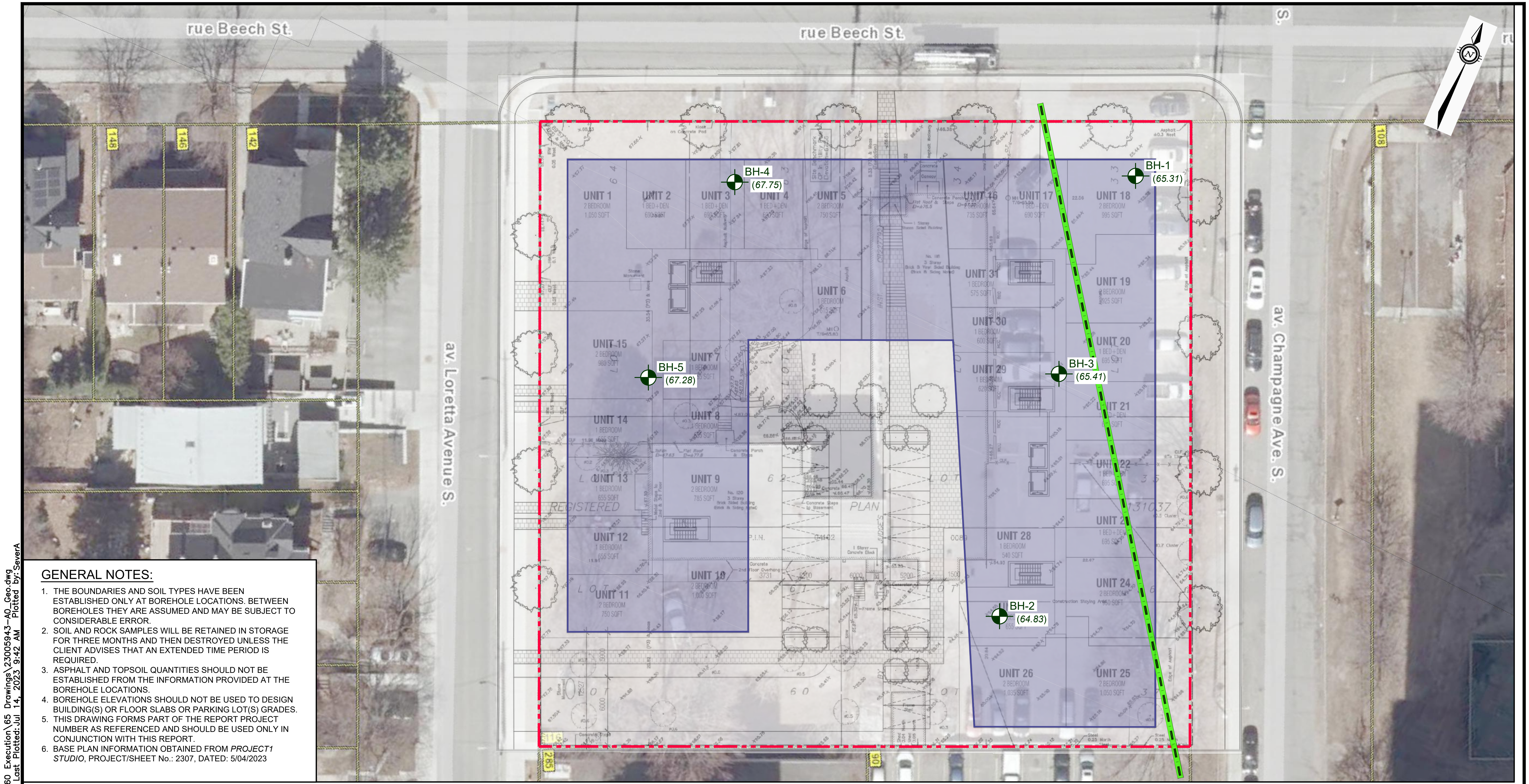
0 100m 200m 400m

HORIZONTAL 1:10,000

exp.

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

DATE MAY 2023		PROPOSED HIGH RISE DEVELOPMENT 116 BEECH STREET, OTTAWA, ONTARIO	project no. OTT-23005943-A0
DESIGN DW	CHECKED IT		scale 1:10,000
DRAWN BY AS		SITE LOCATION PLAN	FIG 1



GENERAL NOTES:

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL AND ROCK SAMPLES WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. ASPHALT AND TOPSOIL QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. BASE PLAN INFORMATION OBTAINED FROM *PROJECT1 STUDIO*, PROJECT/SHEET No.: 2307, DATED: 5/04/2023

LEGEND

— · —	PROPERTY BOUNDARIES
▨	PROPOSED HIGH RISE BUILDINGS FOOTPRINT
⊕ BH-1 (65.31)	BOREHOLE NO. AND LOCATION (*EXP*, 2023) GEODETIC GROUND SURFACE ELEVATION (m) BASED ON THE BENCHMARK PROVIDED BY *FARLEY, SMITH & DENIS SURVEYING LTD.* SURVEY (DATED: 16/09/2022)
🟩	MASW STUDY AREA

ORIGINAL SHEET SIZE = 17" x 11"

0 4m 8m 16m

HORIZONTAL 1:400

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2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

DATE JULY 2023		PROPOSED HIGH RISE DEVELOPMENT 116 BEECH STREET, OTTAWA, ONTARIO	project no. OTT-23005943-A0
DESIGN DW	CHECKED IT		scale 1:400
DRAWN BY AS		BOREHOLE LOCATION PLAN	FIG 2

Filename: E:\OTT\OTT-23005943-A0\60 Execution\65 Drawings\23005943-A0_Geo.dwg
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Log of Borehole BH-01

Project No: OTT-23005943-A0

Project: Proposed High-Rise Redevelopment

Location: 116 Beech Street, Ottawa, Ontario

Date Drilled: 'June 6, 2023

Drill Type: CME-75 Track-Mounted Drill Rig

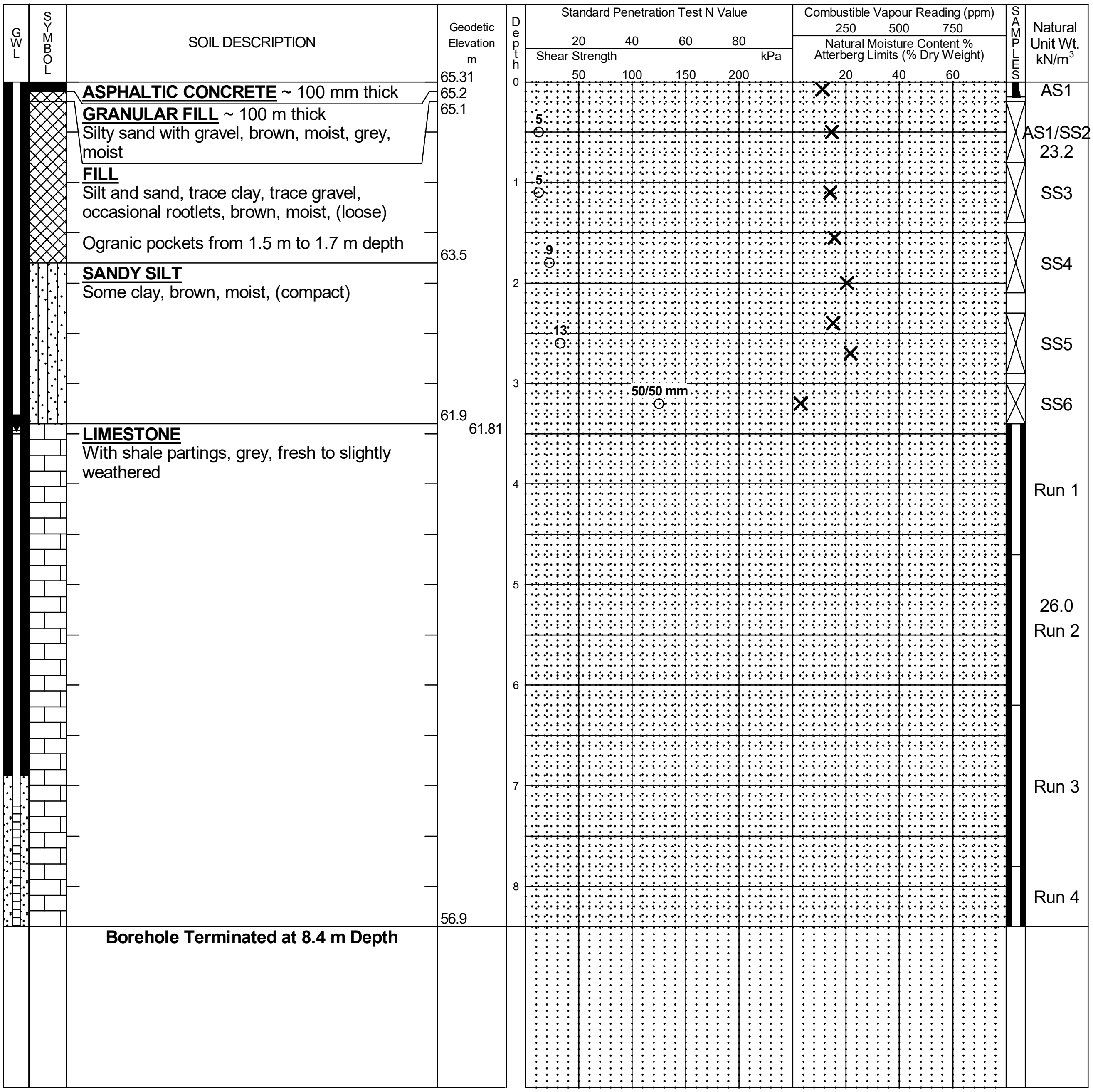
Datum: Geodetic Elevation

Logged by: MD Checked by: DW

Figure No. 3

Page. 1 of 1

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		



ASPHALTIC CONCRETE ~ 100 mm thick

GRANULAR FILL ~ 100 m thick
Silty sand with gravel, brown, moist, grey, moist

FILL
Silt and sand, trace clay, trace gravel, occasional rootlets, brown, moist, (loose)

Ogranic pockets from 1.5 m to 1.7 m depth

SANDY SILT
Some clay, brown, moist, (compact)

LIMESTONE
With shale partings, grey, fresh to slightly weathered

Borehole Terminated at 8.4 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 32 mm well installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-23005943-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
6/22/2023	3.5	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.4 - 4.7	100	91
2	4.7 - 6.2	100	90
3	6.2 - 7.8	100	87
4	7.8 - 8.4	100	100

Log of Borehole BH-02

Project No: OTT-23005943-A0

Project: Proposed High-Rise Redevelopment

Location: 116 Beech Street, Ottawa, Ontario

Figure No. 4

Page. 1 of 1

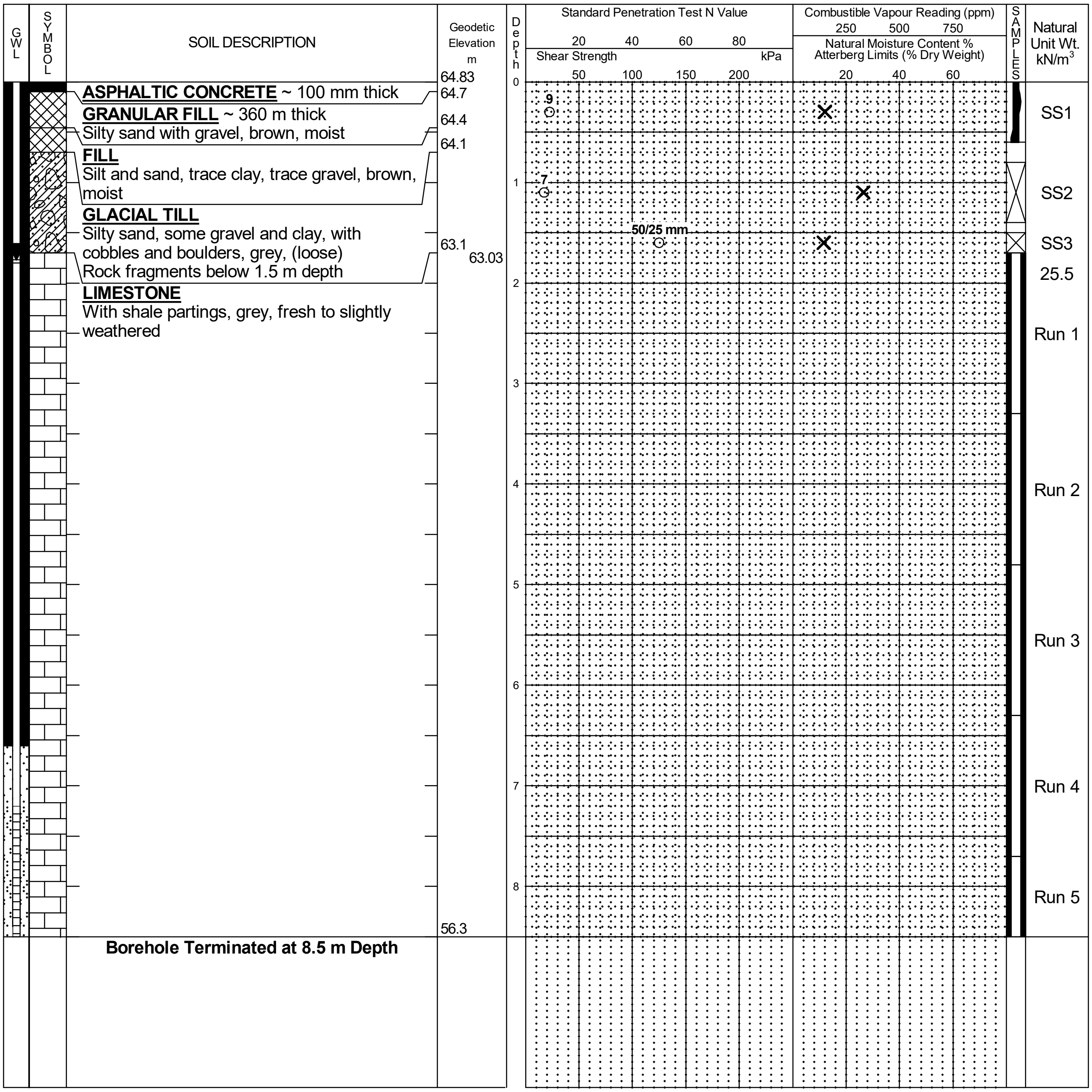
Date Drilled: 'June 7, 2023

Drill Type: CME-75 Track-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: JE 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	▲



ASPHALTIC CONCRETE ~ 100 mm thick

GRANULAR FILL ~ 360 m thick
Silty sand with gravel, brown, moist

FILL
Silt and sand, trace clay, trace gravel, brown, moist

GLACIAL TILL
Silty sand, some gravel and clay, with cobbles and boulders, grey, (loose)
Rock fragments below 1.5 m depth

LIMESTONE
With shale partings, grey, fresh to slightly weathered

Borehole Terminated at 8.5 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 32 mm well installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-23005943-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
6/22/2023	1.8	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	1.7 - 3.3	100	71
2	3.3 - 4.8	99	59
3	4.8 - 6.3	100	95
4	6.3 - 7.7	96	96
5	7.7 - 8.5	100	89

Log of Borehole BH-03

Project No: OTT-23005943-A0

Project: Proposed High-Rise Redevelopment

Location: 116 Beech Street, Ottawa, Ontario

Date Drilled: 'June 7, 2023

Drill Type: CME-75 Track-Mounted Drill Rig

Datum: Geodetic Elevation

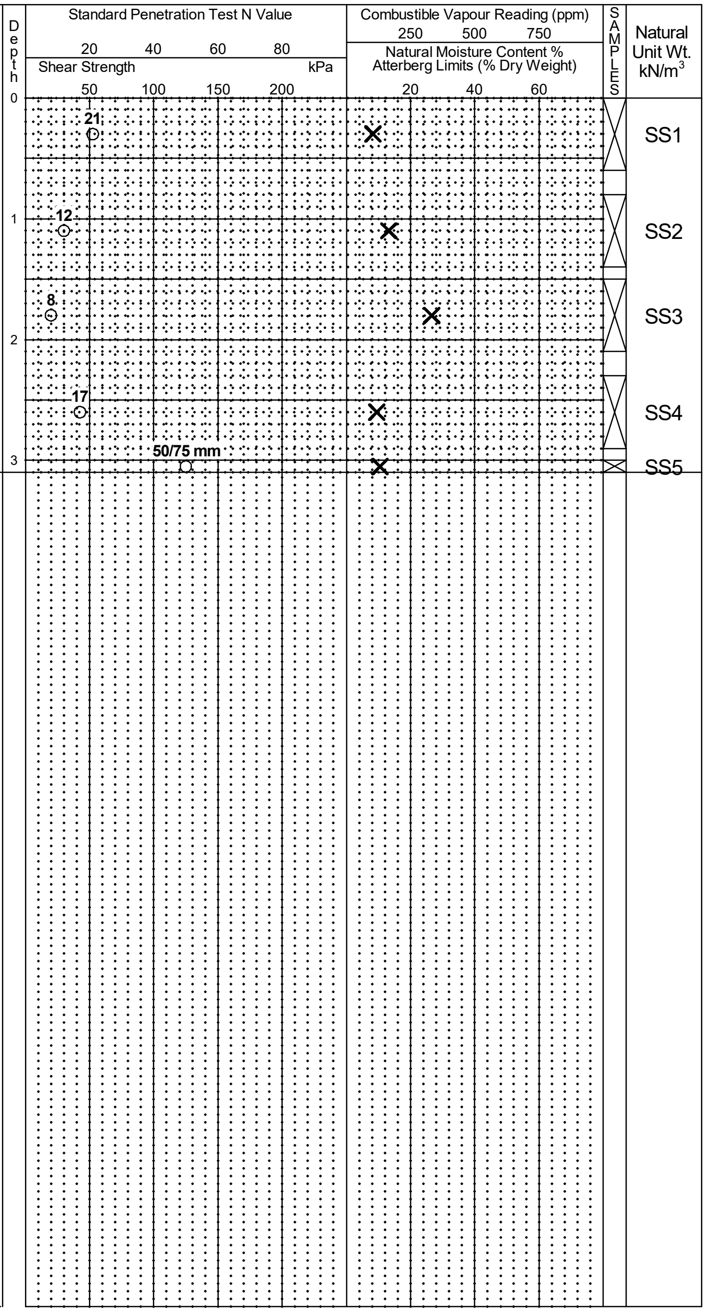
Logged by: JE Checked by: DW

Figure No. 5

Page. 1 of 1

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

GWL	SOIL DESCRIPTION	Geodetic Elevation m	Depth (m)	SAMPLES
		65.41	0	
	ASPHALTIC CONCRETE ~ 50 mm thick	65.4		
	GRANULAR FILL ~ 250 m thick Silty sand with gravel, brown, moist	65.1		SS1
	FILL Silt and sand, trace clay, trace gravel, brown, moist, (compact)	64.0	1	SS2
	SANDY SILT Some clay, brown, moist, (loose)	63.2	2	SS3
	GLACIAL TILL Silty sand, some gravel and clay, with cobbles and boulders, grey, (compact)	62.3	3	SS4 SS5
	Auger Refusal at 3.1 m Depth			



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole was backfilled upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-23005943-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

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

Log of Borehole BH-04

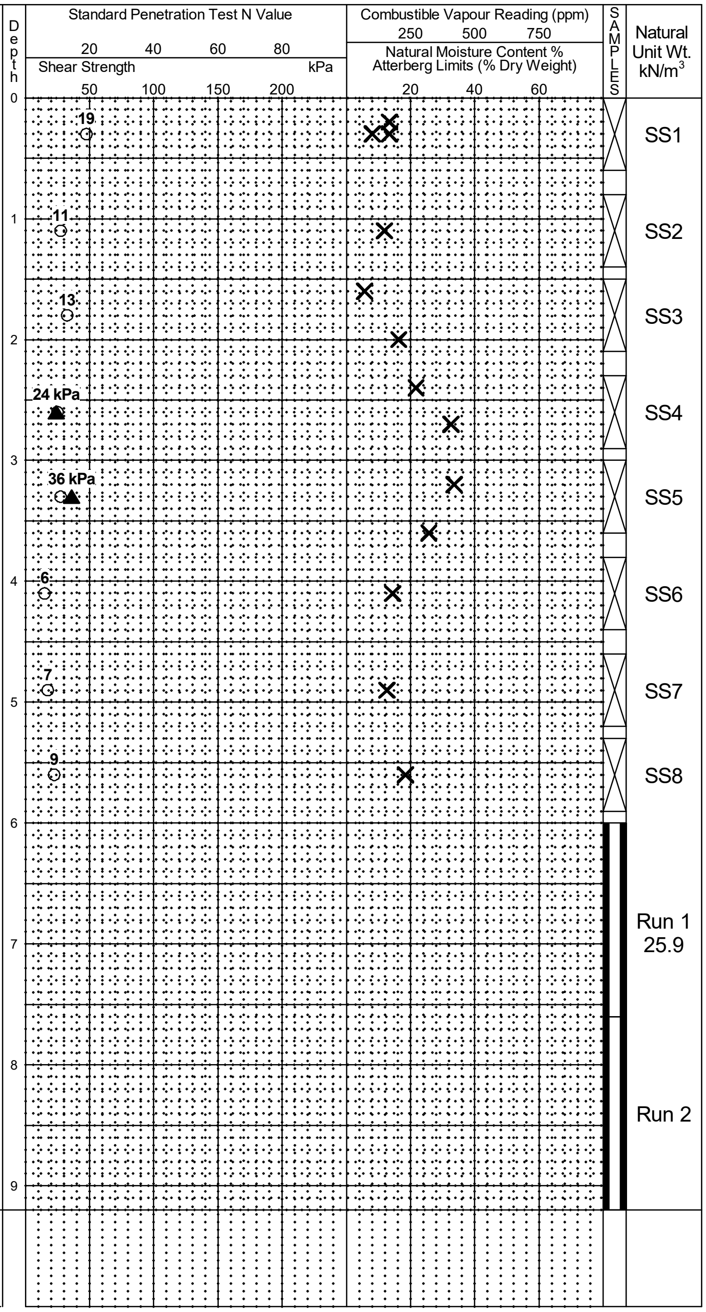
Project No: OTT-23005943-A0
Project: Proposed High-Rise Redevelopment
Location: 116 Beech Street, Ottawa, Ontario
Date Drilled: 'June 6, 2023
Drill Type: CME-75 Track-Mounted Drill Rig
Datum: Geodetic Elevation
Logged by: MD Checked by: DW

Figure No. 6
Page. 1 of 1

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
	67.75
TOPSOIL ~ 250 mm thick	67.5
FILL Silt and sand, trace clay, trace gravel, brown, moist, (compact)	66.0
SILTY SAND Brown, moist, (compact)	65.2
SILTY CLAY Brown, moist, (soft to firm) Wet below 3.0 m depth	64.65 64.3
GLACIAL TILL Silty sand, some gravel and clay, with cobbles and boulders, grey, (loose)	61.8
LIMESTONE With shale partings, grey, fresh to slightly weathered	58.6

Borehole Terminated at 9.2 m Depth



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. 32 mm well installed upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-23005943-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
6/22/2023	3.1	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6 - 7.6	90	57
2	7.6 - 9.2	98	98

Log of Borehole BH-05

Project No: OTT-23005943-A0

Project: Proposed High-Rise Redevelopment

Location: 116 Beech Street, Ottawa, Ontario

Figure No. 7

Page. 1 of 1

Date Drilled: 'June 6, 2023

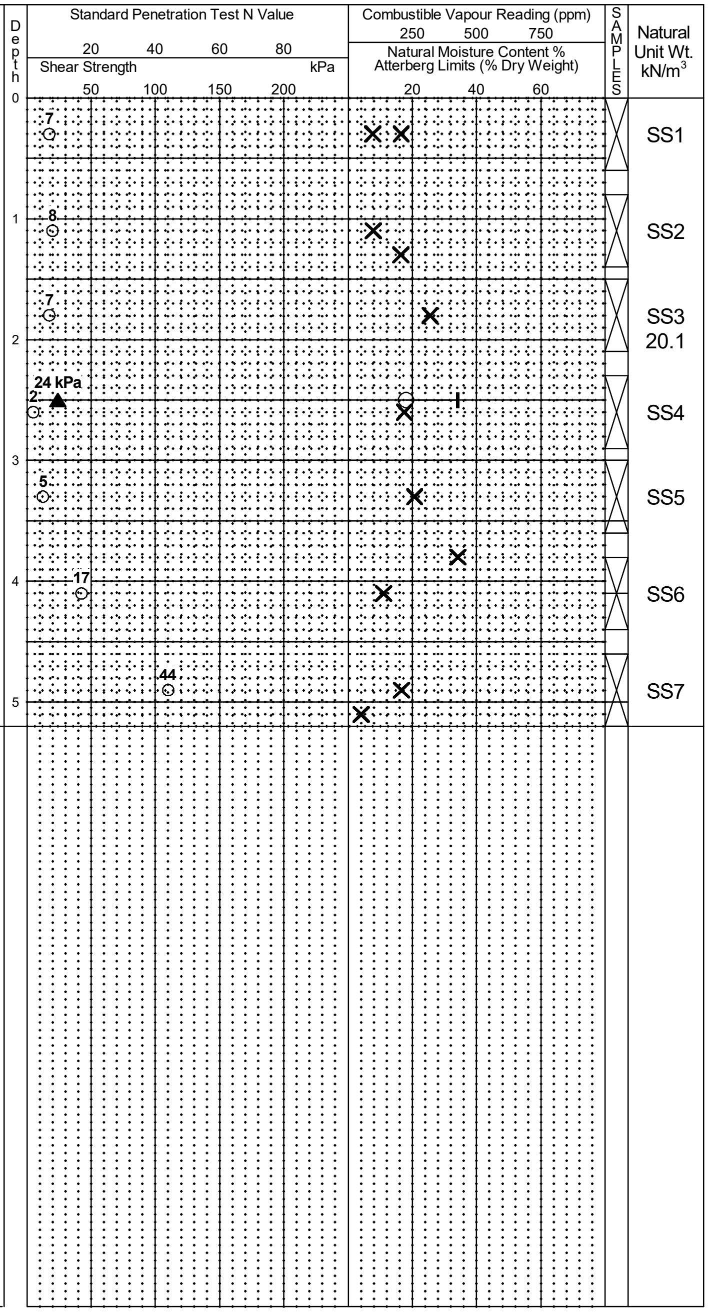
Drill Type: CME-75 Track-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: MD 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
	67.28
TOPSOIL ~ 225 mm thick	67.1
FILL Silt and sand, trace clay, trace gravel, moist, (loose) Some gravel and occasional rootlets from 0.3 m to 0.6 m depth	65.9
SILTY CLAY Low plasticity, with silty sand seams, brown, moist, (soft)	64.6
GLACIAL TILL Silty sand, some gravel and clay, with cobbles and boulders, grey, (very loose to dense) Rock fragments below 5.0 m depth	62.1
Auger Refusal at 5.2 m Depth	



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole was backfilled upon completion
3. Field work supervised by an exp representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-23005943-A0

WATER LEVEL RECORDS

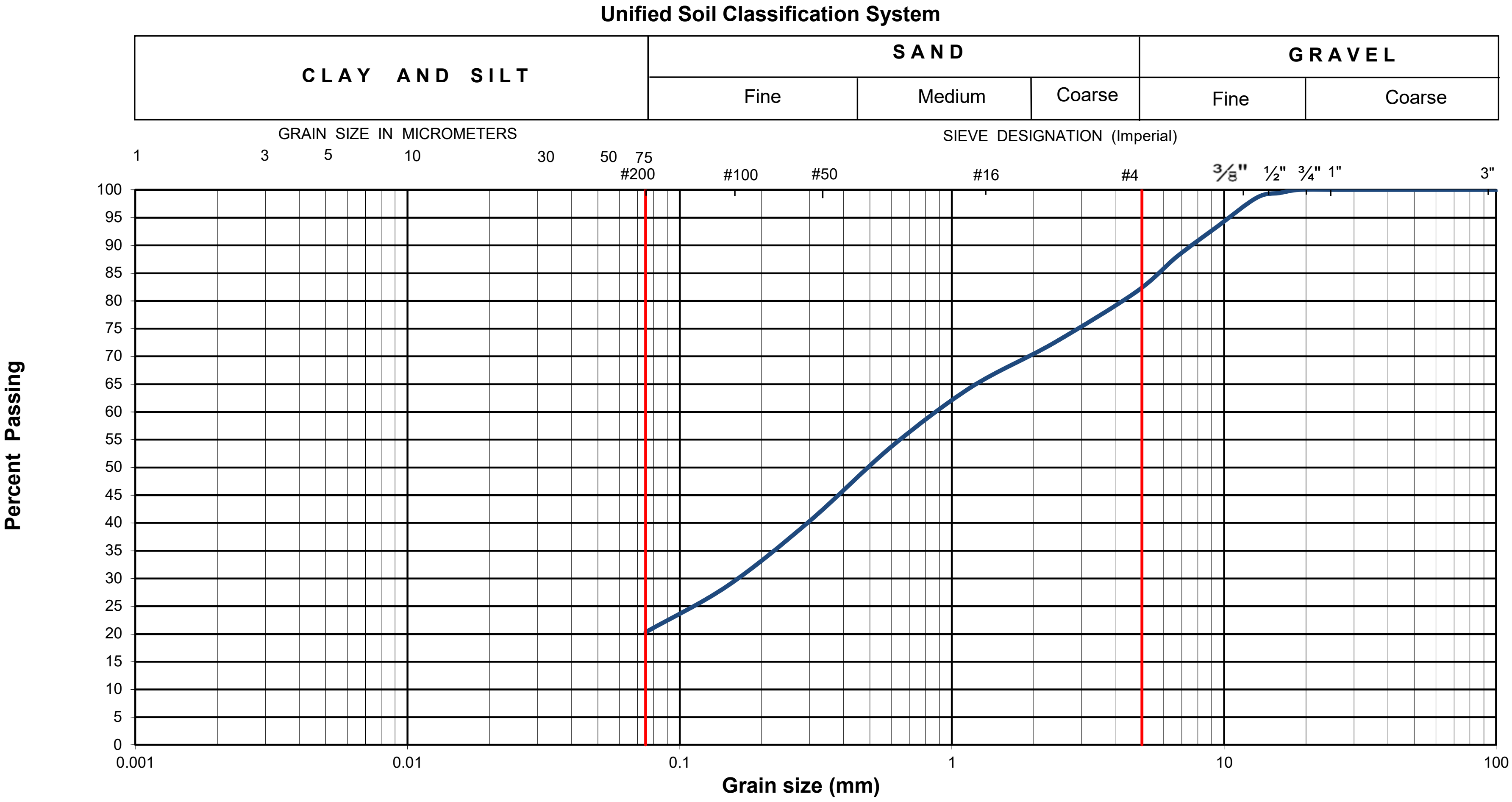
Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

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

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

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



EXP Project No.:	OTT-23005943-A0	Project Name :	Proposed High-Rise Redevelopment				
Client :	Katasa Groupe	Project Location :	116 Beech Street, Ottawa, ON				
Date Sampled :	June 7, 2023	Borehole No:	BH2	Sample:	SS1	Depth (m) :	0-0.8
Sample Composition :		Gravel (%) 18	Sand (%) 62	Silt & Clay (%) 20		Figure :	8
Sample Description :	Granular Fill: Silty Sand with Gravel (SM)						

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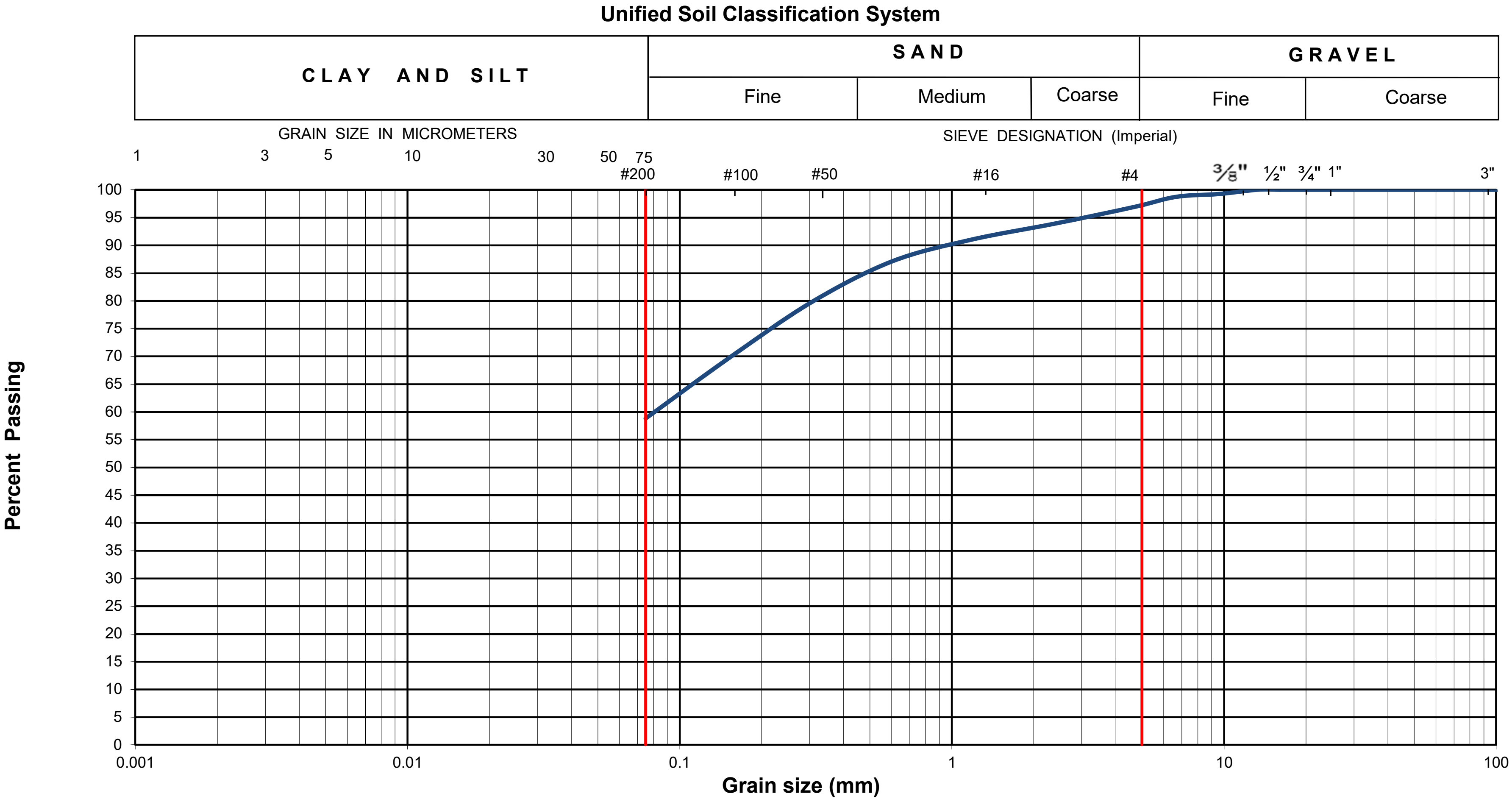
Ottawa, ON   K2B 8H6

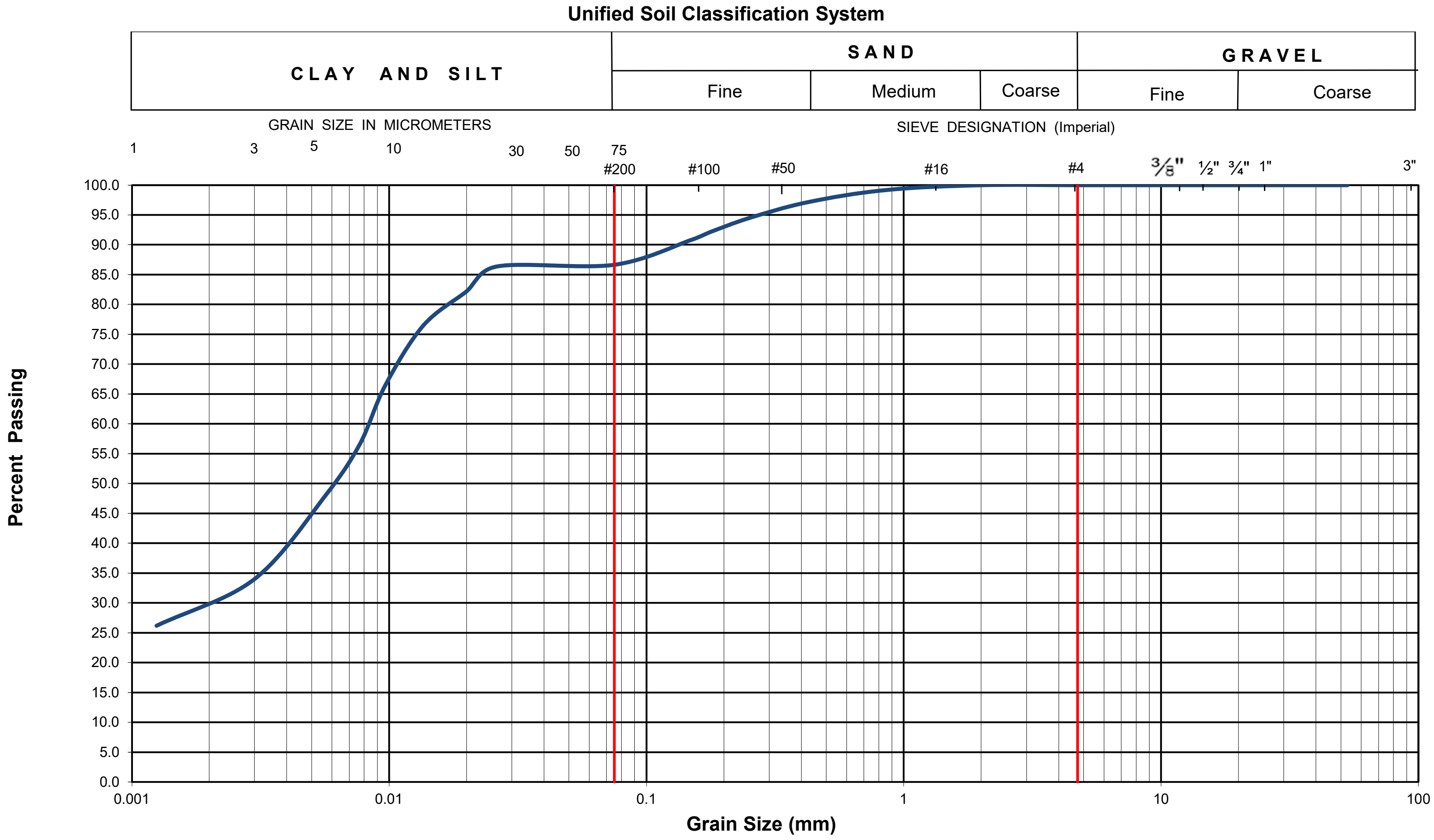
Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136



EXP Project No.:	**OTT-23005943-A0**	**Project Name :**	**Proposed High-Rise Redevelopment**					
Client :	**Katasa Groupe**	**Project Location :**	**116 Beech Street, Ottawa, ON**					
Date Sampled :	**June 7, 2023**	**Borehole No:**		**BH2**	**Sample:**	**SS2**	**Depth (m) :**	**0.8-1.4**
Sample Composition :		**Gravel (%)**	**3**	**Sand (%)**	**38**	**Silt & Clay (%)**	**59**	**Figure :** **9**
Sample Description :	**FILL: Silt with Sand (ML)**							

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

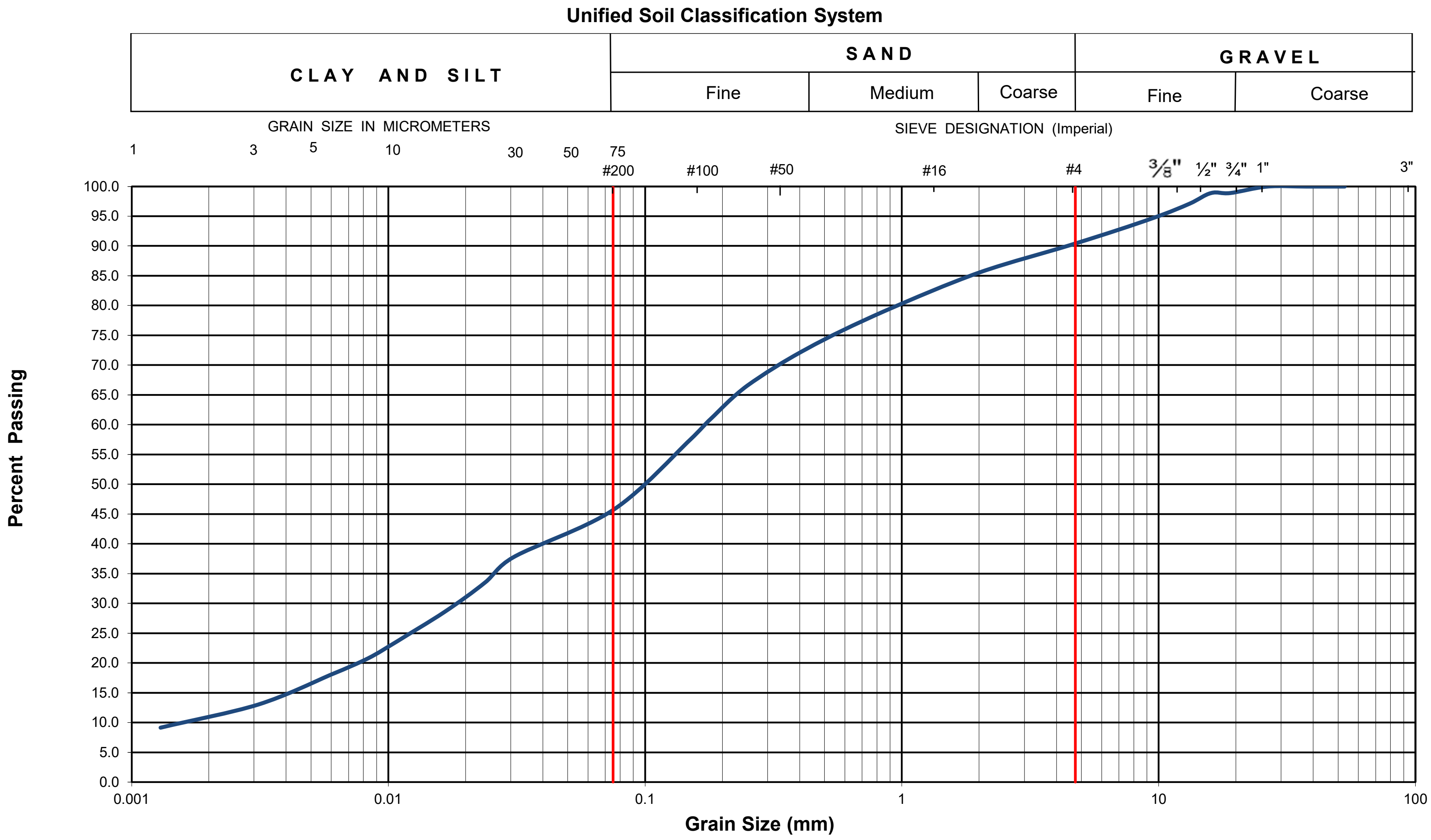
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-23005943-A0	**Project Name :**	Proposed High-Rise Redevelopment				
Client :	Katasa Groupe	**Project Location :**	116 Beech Street, Ottawa, ON				
Date Sampled :	June 5, 2023	**Borehole No:**	BH5	**Sample No.:**	SS4	**Depth (m) :**	2.3-2.9
Sample Description :		**% Silt and Clay**	87	**% Sand**	13	**% Gravel**	0
Sample Description :		Silty Clay of Low Plasticity (CL)				**Figure :**	10

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

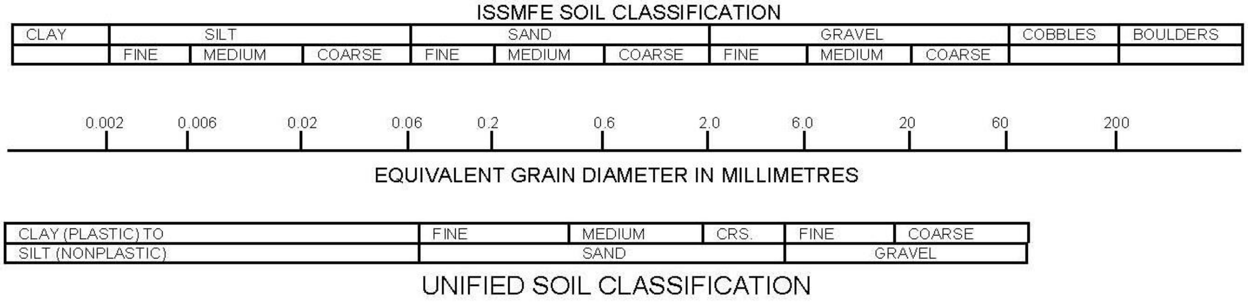
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-23005943-A0	Project Name :	Proposed High-Rise Redevelopment						
Client :	Katasa Groupe	Project Location :	116 Beech Street, Ottawa, ON						
Date Sampled :	June 5, 2023	Borehole No:	BH4	Sample No.:	SS7			Depth (m) :	4.6-5.2
Sample Description :		% Silt and Clay	46	% Sand	44	% Gravel	10	Figure :	11
Sample Description :	Glacial Till: Silty Sand (SM)								

Notes On Sample Descriptions

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



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

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

Appendix A – Multi-channel Analysis of Surface Waves 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 29th, 2023

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

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**
116 Beech Street, Ottawa (ON)

[Project: OTT-23005943-A0]

Dear Sir,

Geophysics GPR International inc. has been mandated by **exp** Services inc. to carry out seismic surveys at 116 Beech Street, in Ottawa (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Spatial AutoCorrelation (SPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocity values were calculated for the soil and the rock, to determine the Site Class.

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

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

MASW PRINCIPLE

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

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

INTERPRETATION

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

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

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

SURVEY DESIGN

The seismic spreads were installed on a parking lot (Figure 2). The geophone spacing was 3.0 metres for the main spread, using 24 geophones. Two shorter seismic spreads, with geophone spacings of 0.5 and 1.0 metre, were dedicated to the near surface materials. The seismic records were produced with a seismograph Terraloc Pro 2 (from ABEM Instrument), and the geophones were 4.5 Hz.

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

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

RESULTS

The MASW calculated V_S results are illustrated at Figure 5. Some low to very low seismic velocities were calculated from the surface to 2 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:

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

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

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

The calculated $\overline{V}_{S30}$ value of the actual site is 933.1 m/s (Table 1), corresponding to the Site Class "B". However, the Site Classes A and B are not to be used if there is 3 metres or more of soils between the rock and the bottom of the spread footing, pile cap or mat foundation. In the case the bottom of the foundation would be 2.2 metres or less from the rock, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" (Table 2).

CONCLUSION

Geophysical surveys were carried out to identify the Site Class at 116 Beech Street, in Ottawa (ON). The seismic surveys used the MASW and the SPAC analysis, and the seismic refraction to calculate the $\overline{V}_{S30}$ value. Its calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 933 m/s, corresponding to the Site Class "B" ($760 < \overline{V}_{S30} \leq 1500$ m/s), as determined through the MASW and SPAC methods, Table 4.1.8.4.-A of the NBC (2015), and the Building Code, O. Reg. 332/12. It must be noted that the Site Classes A and B are not to be used if there is 3 metres or more of soils between the rock and the bottom of the spread footing, pile cap or mat foundation.

In the case the bottom of the foundation would be 2.2 metres or less from the rock surface, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" ($\overline{V}_{S30} > 1500$ m/s).

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

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

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

P.Eng.

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

LICENSED PROFESSIONAL ENGINEER
J.-L. ARSENAULT
100148122
PROVINCE OF ONTARIO
2023-06-29

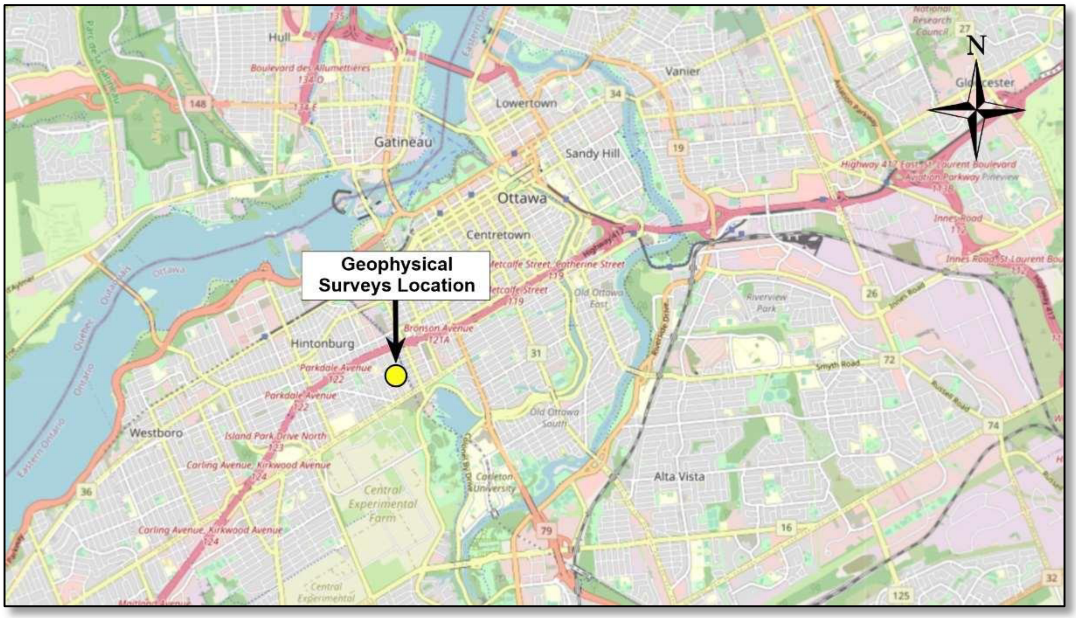


Figure 1: Regional location of the Site

(source: *OpenStreetMap©*)

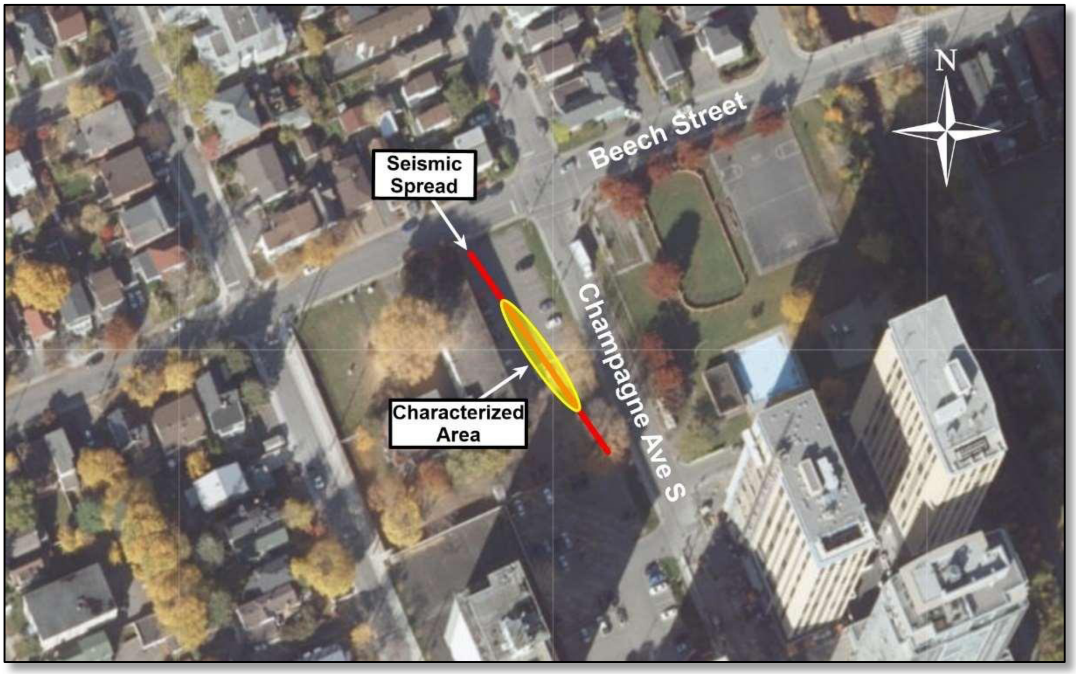


Figure 2: Location of the seismic spreads

(source: *geoOttawa*)

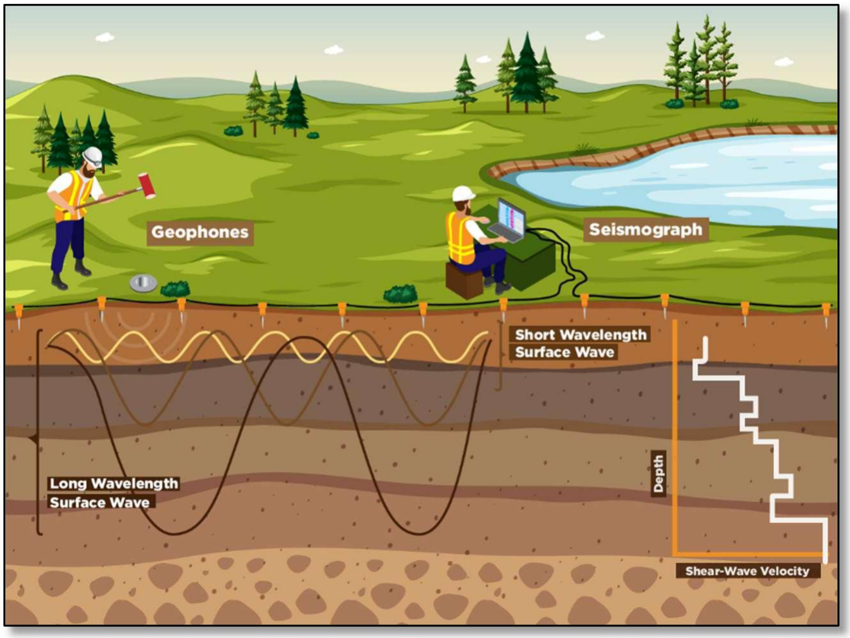
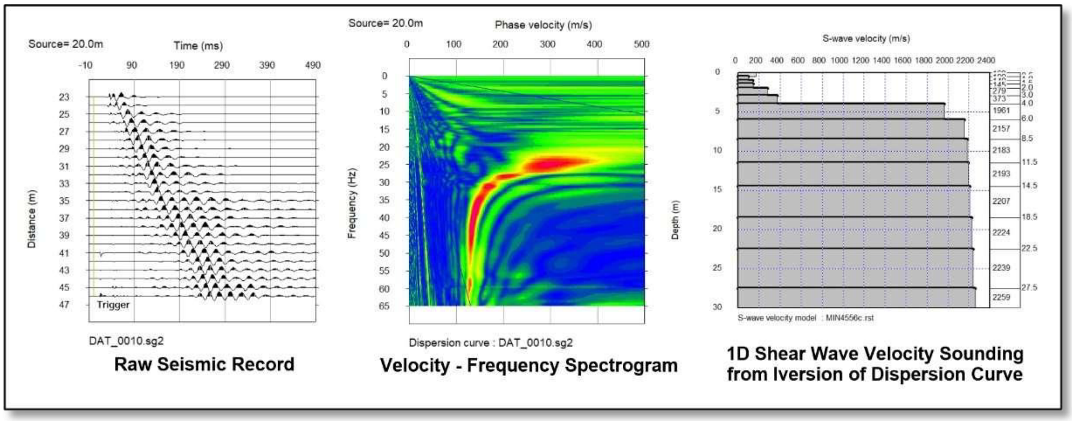


Figure 3: MASW Operating Principle



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

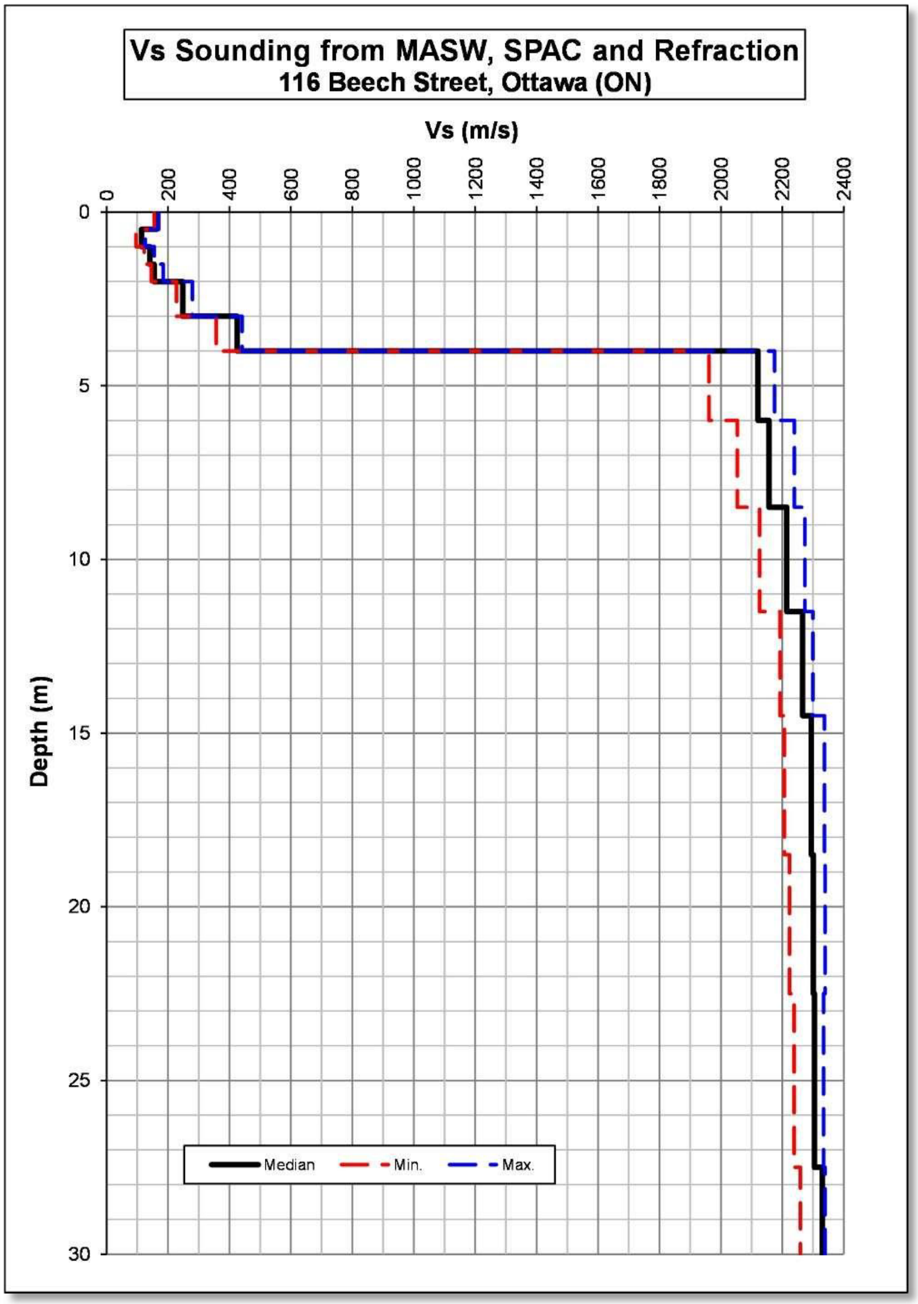


Figure 5: MASW Shear-Wave Velocity Sounding

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	**155.4**	**162.2**	**169.9**	**Grade Level (June 14, 2023)**				
0.5	**95.7**	**114.0**	**125.1**	0.50	0.50	0.003082	0.003082	162.2
1.0	**122.5**	**141.0**	**155.1**	0.50	1.00	0.004388	0.007470	133.9
1.5	**145.7**	**154.1**	183.9	0.50	1.50	0.003547	0.011016	136.2
2.0	228.0	248.0	279.4	0.50	2.00	0.003244	0.014260	140.2
3.0	356.8	424.9	441.4	1.00	3.00	0.004033	0.018293	164.0
4.0	1961.3	2120.8	2175.0	1.00	4.00	0.002354	0.020647	193.7
6.0	2053.8	2156.7	2239.1	2.00	6.00	0.000943	0.021590	277.9
8.5	2125.9	2214.7	2273.4	2.50	8.50	0.001159	0.022749	373.6
11.5	2192.6	2266.1	2299.7	3.00	11.50	0.001355	0.024104	477.1
14.5	2207.0	2294.2	2337.4	3.00	14.50	0.001324	0.025427	570.3
18.5	2223.5	2300.9	2339.4	4.00	18.50	0.001744	0.027171	680.9
22.5	2238.7	2304.6	2334.4	4.00	22.50	0.001738	0.028909	778.3
27.5	2258.5	2330.7	2339.4	5.00	27.50	0.002170	0.031079	884.8
30				2.50	30.00	0.001073	0.032152	933.1

V_{S30} (m/s)	**933.1**
Class	**B** (1)

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

TABLE 2
Limit for the Site Class A

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	155.4	162.2	169.9	**Limit for the Site Class A (2.2 metres of soils)**				
0.5	95.7	114.0	125.1					
1.0	122.5	141.0	155.1					
1.5	145.7	154.1	183.9					
1.8	145.7	154.1	183.9					
2.0	228.0	248.0	279.4	0.20	0.20	0.001298	0.001298	154.1
3.0	356.8	424.9	441.4	1.00	1.20	0.004033	0.005330	225.1
4.0	1961.3	2120.8	2175.0	1.00	2.20	0.002354	0.007684	286.3
6.0	2053.8	2156.7	2239.1	2.00	4.20	0.000943	0.008627	486.8
8.5	2125.9	2214.7	2273.4	2.50	6.70	0.001159	0.009786	684.6
11.5	2192.6	2266.1	2299.7	3.00	9.70	0.001355	0.011141	870.7
14.5	2207.0	2294.2	2337.4	3.00	12.70	0.001324	0.012465	1018.9
18.5	2223.5	2300.9	2339.4	4.00	16.70	0.001744	0.014208	1175.4
22.5	2238.7	2304.6	2334.4	4.00	20.70	0.001738	0.015947	1298.1
27.5	2258.5	2330.7	2339.4	5.00	25.70	0.002170	0.018116	1418.6
31.8				4.30	30.00	0.001845	0.019961	1502.9

V_{S30} *	**1502.9**
Class	**A**

Appendix B – Bedrock Core Photographs

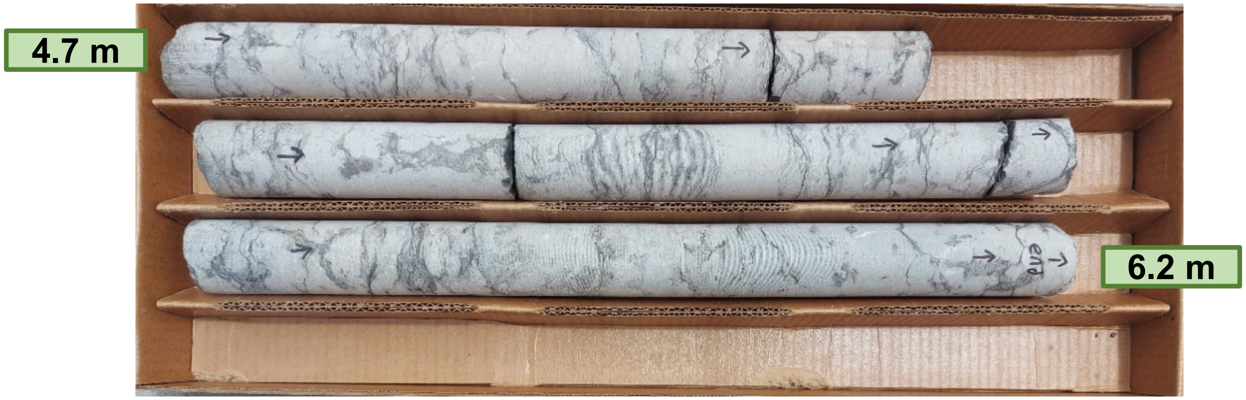
DRY BEDROCK CORES

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: **BH1**	Core Runs Run 1: 3.4 m - 4.7 m	project Geotechnical Investigation - Proposed High-Rise Development 116 Beech Street	Project N0: OTT-23005943-A0
Date Cored Jun 06, 2023		Rock Core Photographs	FIG B-1

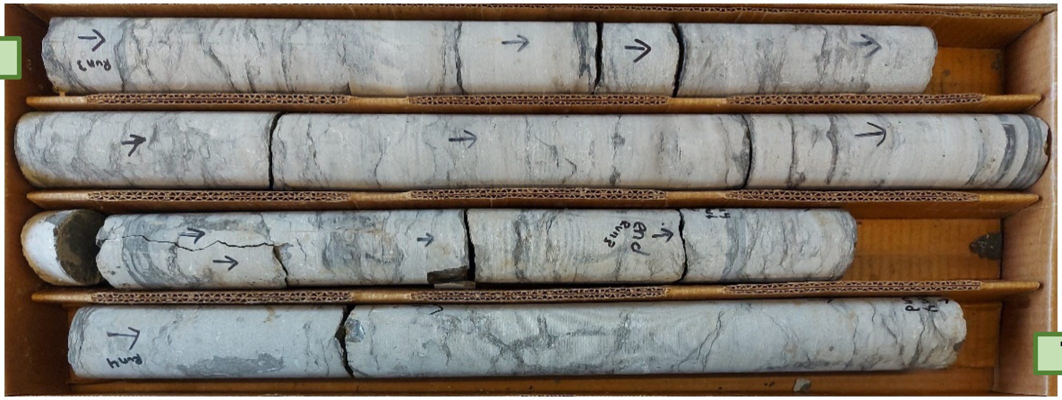
DRY BEDROCK CORES



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

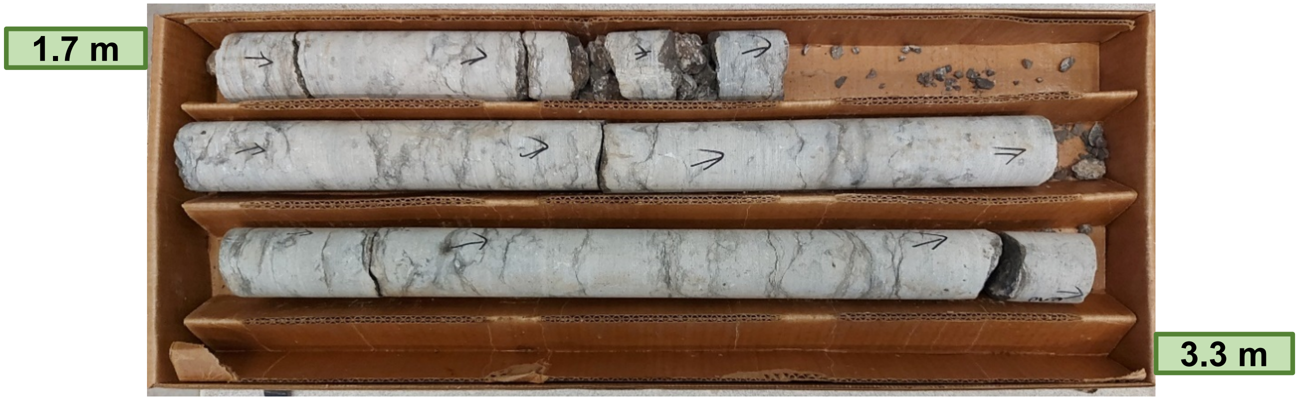
Borehole No: **BH1**	Core Runs Run 2: 4.7 m - 6.2 m	project	Geotechnical Investigation - Proposed High-Rise Development 116 Beech Street	Project N0: OTT-23005943-A0
Date Cored Jun 06, 2023			Rock Core Photographs	FIG B-2

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: **BH1**	Core Runs Run 3: 6.2 m - 7.8 m	project Geotechnical Investigation - Proposed High-Rise Development 116 Beech Street	Project N0: OTT-23005943-A0
Date Cored Jun 06, 2023		Rock Core Photographs	FIG B-3

DRY BEDROCK CORES



WET BEDROCK CORES

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2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

Borehole No: **BH2**	Core Runs Run 1: 1.7 m - 3.3 m	project Geotechnical Investigation - Proposed High-Rise Development 116 Beech Street	Project N0: OTT-23005943-A0
Date Cored Jun 07, 2023		Rock Core Photographs	FIG B-4

DRY BEDROCK CORES

3.3 m

4.8 m

WET BEDROCK CORES

3.3 m

4.8 m

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: **BH2**	Core Runs Run 2: 3.3 m - 4.8 m	project Geotechnical Investigation - Proposed High-Rise Development 116 Beech Street	Project N0: OTT-23005943-A0
Date Cored Jun 07, 2023		Rock Core Photographs	FIG B-5

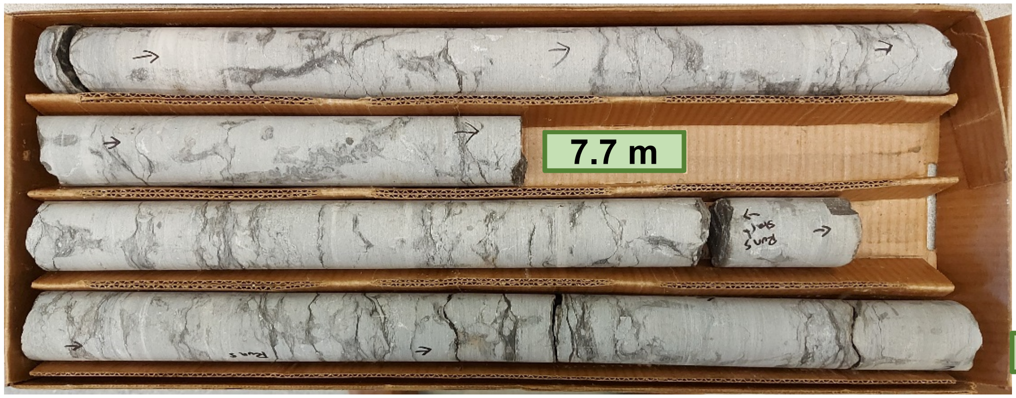
DRY BEDROCK CORES

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: **BH2**	Core Runs Run 3: 4.8 m - 6.3 m	project Geotechnical Investigation - Proposed High-Rise Development 116 Beech Street	Project N0: OTT-23005943-A0
Date Cored Jun 07, 2023		Rock Core Photographs	FIG B-6

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:
BH2

Core Runs
Run 4: 6.3 m - 7.7 m
Run 5: 7.7 m - 8.5 m

project
Geotechnical Investigation -
Proposed High-Rise Development
116 Beech Street

Project N0:
OTT-23005943-A0

Date Cored
Jun 07, 2023

Rock Core Photographs

FIG B-7

DRY BEDROCK CORES

WET BEDROCK CORES

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t: +1.613.688.1899 | f: +1.613.225.7337
2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

Borehole No: **BH4**	Core Runs Run 1: 6.0 m - 7.6 m	project Geotechnical Investigation - Proposed High-Rise Development 116 Beech Street	Project N0: OTT-23005943-A0
Date Cored Jun 06, 2023		Rock Core Photographs	FIG B-8

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: **BH4**	Core Runs Run 2: 7.6 m - 9.2 m	project Geotechnical Investigation - Proposed High-Rise Development 116 Beech Street	Project N0: OTT-23005943-A0
Date Cored Jun 06, 2023		Rock Core Photographs	FIG B-9

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-23005943-A0

AGAT WORK ORDER: 23Z038356

SOIL ANALYSIS REVIEWED BY: Nivine Basily, Inorganics Report Writer

DATE REPORTED: Jun 28, 2023

PAGES (INCLUDING COVER): 5

VERSION*: 1

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

Notes

Disclaimer:

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

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

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

Certificate of Analysis

AGAT WORK ORDER: 23Z038356

PROJECT: OTT-23005943-A0

CLIENT NAME: EXP SERVICES INC **ATTENTION TO: Daniel Wall**

SAMPLING SITE: **SAMPLED BY:**

(Soil) Inorganic Chemistry

DATE RECEIVED: 2023-06-20 **DATE REPORTED: 2023-06-28**

			SAMPLE DESCRIPTION:	BH1 Run 1 11'2"-11'5"	BH4 SS8 17. 5'-19.5'
			SAMPLE TYPE:	Rock	Soil
			DATE SAMPLED:	2023-06-07	2023-06-05
Parameter	**Unit**	**G / S**	**RDL**	**5088394**	**5088397**
Chloride (2:1)	µg/g		2	67	70
Sulphate (2:1)	µg/g		2	25	100
pH (2:1)	pH Units		NA	8.88	8.98
Resistivity (2:1) (Calculated)	ohm.cm		1	4900	3510

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

5088394-5088397 pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.

Analysis performed at AGAT Toronto (unless marked by *)

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
NIVINE BASILY
CHEMIST

Certified By: N Basily

Quality Assurance

CLIENT NAME: EXP SERVICES INC **AGAT WORK ORDER: 23Z038356**

PROJECT: OTT-23005943-A0 **ATTENTION TO: Daniel Wall**

SAMPLING SITE: **SAMPLED BY:**

Soil Analysis

RPT Date: Jun 28, 2023			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
(Soil) Inorganic Chemistry															
Chloride (2:1)	5095348		407	407	0.0%	< 2	93%	70%	130%	102%	80%	120%	104%	70%	130%
Sulphate (2:1)	5095348		739	737	0.3%	< 2	100%	70%	130%	105%	80%	120%	101%	70%	130%
pH (2:1)	5092758		8.23	8.21	0.2%	NA	94%	80%	120%						

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

NIVINE BASILY

Certified By: Nivine Basily

Method Summary

CLIENT NAME: EXP SERVICES INC **AGAT WORK ORDER: 23Z038356**

PROJECT: OTT-23005943-A0 **ATTENTION TO: Daniel Wall**

SAMPLING SITE: **SAMPLED BY:**

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

Chain of Custody Record

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

Laboratory Use Only

Work Order #: 23Z038356

Cooler Quantity: n/a - no ice/packs

Arrival Temperatures: 28.6|28.6|28.2 4.2|4.7|

Custody Seal Intact: ☐Yes ☐No ☒N/A

Notes: bagged ice

Report Information:

Company: Exp Services Inc.
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 ☑ 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-23005943-A0
Site Location: 116 Beech St. Ottawa
Sampled By: EXP
AGAT Quote #: PO:

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

Invoice Information:

Bill To Same: Yes ☑ No ☐

Company:
Contact:
Address:
Email:

Sample Matrix Legend

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

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI, DOC Y / N	O. Reg 153							O. Reg 558	O. Reg 406										Potentially Hazardous or High Concentration (Y/N)
							Metals & Inorganics	Metals - ☐ CrVI, ☐ Hg, ☐ HWSB	BTEX, F1-F4 PHCs	Analyze F4G if required ☐ Yes ☐ No	PAHs	PCBs	VOC	Landfill Disposal Characterization TCLP: TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	Excess Soils SPLP Rainwater Leach SPLP: ☐ Metals ☐ VOCs ☐ SVOCs	Excess Soils Characterization Package pH, ICPMS Metals, BTEX, F1-F4	Salt - EC/SAR	pH	Sulphate	chloride	resistivity				
BH 1 Run 1 11'2"-11'5"	June 7	AM PM	1	rock														✓	✓	✓	✓				
BH 4 SS8 17.5-19.5'	June 5	AM PM	1	S														✓	✓	✓	✓				
		AM PM																							
		AM PM																							
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		AM PM																							
		AM PM																							

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Jeff MacMillan	June 20/23	4:36 PM	C. Griffiths	JUN 20 2023	16h40
CC to Auro	JUN 21 2023	15h30		June 22	8:40 Am

Page 1 of 1

Nº: T 129246

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

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