Preliminary Geotechnical Investigation
Proposed Residential Development
178 Nepean Street, 219/223 Bank Street, Ottawa, ON

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Smart Living Properties

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Executive Summary

A preliminary geotechnical investigation was undertaken at the site of the proposed multi-use residential/commercial development to be located at 178 Nepean Street and 219/223 Bank Street, Ottawa, Ontario (Figure 1). Terms of reference for this project was provided in exp proposal OTT-22018926-A0 dated July 21, 2022.

The proposed development will consist of a new nine (9) storey building with one basement level. The façade of the original buildings along Nepean Street, Bank Street and Lisgar Street will be preserved and incorporated in the design of the new building. The proposed building will include 245 residential units, 9 commercial retail units at the ground floor level and 6 commercial retail units in the basement level. There will be no underground parking. Drawing C200 titled "Grading Plan" by EXP, dated December 9, 2022 indicates that the proposed development will have a final floor elevation of 72.83 m with a proposed underside of footing elevation of 70.5 m. Based on the Annis, O'Sullivan and Vollebekk Ltd. survey of 178 Nepean Street, the existing floor elevation ranges from 72.67 m 72.82. Based on this survey minimal to no grade raise will be required as part of the proposed development.

The site is currently occupied by low rise budlings over most of the site area and therefore areas that were available for drilling and for equipment were limited due to the current building lines extensions to the city limits (sidewalks), access issue as well as well as underground services. As indicated in our proposal, this investigation should be considered as preliminary and additional and more detailed investigation is required to be completed once the existing buildings are demolished or access to additional areas in the interior of the existing buildings are granted.

The geotechnical investigation consists of three (3) boreholes (Borehole Nos. 1 to 3) advanced to termination/ refusal depths ranging from 7.0 m to 9.9 m below the existing ground surface/floor slab. The borehole fieldwork was completed in two stages, with the exterior boreholes (Borehole Nos. 2 and 3) drilled on November 23, 2022 and the interior borehole (Borehole No. 1) drilled on December 5, 2022. Borehole No. 1 was drilled in the southwest corner of 223 Bank Street basement using hand portable drilling equipment. Boreholes No. 2 and 3 were drilled using a geoprobe track mounted drill rig on the exterior of the existing buildings.

The investigation revealed the subsurface condition in Borehole No. 1 consists of a concrete slab, fill and hard native clay overlying loose to compact glacial till, the latter extending to a depth of 8.3 m (Elevation 62.1 m) below basement floor slab where shale bedrock was contacted. The subsurface conditions in Borehole Nos. 2 and 3 consists of fill over stiff to very stiff clay, extending to depths of 4.0 m to 4.3 m below the existing ground surface, underlain by very loose to compact glacial till. Refusal to sampling was encountered in Borehole Nos. 2 and 3 at depths of 7.1 and 7.0 m (Elevation 65.2 m and 65.1 m), respectively. Based on the available information, it is suspected that refusal to sampling in Borehole Nos. 2 and 3 was likely met on boulders within the glacial till deposit. The groundwater table was measured at 6.1 to 7.0 m depths (Elevation 65.1 m to Elevation 64.3 m). It is suspected that the groundwater has not yet stabilized over the short period where reading was taken following the drilling operation. Based on the moisture content measure in Borehole Nos. 2 and 3, the groundwater is expected to be between 2.0 m to 4.0 m below grade. Additional groundwater readings should be collected to confirm the stabilized water level on-site.

Preliminary liquefaction analysis completed by exp revealed the potential of liquefaction of the loose to compact glacial till contacted in Borehole Nos. 2 and 3. The dense glacial till in Borehole No. 1 was found to be non-liquifiable. Therefore, additional, and more detailed investigation is required to be completed at the site to collect additional data on the liquification potential of the overburden and its extent in the remainder of the site and to confirm seismic site class.

Based on the available data, the seismic site class for the site is **Class F.** As indicated above additional investigation is required to confirm this classification. If confirmed, site improvement can be implemented to address the liquefaction potential of some of the soils and to improve the seismic site class which can be likely to be raised to Site **Class C**. A specialized site densification contractor must be contacted to explore the possibility of site improvement and to establish the best method to achieve it. However, this need to be confirmed following the completion of additional boreholes and testing at the site such as Multi-channel Analysis of Surface Waves (MASW) testing.

It is understood that the proposed underside of footing elevation for proposed building is 70.5 m. Based on a review of the boreholes, footings at an elevation of Elevation 70.5 m, would be founded on or just above the stiff to very stiff native clay which is underlain by liquifiable glacial till in Borehole Nos. 2 and 3 and by non-liquifiable glacial till in Borehole No. 1. Assuming that the issue of liquifiable till is addressed by site improvement, it is unlikely the available SLS bearing of the stiff silty clay will be capable to support the footings of the proposed nine (9) storey building.

Therefore, it is considered that the proposed nine storey building should be supported by closed end steel pipe or steel H-piles driven to practical refusal expected in the upper 1.0 m of the shale bedrock that was contacted at 8.3 m depth below existing grade (Elevation 62.1 m) in Borehole No. 1. The closed end pipe piles are typically used in the Ottawa area and can be more economical compared to H-piles. Further, H-piles tend to extend deeper into the shale bedrock to achieve practical refusal and required set compared with closed end pipe piles. Therefore, closed end pipe piles are recommended for this project.

Based on the available data and preliminary liquification analysis, the slab of the proposed structure can be constructed as slab-on-grade in the vicinity of Borehole No. 1. In the vicinity of Borehole Nos. 2 and 3 the floor slab would need to be a structural slab supported on piles foundation since the post seismic settlement at these two locations is expected to exceed the maximum tolerable limit.

If the liquefaction of the glacial till is confirmed at the site as part of the detailed investigation site improvement may be explored in order to accommodate the construction of the slab-on-grade over the entire building as well improve the seismic site class. This will have to confirmed as part of the detailed investigation. A specialized site improvement contractor should be consulted for this approach to establish its feasibility.

The lowest floor level for is anticipated to be at approximately 2.0 m below the existing grade and above the recorded groundwater levels. Perimeter drains are required. The need for underfloor drainage will be assessed based on the results of the additional investigation.

The fill required at the site would have to be imported and should conform to OPSS Granular A or B, Type II as specified in the report.

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

This executive summary is a brief synopsis of the report and should not be read in lieu of reading the report in its entirety.

1.0 Introduction

A preliminary geotechnical investigation was undertaken at the site of the proposed multi-use residential/commercial development to be located at 178 Nepean Street and 219/223 Bank Street, Ottawa, Ontario (Figure 1). Terms of reference for this project was provided in exp proposal OTT-22018926-A0 dated July 21, 2022.

The proposed development will consist of a new nine (9) storey building with one basement level. The façade of the original buildings along Nepean Street, Bank Street and Lisgar Street will be preserved and incorporated in the design of the new building. The proposed building will include 245 residential units, 9 commercial retail units at the ground floor level and 6 commercial retail units in the basement level. There will be no underground parking. Drawing C200 titled "Grading Plan" by EXP, dated December 9, 2022 indicates that the proposed development will have a final floor elevation of 72.83 m with a proposed underside of footing elevation of 70.5 m. Based on the Annis, O'Sullivan and Vollebekk Ltd. survey of 178 Nepean Street, the existing floor elevation ranges from 72.67 m 72.82. Based on this survey minimal to no grade raise will be required as part of the proposed development.

The site is currently occupied by low rise budlings over most of the site area and therefore areas that were available for drilling and for equipment's were limited due to the current building lines extensions to the city limits (sidewalks), access issue as well as well as underground services. As indicated in our proposal, this investigation should be considered as preliminary and additional and more detailed investigation would require to be completed once the existing buildings are demolished or access to additional areas in the interior of the existing buildings are granted.

The preliminary geotechnical investigation was undertaken to:

(a) Establish subsurface soil and groundwater conditions three (3) borehole locations.

(b) Discuss feasibility of raising the grade at the site.

(c) Discuss foundation alternatives available including founding depth, Serviceability Limit State (SLS) bearing pressure, and factored geotechnical resistance at Ultimate Limit State (ULS) of the founding strata.

(d) Anticipated total and differential settlements for different foundation options.

(e) Classify the site for Seismic Site Response in accordance with the 2012 Ontario Building Code requirements and provide a general comment regarding the liquefaction potential of on-site soils.

(f) Discuss slab-on-grade floor construction.

(g) Static and seismic earth forces on basement walls;

(h) Comment on excavation conditions anticipated and dewatering requirements during construction.

(i) Discuss backfilling requirements and the suitability of the on-site soil for backfilling purposes.

(j) Comment on subsurface concrete requirements.

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

2.0 Site Description

The site area is approximately 0.5 acres in size with frontage along Nepean Street, Lisgar Street and Bank Street. The site is within a mature neighbourhood which includes retail shops and services and multi storey residential units. The existing property consists of multi-storey residential buildings with commercial retail at street level. The site is bounded to the north, east and south by Nepean Street, Bank Street and Lisgar Street, respectively. The site is bounded to the east by residential buildings and a parking lot.

The ground surface elevations at exterior borehole locations ranges from Elevation 72.31 m to 72.11 m. The elevation for the borehole drilled in the basement was Elevation 70.40 m.

3.0 Procedure

The geotechnical investigation consists of three (3) boreholes (Borehole Nos. 1 to 3) advanced to termination/ refusal depths ranging from 7.0 m to 9.9 m below the existing ground surface/basement floor slab. The borehole fieldwork was completed in two stages, with the exterior boreholes (Borehole Nos. 2 and 3) drilled on November 23, 2022 and the interior borehole (Borehole No. 1) drilled on December 5, 2022. The fieldwork for both stages was supervised on a full-time basis by a representative from EXP.

The locations and the geodetic elevations of the boreholes were established on site by EXP and are shown on the Borehole Location Plan, Figure 2.

The borehole locations were cleared of private and public underground services, prior to the start of drilling by USL-1 Underground Service Locators acting as sub-contractor to exp.

Borehole No. 1 was drilled in the southwest corner the basement of 223 Bank Street using hand portable drilling equipment advanced with a combination of continuous Standard Penetration Testing (SPTs) using a third weight hammer and advancing of a casing using a Hilti drill to a termination depth of 9.9 m below the basement level. The SPT "N" values from this borehole have been corrected to N values from a standard weight hammer. The presence of the bedrock was proven in this borehole beyond the refusal depth by conventional rock coring techniques using the N-size core barrel. A field record of wash water return, colour of wash water and any sudden drops of the core barrel were kept during rock coring operations.

Boreholes No. 2 and 3 were drilled using a geoprobe track mounted drill rig equipped soil sampling capabilities. SPT were performed in all the boreholes at depth intervals of 0.75 m to 1.5 m with soil samples retrieved by the split-barrel sampler. The undrained shear strength of the clayey soil was measured by conducting a penetrometer test on selected recovered soil samples and in-situ shear vane tests at selected depth intervals.

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.

Nineteen (19) mm diameter standpipes with slotted section were installed in all the boreholes for long-term monitoring of the groundwater levels. The standpipes 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 drilling.

On completion of the fieldwork, the soil and rock samples were transported to the EXP laboratory in Ottawa. The soil and rock samples were visually examined in the laboratory by a senior geotechnical engineer and logs of boreholes prepared. All soil samples were classified in accordance with the Unified Soil Classification System (USCS) and the modified Burmeister System (as per the 2006 Fourth Edition Canadian Foundation Engineering Manual (CFEM)).

The geotechnical engineer also assigned the laboratory testing program which is summarized in Table I.

Table I: Summary of Laboratory Testing Program	
Type of Test	Number of Tests Completed
Soil Samples	
Moisture Content Determination	32
Unit Weight Determination	5
Grain Size Analysis	4
Atterberg Limit Determination	4
Rock Samples	
Unconfined Compressive Strength	1
Unit Weight Determination	1

4.0 Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface conditions and groundwater levels from the boreholes are given on the attached Borehole Logs, Figure Nos. 3 to 5. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

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

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

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

4.1 Asphaltic Concrete

A 75 mm thick layer of asphaltic concrete was encountered at the surface of Borehole No. 3.

4.2 Concrete

A 100 mm to 125 mm thick concrete slab was encountered at the surface of Boreholes Nos 1 and 2.

4.3 Granular Fill

A 50 mm thick layer granular fill was encountered underlying the concrete in Borehole No. 1.

4.4 Fill

The granular fill in Borehole No. 1 and the concrete or asphaltic concrete in Boreholes Nos. 2 and 3 are underlain by silty sand with gravel fill which extends to 0.2 to 1.4 m depth (Elevation 71.0 m to 70.2 m). Based on the SPT N-values of 4 to 10 the fill is in a loose to compact state. The natural moisture content of the fill ranges from 4 percent to 17 percent.

4.5 Clay

A clay deposit was contacted below the fill and extends to depths of 2.1 to 4.3 m (Elevation 68.3 m to Elevation 68.0 m). The clay has an undrained shear strength ranging from 60 kPa to 200 kPa indicating a firm to hard consistency. The natural moisture content and unit weight of the clay ranges from 34 percent to 77 percent and 17.3 kN/m^3 to 18.7 kN/m^3, respectively.

The results from the grain-size analysis and Atterberg limits determination of two (2) samples of the clay are summarized in Table II. The grain-size distribution curves are shown in Figures 6 and 7.

Table II Summary of Results from Grain-Size Analysis and Atterberg Limit Determination – Clay Samples

Borehole No. (BH) – Sample No. (SS)	Depth (m)	Grain-Size Analysis (%)				Atterberg Limits (%)				Soil Classification (USCS)
		Gravel	Sand	Silt	Clay	Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	
BH 1-SS3	0.8-1.4	0	3	41	56	47	26	57	31	Clay of High Plasticity (CH)
BH3-SS3	4.6-5.2	0	8	39	53	40	29	63	34	Clay of High Plasticity (CH)

Based on a review of the laboratory test results, the soil may be classified as a clay of high plasticity (CH) in accordance with the USCS.

4.5 Glacial Till

The clay in all the boreholes is underlain by a glacial till deposit contacted at 2.1 m to 4.3 m depths (Elevation 68.3 m to Elevation 68.0 m). The glacial till contains varying amounts of gravel, sand, silt and clay as well as cobbles and boulders. Based on SPT N-values of 9 to 65 the glacial till is generally in a loose to very dense state. High N-values for low sampler penetration, such as 50 for 50 mm of sampler penetration were recorded and may be a result of the sampler contacting cobbles or boulders within the glacial till deposit. In Borehole No. 2 low SPT N-values ranging from the weight of the SPT hammer to 2 blows were recorded indicating a very loose state. The natural moisture content of the glacial till ranges from 8 percent to 19 percent.

The results from the grain-size analysis conducted on two (2) samples of the glacial are summarized in Table III. The grain-size distribution curves are shown in Figures 8 and 9.

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

Borehole No. (BH) – Sample No. (SS)	Depth (m)	Grain-Size Analysis (%)				Soil Classification (USCS)
		Gravel	Sand	Silt	Clay	
BH3 SS7	5.3-5.9	15	43	33	9	Silty sand with gravel (SM)
BH4 SS6	1.5 - 2.1	14	38	34	14	Silty Sand (SM)

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

4.6 Refusal and Shale Bedrock

Refusal was encountered in Borehole Nos. 2 and 3 at 7.0 m to 7.1 m depth (Elevation 65.2 m to 65.1 m) and likely met on cobbles and boulders within the glacial till layer. In Borehole No. 1 refusal was encountered at 5.8 m depth (Elevation 64.6 m). Beyond refusal, this borehole was cased and advanced further using casing and coring with N size coring equipment. The coring operation revealed that refusal was met on cobbles and boulders within the glacial till overburden which extended to a depth of 8.3 m depth (Elevation 62.1) where shale bedrock was encountered and proven to 9.9 m depth. The shale is grey to black in colour with a Total Core Recovery (TCR) and a Rock Quality Designation (RQD) of 100 percent to 65 percent, respectively. The RQD value indicates a fair quality shale bedrock. The results of the unit weight and unconfined compressive strength tests undertaken on one (1) sample of the intact rock are given on Table IV.

Table IV: Unit Weight and Unconfined Compressive Strength of Rock Cores			
Borehole #	**Depth (m)**	**Unit Weight (kN/m^3)**	**Unconfined Compressive Strength (MPa)**
1	9.4 – 9.5	25.3	32.1

On the basis of its unconfined compressive strength, the rock may be described as medium strong.

4.7 Groundwater

A summary of the groundwater level measurements taken in the boreholes equipped with standpipes on December 7,2022 is shown in Table VII.

Table V: Summary of Groundwater Level Measurements			
Borehole No. (BH)	Ground Surface Elevation (m)	Elapsed Time in Days from Date of Installation	Depth Below Ground Surface (Elevation), m
BH-01	70.40	2 days	6.1 (64.3)
BH-02	72.31	14 days	Dry to 7.1 (65.5 m)
BH-03	72.11	14 days	7.0 (65.1)

The above table indicates the groundwater level to range from 6.1 to 7.0 m depths (Elevation 65.1 m to Elevation 64.3 m) and was found to be deeper than 7.1 m (Elevation 65.5 m) in Borehole No. 2. The water level readings were taken two (2) to fourteen (14) days following completion of drilling and may have not stabilized over the short time where readings were collected.

Based on the moisture content measure in Borehole Nos. 2 and 3, the groundwater is expected to be between 2 m to 4 m below grade. Additional groundwater readings should be collected to confirm the stabilized water level on-site.

The groundwater levels were determined in the boreholes at the time and under the condition stated in this report. 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.

5.0 Seismic Site Classification

The geotechnical conditions at the site consists of 75 mm to 125 mm of asphaltic concrete or concrete underlain by granular fill extending to 0.2 m to 1.4 m depth (Elevation 71.0 m to 70.2 m). The fill is underlain by a deposit of clay which extends to depths of 2.1 to 4.3 m (Elevation 68.3 m to Elevation 68.0 m). Underlying the clay is a layer of glacial till with SPT N-values ranging from the weight of the SPT hammer to 65 indicating the glacial till is in a very loose to very dense state. Shale bedrock was confirmed at 8.3 m depth (Elevation 62.1). The groundwater level ranges from 6.1 to 7.0 m. (Elevation 65.1 m to Elevation 64.3 m).

Exp has carried out preliminary liquefaction analysis using the limited available data as per the method derived from D.P. Coduto (1999) in “Geotechnical Engineering Principles and Practices” and the guidelines presented in the “Canadian Foundation Engineering Manual, 4th Edition” for all three boreholes at the site. This analysis used a design earthquake magnitude of 7.0 and calculated two cases, the first for short term conditions with the groundwater level ranging from 65.1 m to 64.3 m and the second case for long term conditions with the groundwater level at the top of glacial till, at elevation 68.4 m. The results of this analysis indicate the Borehole No. 1 is not considered to have liquifiable soil. The glacial till deposit in Boreholes Nos. 2 and 3 were liquifiable with estimated post-liquefaction settlements of 240 mm and 67 mm, respectively.

Based on these results the site class is “F” should be assigned as per OBC (2012).

As indicated previously, an additional investigation is required to confirm the above classification, the presence of liquifiable soils and its extent throughout the site. If confirmed, site improvement can be implemented to address the liquefaction potential of some of the soils and to improve the seismic site class which can be likely to be raised to Site **Class C**. A specialized site densification contractor must be contacted to explore the possibility of site improvement and best method to achieve it.

The additional field testing should comprise of drilling additional boreholes to obtain complete coverage of the site as well as completion and a Multi-channel Analysis of Surface Waves (MASW)..

6.0 Grade Raise Restrictions

Based on the Annis, O'Sullivan and Vollebekk Ltd. survey of 178 Nepean Street, the existing floor elevation ranges from 72.67 m to 72.82 m. Drawing C200 by EXP indicates that the proposed development will have a final floor elevation of 72.83 m. Assuming the existing floor elevations of 219/223 Bank Street are similar to 178 Nepean Street, minimal to no grade raise is expected. The proposed grade raises are considered feasible from a geotechnical point of view.

7.0 Foundation Considerations

It is understood that the proposed underside of footing elevation for proposed building is 70.5 m. Based on a review of the boreholes, footings at an elevation of Elevation 70.5 m, would be founded on the stiff native clay which is underlain by possible liquifiable glacial till in Borehole Nos. 2 and 3 and by non-liquifiable till in Borehole No. 1.

Assuming that the issue of liquifiable glacial till is addressed by site improvement, it is unlikely the available SLS bearing of the stiff to very stiff silty clay will be capable to support the footings of the proposed nine (9) storey building.

Based on the available data, the proposed nine storey building may be supported by closed end steel pipe or steel H-piles driven to practical refusal expected in the upper 1 m of the shale bedrock that was contacted at 8.3 m depth below existing grade (Elevation 62.1 m) in Borehole No. 1. The closed end pipe piles are typically used in the Ottawa area and can be more economical compared to H-piles. Further, H-piles tend to extend deeper into the shale bedrock to achieve practical refusal and required set compared with closed end pipe piles. Therefore, closed end pipe piles are recommended for this project.

The factored geotechnical resistance at ULS for various pile sections is shown in Table VI. The factored geotechnical resistance values at ULS are based on steel piles with a yield strength of 350 MPa and concrete compressive strength of 35 MPa and includes a geotechnical resistance factor of 0.4. Since the piles are expected to meet refusal within the shale bedrock, the factored geotechnical resistance at ULS will govern the design, since the bearing pressure at SLS for 25 mm of settlement will be greater than the factored geotechnical resistance at ULS.

Table VI: Factored Geotechnical Resistance at Ultimate Limit State (ULS) for Steel Pipe and H-Piles

Pile Section	Pile Section Size	Factored Geotechnical Resistance at ULS (kN)
Steel Pipe	245 mm O.D. by 10 mm wall thickness	1275
	245 mm O.D. by 12 mm wall thickness	1445
	324 mm O.D. by 12 mm wall thickness	2120
Steel H	HP 310 x 79	1260
	HP 310 x 110	1775
	HP 310 x 125	2000

Total settlement of piles designed for the above recommended factored geotechnical resistance at ULS are expected to be less than 10 mm.

To achieve the capacity given previously, the pile driving hammer must seat the pile into shale bedrock without overstressing the pile material. For guidance purposes, it is estimated that a hammer with rated energy of 54 kJ to 70 kJ (40,000 to 52,000 ft. lbs.) per blow would be required to drive the piles to practical refusal in the shale bedrock. Practical refusal is considered to have been achieved at a set of 5 blows for 6 mm or less of pile penetration. However, the driving criteria for a particular hammer-pile system must be established at the beginning of the project. This may be achieved with a Pile Driving Analyzer.

The glacial till is expected to contain cobbles and boulders. It is therefore recommended that the pile tips should be reinforced with a 25-mm thick steel plate and equipped with a driving shoe in accordance with Ontario Provincial Standard Drawing (OPSD) 3001.100, Type II, dated November 2017 and shown in Appendix B.

A number of test piles should be monitored with the Pile Driving Analyzer (PDA) during the initial driving and re-striking at the beginning of the project and 3 percent of the piles tested should be subjected to CAPWAP analysis. This monitoring will allow for the evaluation of transferred energy into the pile from the hammer, determination of driving criteria and an evaluation of the geotechnical resistance at ULS of the piles. Depending on the results of the pile driving analysis, the pile capacity may have to be proven by at least one pile load test for each pile type before production piling begins. If necessary, the pile load test should be performed in accordance with American Society for Testing and Materials (ASTM) D 1143.

Closed-end pipe piles tend to displace a relatively large volume of soil. When driven in a cluster or group, they may tend to jack up the adjacent piles in the group. Consequently, the elevation of the top of each pile in a group should be monitored immediately after driving and after all the piles in the group have been driven. This is to ensure that the piles are not heaving. Any piles found to heave more than 3 mm should be re-tapped.

Piles driven at the site may be subject to relaxation, i.e. loss of load carrying capacity with time. Therefore, it is recommended that the piles should be re-struck, minimum of 24 hours after initial driving to determine if the piles have relaxed. If relaxation is observed, this procedure should be repeated every 24 hours until it can be proven that relaxation is no longer a problem.

The installation of the piles at the site should be monitored on a full-time basis by a geotechnician working under the direction and supervision of a qualified geotechnical engineer to verify that the piles are driven in accordance with the project specifications.

The concrete grade beams and pile caps for heated structures should be protected from frost action by providing the beams and caps with 1.5 m of earth cover. For non-heated structures, the pile caps and beams should be provided with 2.4 m of earth cover in areas where the snow will be removed and 2.1 m of cover in areas where the snow will not be removed. Alternatively, frost protection may be provided by rigid insulation or a combination of earth cover and rigid insulation.

If site improvement is considered feasible to address the liquification potential as well as potential improvement the site class and SLS of the loose till layers, other options for founding of the building on shallow foundation such as raft foundation can be explored as part of the detailed investigation.

8.0 Floor Slab and Drainage Requirements

Based on the available data and preliminary liquification analysis, the slab of the proposed structure can be constructed as slab-on-grade in the vicinity of Borehole No. 1. In the vicinity of Borehole Nos. 2 and 3 the floor slab would need to be a structural slab supported on piles foundation since the post seismic settlement at these two locations is expected to exceed the maximum tolerable limit.

As part of the detailed investigation and if the liquefaction of the glacial till is confirmed at the site, site improvement may be explored in order to allow the construction of the slab on grade over the entire building as slab on grade as well improve the seismic site class. This will have to confirmed as part of the detailed investigation. A specialized site improvement contractor should be consulted for this approach to establish its feasibility. The lowest floor level for is anticipated to be at approximately 2.0 m below the existing grade and above the recorded groundwater levels.

Perimeter drains are required. The need for underfloor drainage will be assessed based on the results of the additional investigation.

9.0 Lateral Earth Pressures Against Subsurface Walls

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

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

$$P = K_0 h (½ \gamma h + q)$$

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

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

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

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

q = surcharge load stress, kPa

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

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

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

H = height of wall, m

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

$\frac{a_h}{g}$ = earth pressure coefficient = 0.32 for 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.

10.0 Subsurface Concrete Requirements

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

Table VII: Corrosion Test Results on Shale Bedrock Samples

Borehole – Sample No.	Depth (m)	Soli Type	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
BH 1 SS3	2.3 – 2.4	Clay	8.37	0.008	0.026	1330

The results indicate the soils have a negligible sulphate attack on subsurface concrete. The concrete should be designed in accordance with CSA A.23.1-14.

The results of the resistivity tests indicate that is corrosive to moderately corrosive to bare steel as per the National Association of Corrosion Engineers (NACE). Appropriate measures should be taken to protect the buried bare steel from corrosion.

11.0 Excavations

11.1 Excess Soil Management

A new Ontario Regulation 406/19 made under the Environmental Protection Act (November 28, 2019) was implemented as of January 1, 2021. The new regulation dictates the testing protocol that will be required for the management and disposal of excess soils. As set forth in the regulation, specific analytical testing protocols will need to be implemented and followed based on the volume of soil to be managed. The testing protocols are specific as to whether the soils are stockpiled or in situ. In either scenario, the testing protocols are far more onerous than have been historically carried out as part of standard industry practices. These decisions should be factored in and accounted for prior to the initiation of the project-defined scope of work. EXP would be pleased to assist with the implementation of a soil management and testing program that would satisfy the requirements of Ontario Regulation 406/19.

11.2 Excavations

Excavations at the site are expected to extend to a maximum depth of 3.0 m. The excavation will extend through the concrete or asphaltic concrete, fill and terminate within the clay or glacial layer. The excavations are anticipated to be above the groundwater level.

Excavations maybe undertaken by conventional heavy equipment capable of removing debris, cobbles and boulders present within the fill and cobbles and boulders within the native soils.

The excavation within the subsurface soils should comply with the most recent Occupational Health and Safety Act (OHSA), Ontario Regulations 213/91 (August 1, 1991). Based on the definitions contained in OHSA, the subsurface soils at the site are classified as Type 3 soil and sidewalls of open cut excavations must be cut back at 1H:1V from the bottom of the excavation. Below the groundwater table, the excavation side slopes are expected to slough and will eventually stabilize at a slope of 2H:1V to 3H:1V.

It is expected that side slopes noted above for the construction of the proposed building will not be able to be achieved due to space restrictions on site and consideration for the existing building façade. Excavation for the new building construction would have to be undertaken within the confines of an engineered support system (shoring system).

The need for a shoring system, the most appropriate type of shoring system and the design and installation of the shoring system should be determined by the contractors bidding on this project. The design of the shoring system should be undertaken by a professional engineer experienced in shoring design and the installation of the shoring system should be undertaken by a contractor experienced in the installation of shoring systems. The shoring system should be designed and installed in accordance with latest edition of Ontario Regulation 213/91 under the OHSA and the 2006 Fourth Edition of the Canadian Foundation Engineering Manual (CFEM). The shoring system as well as adjacent settlement sensitive structures (buildings) and infrastructure should be monitored for movement (deflection) on a periodic basis during construction operations.

Excavations that terminate within the native clay or glacial till above the groundwater table are not expected to experience a base-heave type of failure. Open cut excavations which extend below the groundwater level within the glacial till may be susceptible to instability of the base of the excavation in the form of piping or heave. Should the excavations be expected to extend below the groundwater table, EXP should be contacted prior to the start of excavation to pride comments and recommendations to minimize instability of the excavation bases.

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

11.3 De-Watering Requirements

Seepage of the surface and subsurface water into the excavations is anticipated. However, it should be possible to collect any water entering the excavations in perimeter ditches and to remove it by pumping from sumps. In areas of high infiltration or in areas where more permeable soil layers may exist, a higher seepage rate should be anticipated and will require high-capacity pumps to keep the excavation dry.

For construction dewatering, an Environmental Activity and Sector Registry (EASR) approval may be obtained for water takings greater than 50 m^3 and less than 400 m^3 per day. If more than 400 m^3 per day of groundwater are generated for dewatering purposes, then a Category 3 Permit to Take Water (PTTW) must be obtained from the Ministry of the Environment, Conservation and Parks (MECP). A Category 3 PTTW would require a complete hydrogeological assessment and would take at least 90 days for the MECP to process once the application is submitted. A PPTW or a EASR are not expected for this site, based on the proposed excavation depth of 3.0 m.

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.

11.4 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.

12.0 Backfilling Requirements and Suitability of On-site Soils for Backfilling Purposes

The materials that will be excavated will include existing granular material underlying the asphalt, fill, clay and glacial till. It is anticipated that the majority of the fill required would have to be imported and should conform to Ontario Provincial Standard Specifications (OPSS) for Granular A and B depending on their use at the site.

13.0 General Comments

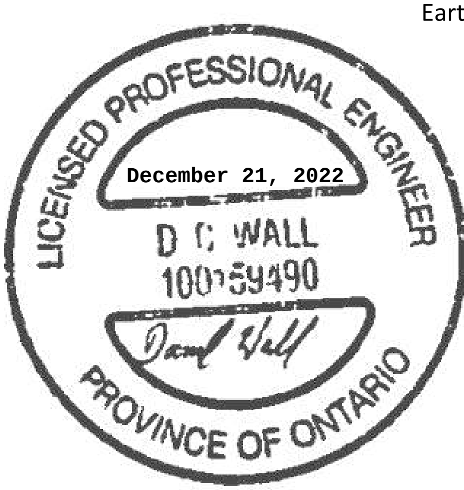
The comments given in this report are intended only for the preliminary guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes required to determine the localized underground conditions between boreholes affecting construction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

As indicated previously, an additional investigation is required in order to confirm the presence and extent of liquifiable soil as well explore the option of site improvement which may allow the lowest slab of the proposed structure to be constructed as slab-on-grade.

It is noted that the recommendations of this report may change if liquefaction analysis indicates that the on-site soils are liquefiable during a seismic event.

Daniel Wall, M.Eng., P.Eng.
Geotechnical Engineer
Geotechnical & Materials Engineering Services
Earth and Environment

Ismail M. Taki, M.Eng., P.Eng.
Senior Manager, Eastern Region
Geotechnical & Materials Engineering Services
Earth and Environment



Figures

ORIGINAL SHEET SIZE = 11" x 8.5"

0 40m 80m 160m

HORIZONTAL 1:4,000

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

DESIGN IT/MZ	PRELIMINARY GEOTECHNICAL INVESTIGATION 178 NEPEAN STREET, 219/223 BANK STREET, OTTAWA, ON	SCALE 1:4,000
DRAWN AS		SKETCH NO
DATE DECEMBER 2022	SITE LOCATION PLAN	FIG 1
FILE NO OTT-22018926-A0		

Filename: E:\OTT\OTT-22018926-A0\60 Execution\65 Drawings\22018926-A0_Geo.dwg Plotted by: SeverA
Last Saved: Dec 21, 2022 9:57 AM Last Plotted: Dec 21, 2022 9:57 AM

NOTES:

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

LEGEND

PROPERTY BOUNDARY

BH-1 (70.4m) BOREHOLE NO. & LOCATION
(X.XX) – GROUND SURFACE ELEVATION (m)

ORIGINAL SHEET SIZE = 11" x 17"

0 3m 6m 12m
HORIZONTAL 1:300

exp Services Inc. 100-2650 Queensview Drive Ottawa, ON K2B 8H6 www.exp.com	DESIGN IT/MZ DRAWN AS	PRELIMINARY GEOTECHNICAL INVESTIGATION 178 NEPEAN STREET, 219/223 BANK STREET, OTTAWA, ON	SCALE 1:300
	DATE DECEMBER 2022 FILE NO OTT-22018926-A0	BOREHOLE LOCATION PLAN	SKETCH NO FIG 2

Filename: E:\OTT\OTT-22018926-A0\60 Execution\65 Drawings\22018926-A0_Geo.dwg
Last Saved: Dec 21, 2022 9:57 AM Last Plotted: Dec 21, 2022 9:58 AM Plotted by: SeverA

Log of Borehole BH-1

Project No: OTT-22018926-A0

Project: Geotechnical Investigation - Proposed Residential Development

Location: 178 Nepean Street and 219/223 Bank Street, Ottawa, ON

Figure No. 3

Page. 1 of 2

Date Drilled: December 5th, 2022

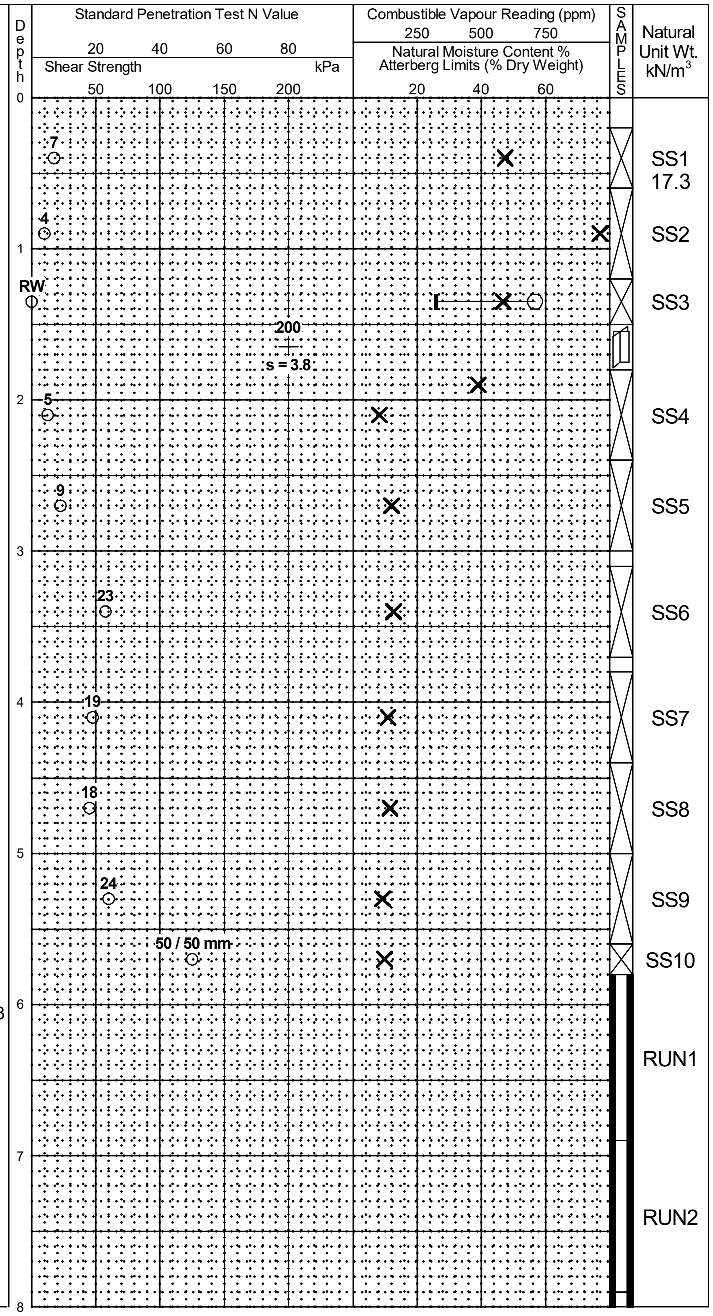
Drill Type: Hand Portable Drilling - Hilti Drill

Datum: Geodetic Elevation

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

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

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			70.4	0
		CONCRETE ~125 mm thick	70.3	
		GRANULAR FILL ~50 mm thick	70.2	
		CLAY With sand, high plasticity, brown grey, moist, (Hard)		
		Grey below 1.2 m depth		1
		GLACIAL TILL Silty sand with clay, gravel, boulders and cobbles, grey, wet, (Loose to Compact)	68.3	2
		GLACIAL TILL Cobble and boulder size shale fragments, black, slightly weathered	64.6	
▼ 64.3				6



Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe was installed in the borehole upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22018926-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
December 7, 2022	6.1	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	5.8 - 6.9	27	0
2	6.9 - 7.9	37	0
3	7.9 - 8.9	66	52
4	8.9 - 9.9	100	65

Log of Borehole BH-1

Project No: OTT-22018926-A0

Project: Geotechnical Investigation - Proposed Residential Development

Figure No. 3

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	Standard Penetration Test N Value (20, 40, 60, 80) / Shear Strength kPa (50, 100, 150, 200)	Combustible Vapour Reading (ppm) (250, 500, 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20, 40, 60)	SAMPLES	Natural Unit Wt. kN/m³
			62.4	8				
		SHALE BEDROCK Black, slightly weathered, (fair quality)	62.1					RUN3
				9				RUN4 25.3
			60.5					
		Borehole Terminated at 9.9 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe was installed in the borehole upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22018926-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
December 7, 2022	6.1	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	5.8 - 6.9	27	0
2	6.9 - 7.9	37	0
3	7.9 - 8.9	66	52
4	8.9 - 9.9	100	65

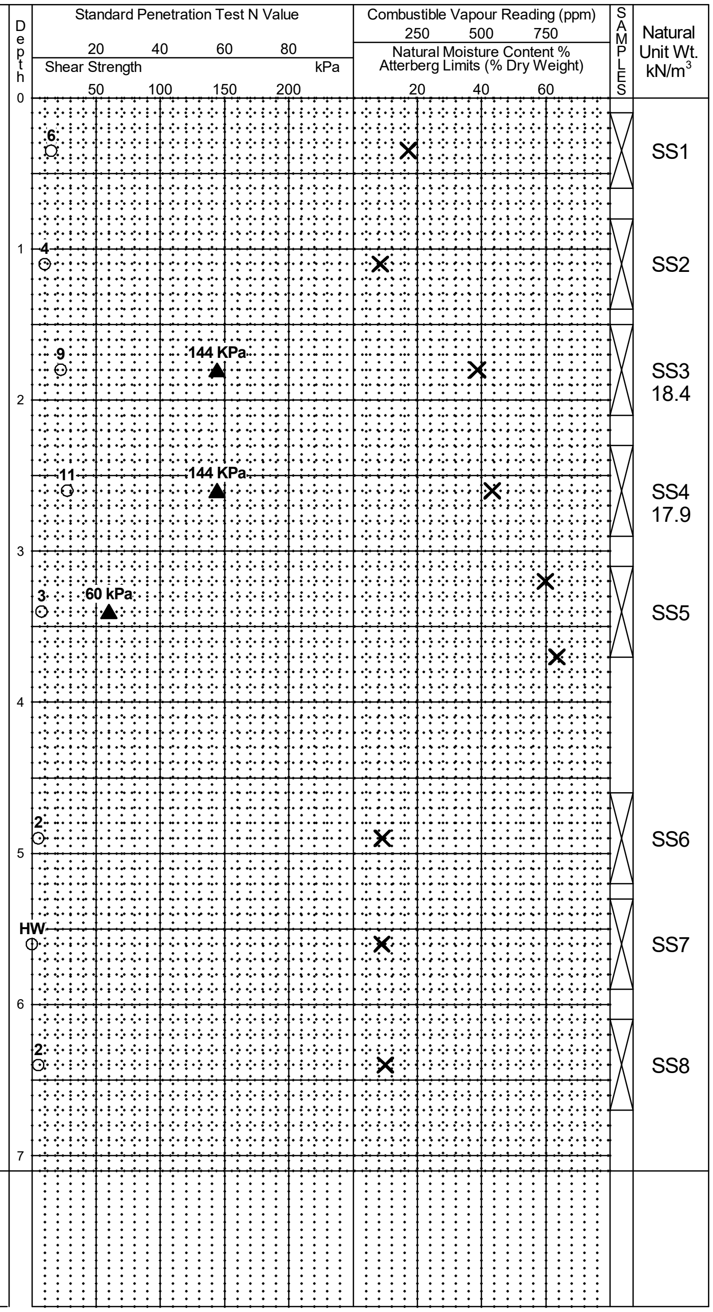
Log of Borehole BH-2

Project No:	OTT-22018926-A0
Project:	Geotechnical Investigation - Proposed Residential Development
Location:	178 Nepean Street and 219/223 Bank Street, Ottawa, ON
Date Drilled:	November 23rd, 2022
Drill Type:	Geoprobe Track-mounted Drill Rig
Datum:	Geodetic Elevation
Logged by:	M.Z.
Checked by:	I.T.

Figure No. 4

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

SOIL DESCRIPTION	Geodetic Elevation m	Depth (m)
	72.31	0
CONCRETE~ 100 mm thick	72.2	
FILL Silty sand with gravel, brown, moist, (Loose)		
CLAY With sand, high plasticity, brown grey, moist, (Very Stiff)	70.9	
Stiff below 3.0 m depth		
GLACIAL TILL Silty sand with clay, gravel, boulders and cobbles, grey, wet, (Very Loose)	68.0	
	65.2	7
Refusal at 7.1 m Depth		



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe was installed in the borehole upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22018926-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
December 7, 2022	dry	

CORE DRILLING RECORD

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

Log of Borehole BH-3

Project No: OTT-22018926-A0

Project: Geotechnical Investigation - Proposed Residential Development

Location: 178 Nepean Street and 219/223 Bank Street, Ottawa, ON

Figure No. 5

Page. 1 of 1

Date Drilled: November 23rd, 2022

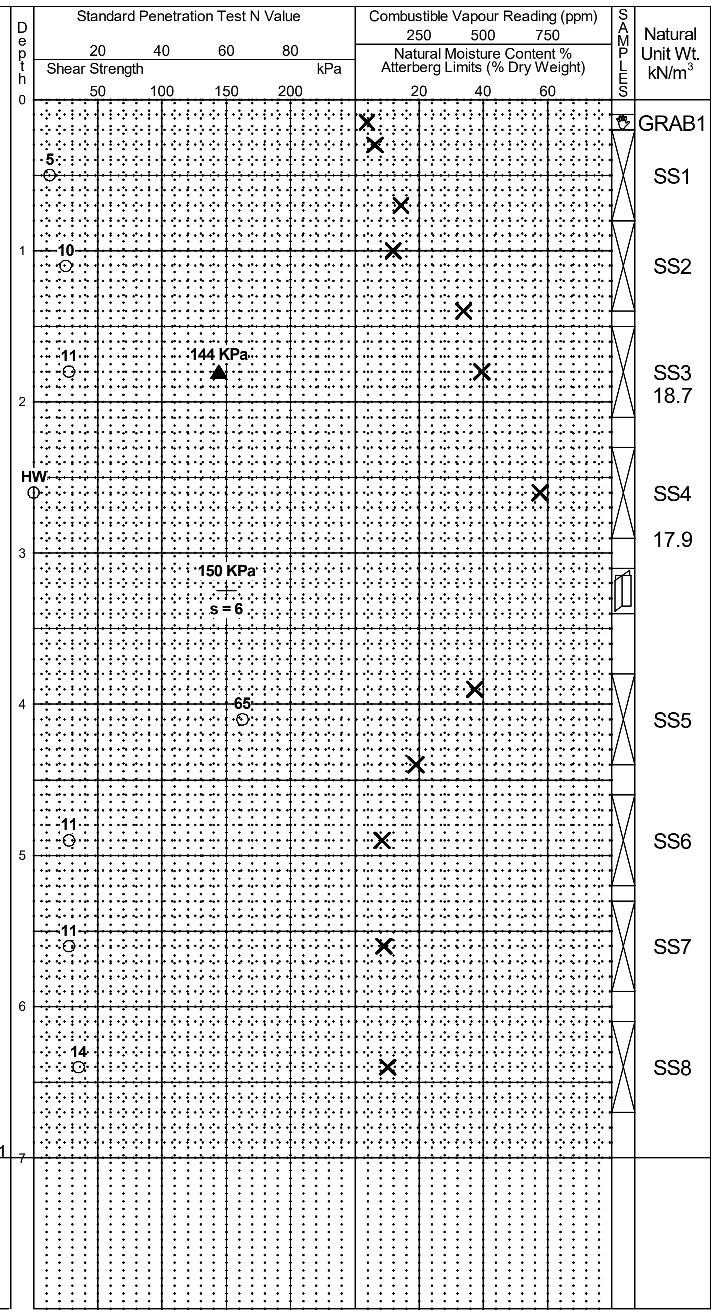
Drill Type: Geoprobe Track-mounted Drill Rig

Datum: Geodetic Elevation

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

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

Geodetic Elevation (m)	Depth (m)	SOIL DESCRIPTION	Samples	Natural Unit Wt. kN/m³
72.11	0			
72.0		**ASPHALT** ~75 mm thick	GRAB1	
71.9		**GRANULAR FILL** Sand and gravel, brown, moist.	SS1	
		FILL Silt sand with gravel and black topsoil inclusions, brown, moist, (Loose to Compact).	SS2	
71.0	1	**CLAY** With sand, high plasticity, brown grey, moist, (Very Stiff).	SS3, SS4, Shelby Tube	18.7 (SS3), 17.9 (SS4)
68.1	4	**GLACIAL TILL** Silty sand with clay, gravel, boulders and cobbles, grey, wet, (Compact to Very Dense).	SS5, SS6, SS7, SS8	
65.1 / 65.11	7	**Refusal at 7.0 m Depth**		



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe was installed in the borehole upon completion.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22018926-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
December 7, 2022	7.0	

CORE DRILLING RECORD

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

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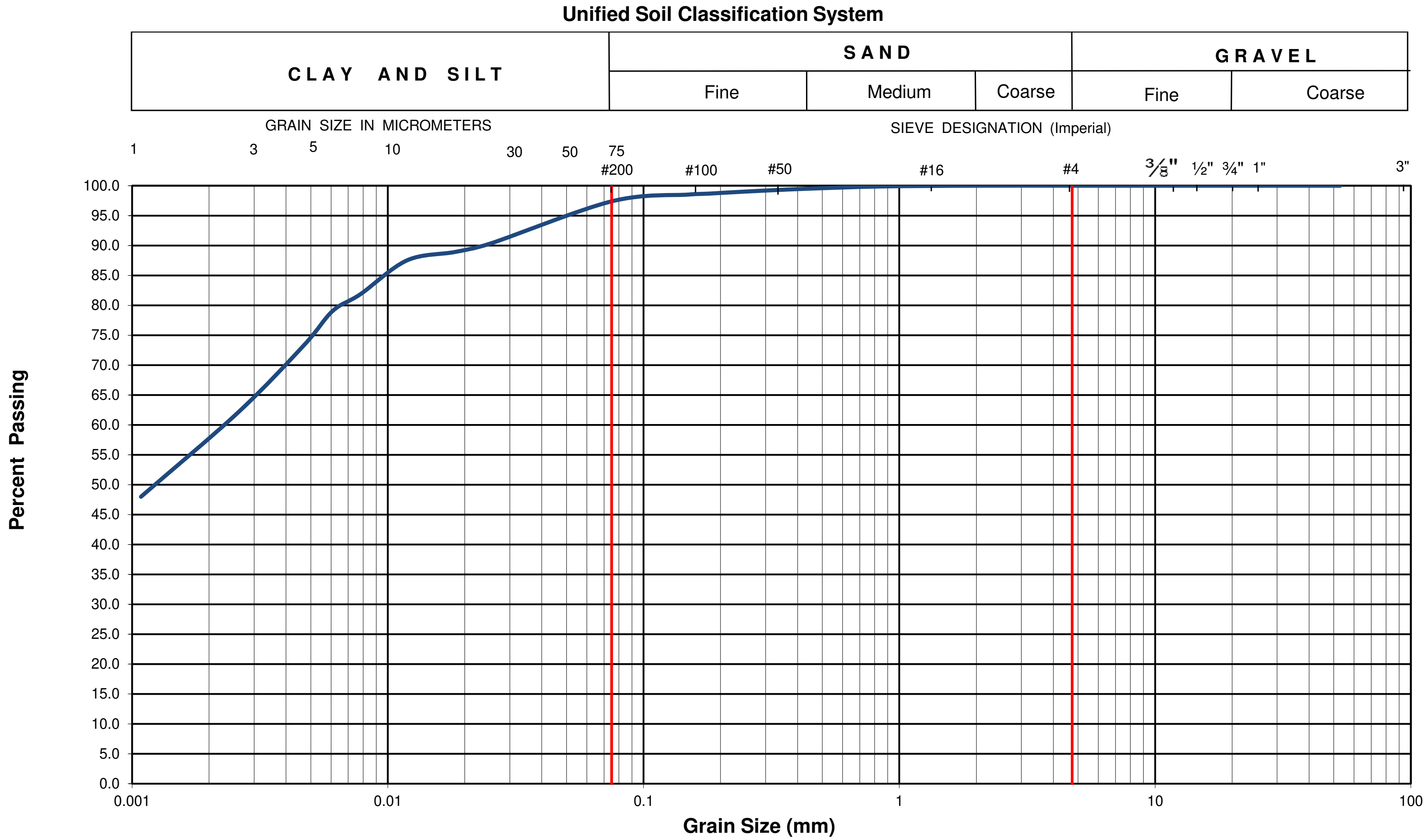
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-22018926-A0	Project Name :	Geotechnical Investigation - Proposed Residential Development					
Client :	Smart Living Properties	Project Location :	178 Nepean Street and 219/223 Bank Street, Ottawa, ON					
Date Sampled :	December 5, 2022	Borehole No:	BH1	Sample No.:	SS3		Depth (m) :	1.2-1.8
Sample Description :		% Silt and Clay	97	% Sand	3	% Gravel 0	Figure :	6
Sample Description :		Clay of High Plasticity (CH)						

EXP Services Inc.
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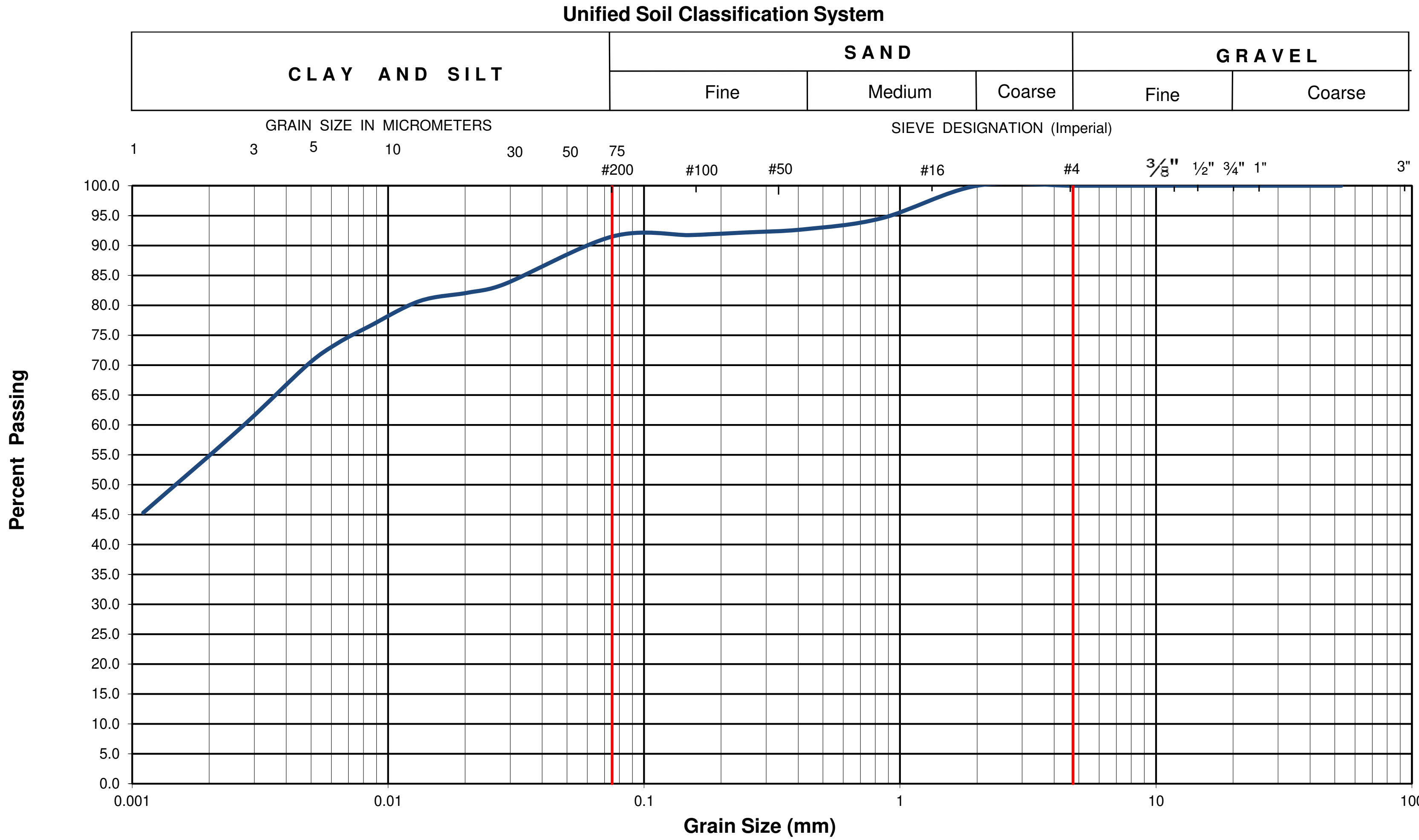
Ottawa, ON K2B 8H6

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



EXP Project No.:	OTT-22018926-A0	Project Name :	Geotechnical Investigation - Proposed Residential Development					
Client :	Smart Living Properties	Project Location :	178 Nepean Street and 219/223 Bank Street, Ottawa, ON					
Date Sampled :	November 23, 2022	Borehole No:	BH3	Sample No.:	SS3		Depth (m) :	1.5-2.1
Sample Description :		% Silt and Clay	92	% Sand	8	% Gravel	0	Figure : 7
Sample Description :		Clay of High Plasticity (CH)						

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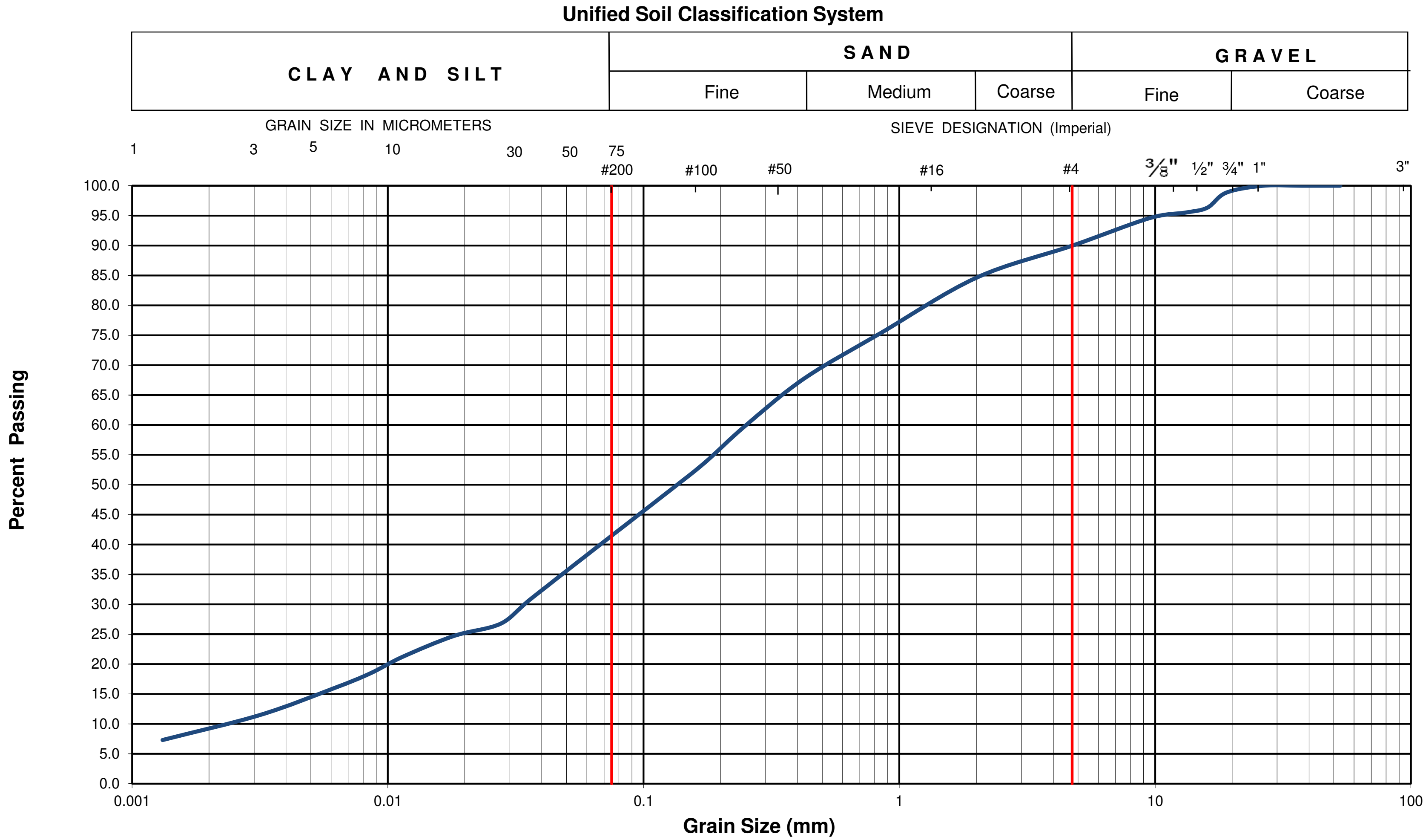
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-22018926-A0	Project Name :	Geotechnical Investigation - Proposed Residential Development					
Client :	Smart Living Properties	Project Location :	178 Nepean Street and 219/223 Bank Street, Ottawa, ON					
Date Sampled :	November 23, 2022	Borehole No:	BH3	Sample No.:	SS7		Depth (m) :	5.3-5.9
Sample Description :		% Silt and Clay	42	% Sand	43	% Gravel 15	Figure :	8
Sample Description :		Silty Sand with Gravel (SM)						

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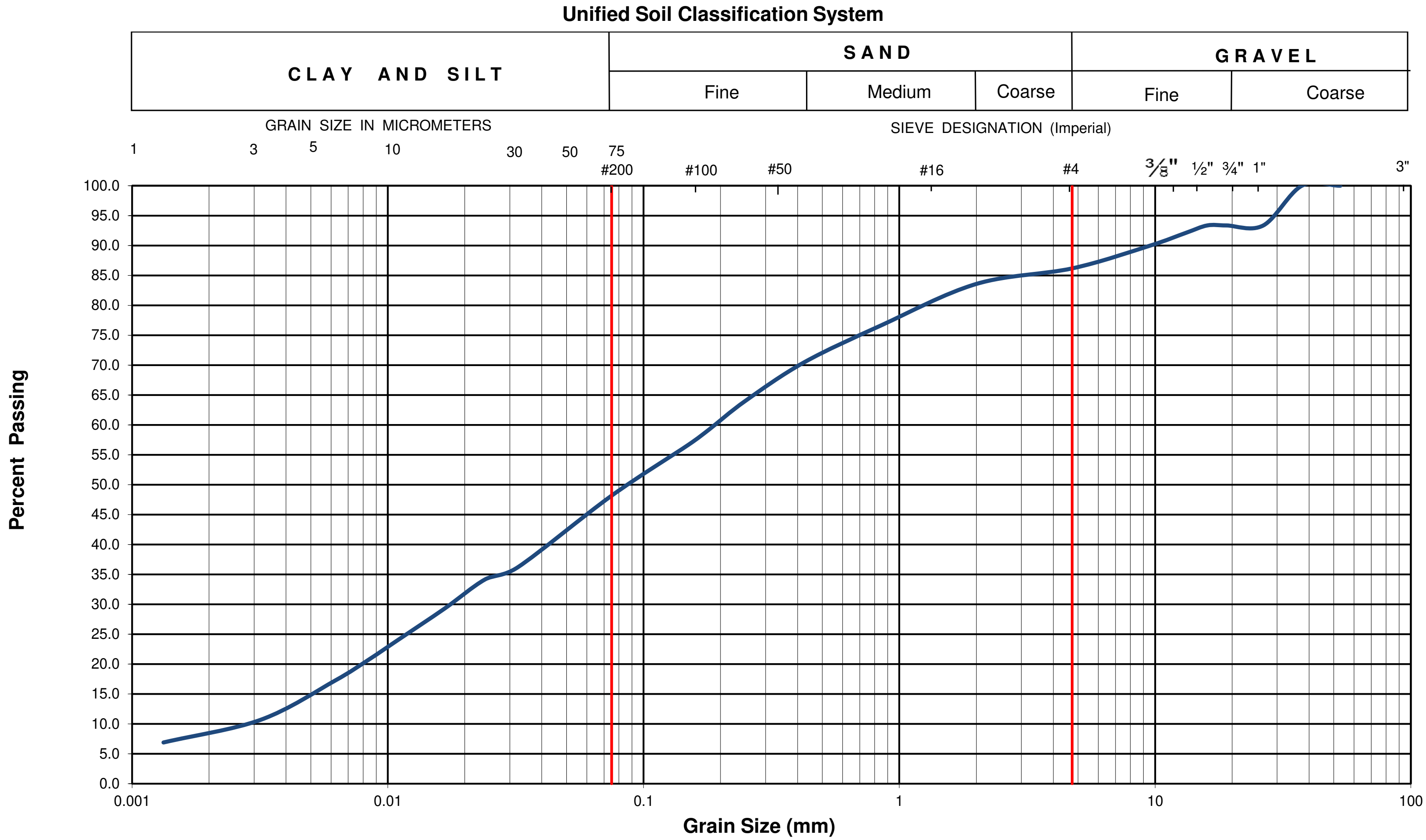
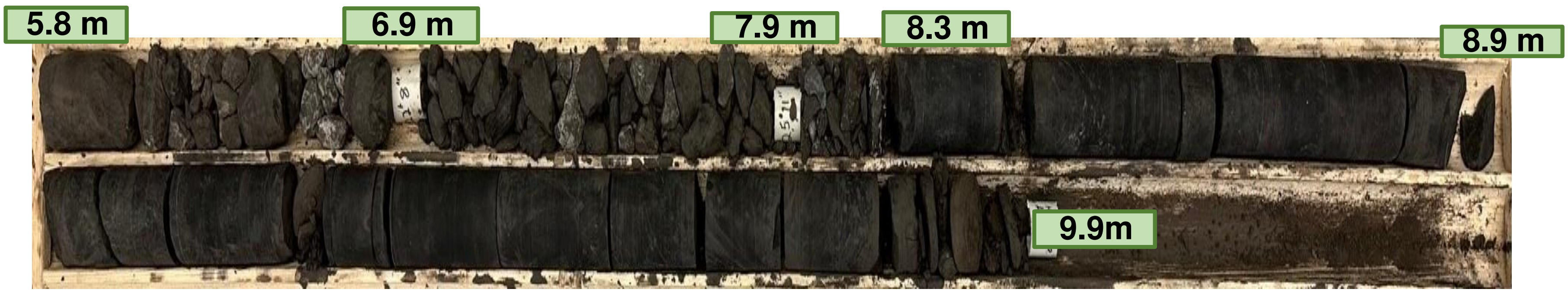
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



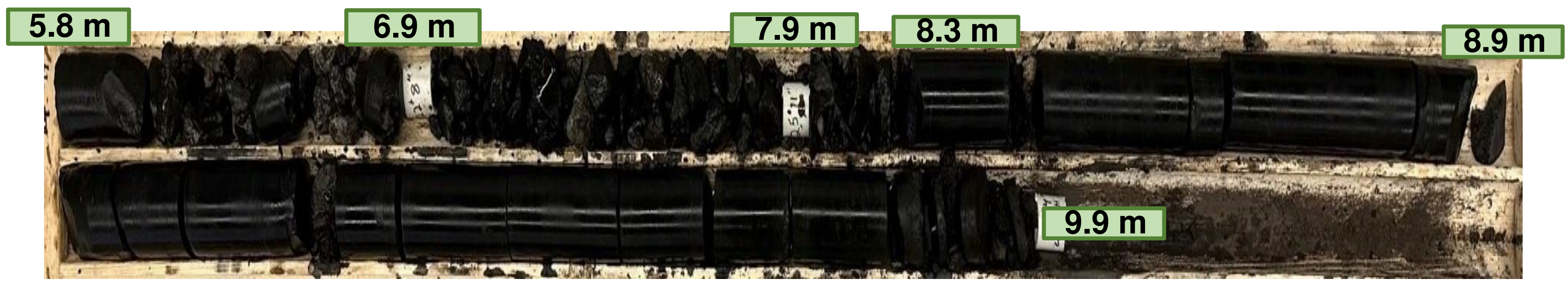
EXP Project No.:	OTT-22018926-A0	Project Name :	Geotechnical Investigation - Proposed Residential Development					
Client :	Smart Living Properties	Project Location :	178 Nepean Street and 219/223 Bank Street, Ottawa, ON					
Date Sampled :	December 5, 2022	Borehole No:	BH1	Sample No.:	SS6		Depth (m) :	3.0-3.7
Sample Description :		% Silt and Clay	48	% Sand	38	% Gravel	14	Figure : 9
Sample Description :		Silty Sand (SM)						

Appendix A: Rock Core Photos

DRY BEDROCK CORES



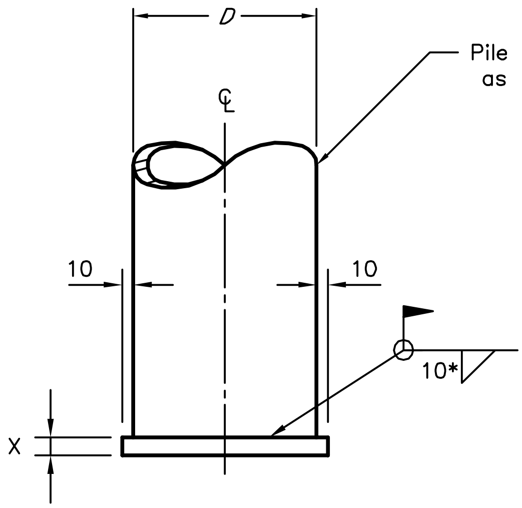
WET BEDROCK CORES



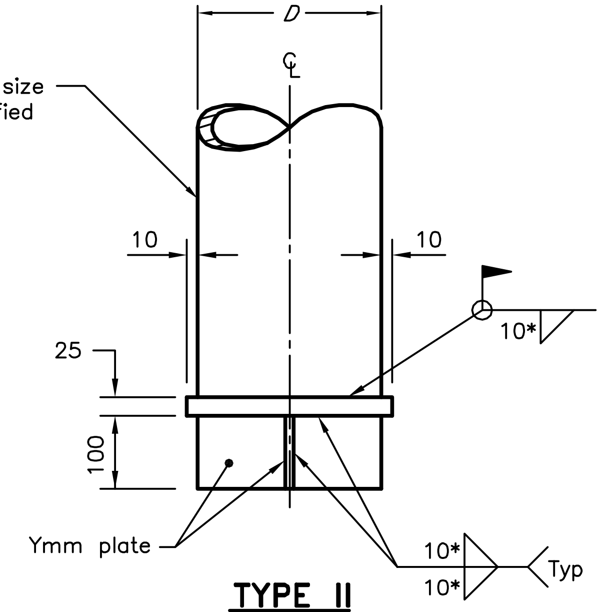
EXP Services Inc. www.exp.com
t: +1.613.688.1899 | f: +1.613.225.7337
2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

borehole no.	Depths:	project	project no.
BH4	Run 1: 5.8 m - 6.9 m Run 2: 6.9 m - 7.9 m Run 3: 7.9 m - 8.9 m Run 4: 8.9 m - 9.8 m	Geotechnical Investigation 178 Nepean Street and 219/223 Bank Street, Ottawa, ON	OTT-22018926-A0
date cored Dec 06, 2022		Rock Core Photographs	FIG A-1

Appendix B: Ontario Provincial Standard Drawing (OPSD) 3001.100, Type II



TYPE I

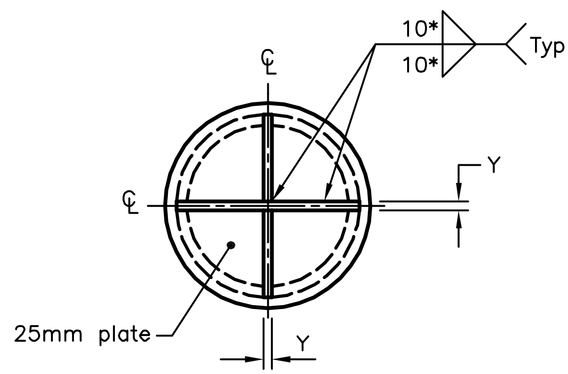


TYPE II

SHOE DETAILS

(*) or tube wall thickness whichever is smaller.

	Plate Thickness	
Pipe Diameter (mm)	X (mm)	Y (mm)
D < 324	25	12
324 ≤ D ≤ 406	40	15
406 < D ≤ 610	50	25



BOTTOM VIEW

NOTES:

A Driving shoe Type I or II as specified.

B Welding shall be according to CSA W59.

C Steel plates shall be according to CSA G40.20/G40.21, Grade 300W/350W.

D All dimensions are in millimeters unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2017	Rev 2
FOUNDATION PILES STEEL TUBE PILE DRIVING SHOE	OPSD 3001.100	

Appendix C: Laboratory Certificate of Analysis Report

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:	Matthew Zammit
PROJECT:	OTT-22018926-AO
AGAT WORK ORDER:	22Z978681
SOIL ANALYSIS REVIEWED BY:	Nivine Basily, Inorganics Report Writer
DATE REPORTED:	Dec 15, 2022
PAGES (INCLUDING COVER):	5
VERSION*:	1

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

Notes

Disclaimer:

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

Certificate of Analysis

AGAT WORK ORDER: 22Z978681
PROJECT: OTT-22018926-AO

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

CLIENT NAME: EXP SERVICES INC | **ATTENTION TO: Matthew Zammit**
SAMPLING SITE: | **SAMPLED BY:EXP**

Inorganic Chemistry (Soil)

DATE RECEIVED: 2022-12-08 | **DATE REPORTED: 2022-12-15**

Parameter	Unit	G / S	RDL	4602231
			SAMPLE DESCRIPTION:	BH#2 SS5 10'-12'
			SAMPLE TYPE:	Soil
			DATE SAMPLED:	2022-11-23
Chloride (2:1)	µg/g		2	82
Sulphate (2:1)	µg/g		2	262
pH (2:1)	pH Units		NA	8.37
Resistivity (2:1) (Calculated)	ohm.cm		1	1330

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

4602231 Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).

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

Resistivity is a calculated parameter.

Analysis performed at AGAT Toronto (unless marked by *)

CHARTERED NIVINE BASILY CHEMIST ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO

Certified By:

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

Quality Assurance

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-22018926-AO
SAMPLING SITE:

AGAT WORK ORDER: 22Z978681
ATTENTION TO: Matthew Zammit
SAMPLED BY:EXP

Soil Analysis

RPT Date: Dec 15, 2022			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
Inorganic Chemistry (Soil)															
Chloride (2:1)	4602231	4602231	82	78	5.0%	< 2	97%	70%	130%	100%	80%	120%	115%	70%	130%
Sulphate (2:1)	4602231	4602231	262	263	0.4%	< 2	104%	70%	130%	101%	80%	120%	NA	70%	130%
pH (2:1)	4602231	4602231	8.37	8.27	1.2%	NA	95%	80%	120%						

Comments: NA signifies Not Applicable.

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

CHARTERED NIVINE BASILY CHEMIST
ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO

Certified By:

AGAT Laboratories

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http://www.agatlabs.com

Method Summary

CLIENT NAME: EXP SERVICES INC | **AGAT WORK ORDER: 22Z978681**

PROJECT: OTT-22018926-AO | **ATTENTION TO: Matthew Zammit**

SAMPLING SITE: | **SAMPLED BY:EXP**

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

AGAT Laboratories

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

Laboratory Use Only

Work Order #: 22Z978681

Cooler Quantity: n/a - bag - no ice/packs

Arrival Temperatures: 21.1 | 21.0 | 21.1
L7 6.9 | 6.7 |

Custody Seal Intact: ☐Yes ☐No ☐N/A

Notes: Bagged Ice

Chain of Custody Record

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

Report Information:

Company: EXP Services Inc. Ottawa
Contact: Matthew Zammit
Address: 2650 Queensview Dr. Unit 100
Ottawa, ON, K2B 8H6
Phone: 613-688-1891 Fax:
Reports to be sent to:
1. Email: matthew.zammit@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-22018926-A0
Site Location: 178 Nepean St., Ottawa
Sampled By: EXP
AGAT Quote #: PO:
Please note: If quotation number is not provided, client will be billed full price for analysis.

Sample Matrix Legend

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

Invoice Information:

Bill To Same: Yes ☑ No ☐

Company:
Contact:
Address:
Email:

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI, DOC (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	chlorides	resistivity		Potentially Hazardous or High Concentration (Y/N)
BH #2 SS5 10-12'	Nov.23	AM PM	1	S														✓	✓	✓	✓		
		AM PM																					
		AM PM																					
		AM PM																					
		AM PM																					
		AM PM																					
		AM PM																					
		AM PM																					
		AM PM																					
		AM PM																					
		AM PM																					

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Jeff MacMillan	Dec. 8/22	1:38 PM	C. Griffiths	DEC 0 8 2022	13h38
C. Griffiths	DEC 0 8 2022	16h00	Sanad Assayed	Dec 9/22	8:35 am

Page 1 of 1

Nº: T 133517

Legal Notification

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