- **Golpro Holdings Inc.**

Geotechnical Investigation

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Final

Project Name:
Proposed Commercial and Residential Development
1356 Clyde Avenue
Ottawa, Ontario

Project Number:
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Date Submitted
March 24, 2020

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Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed Phase One and Phase Two commercial and high-density residential development to be located at 1356 Clyde Avenue, Ottawa, Ontario. Authorization to proceed with this geotechnical investigation was provided by Purchase Order No. CL1216-487 dated December 18, 2019 and issued by Golpro Holdings Inc.

This geotechnical investigation was undertaken concurrently with a Phase One Environmental Site Assessment (ESA) conducted by EXP with the results reported under separate cover.

Design information regarding the proposed development was provided in the architectural drawings prepared by RLA Architecture in Sheet Nos. 01 to 16 (RLA Project No.: 1941) dated March 11 and 12, 2020.

Current plan calls for the development of the site with two (2) low rise (6 storeys) and two (2) high rise (24 and 28 storeys) tower buildings with a three (3) level underground parking garage beneath the buildings. There will be paved outdoor parking lots and access roads in addition to landscaped areas. It is our understanding that the lowest floor slab of the parking garage will be at an approximate 10.5 m depth below existing grade. Information regarding site grade raise was not available at the time of this geotechnical investigation.

The fieldwork for the geotechnical investigation was undertaken on February 4 and 5, 2020 and consists of the drilling of seven (7) boreholes (BH Nos. 1 to 7) to termination and auger refusal depths ranging from 0.8 m to 9.2 m below existing grade. A 32 mm diameter monitoring well with a screened section was installed in selected boreholes for long-term monitoring of the groundwater levels and for future sampling of the groundwater, if required. The fieldwork was monitored on a full-time basis by EXP.

The borehole information indicates the subsurface conditions at the site consist of fill, silty clay, glacial till underlain by limestone bedrock contacted at 0.8 m to 6.3 m depths (Elevation 96.6 m to 90.6 m). The groundwater level ranges from 2.6 m to 3.4 m depths (Elevation 93.9 m to 93.0 m).

The recommendations made in this report are based on the assumption that the site can be permanently dewatered without impacting the neighboring structures and underground services. It is recommended that once the final design details for the development are available, the impact permanent dewatering of the site will have on neighboring structures and underground services should be determined.

Based on the lowest floor slab level of the parking garage at a 10.5 m depth below existing grade and a review of the borehole information, the proposed buildings may be supported by conventional spread and strip footings founded on the sound limestone bedrock below the 10.5 m floor slab depth. The results of the seismic shear wave survey completed at the site indicates that the average shear wave velocity (Vs) below the 10.5 m lowest floor slab depth where the footings will be located is 2100 m/s which will classify the site as **Class A** for seismic site response in accordance with Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC). Therefore, for footings of the proposed building founded below the lowest floor slab depth of 10.5

m, the site classification for seismic response is Class A. The subsurface soils are not considered to be susceptible to liquefaction during a seismic event.

A restriction in the grade raise at this site is not applicable from a geotechnical perspective since all of the compressible and settlement sensitive clayey soil will be excavated down to the bedrock and removed from the site for the construction of the proposed development. The excavated area will be backfilled with granular fill.

Spread and strip footings founded on the competent sound limestone bedrock, free of soil filled seams and below the weathered and fractured zones may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 4500 kPa. 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.

The lowest level floor slab in the underground parking garage may be designed as a slab-on-grade with perimeter and underfloor drainage systems. The floor slab should be set on a bed of well packed 19 mm clear stone at least 200 mm thick placed on an engineered fill pad at least 300 mm thick placed on the sound sandstone bedrock. The lowest level floor slab should be equipped with underfloor drains.

The subsurface walls should be designed to resist lateral static earth pressure as well as lateral dynamic earth pressure during a seismic event. The subsurface walls should be equipped with a perimeter drainage system.

Excavation of the overburden soils may be undertaken using large heavy mechanical equipment capable of removing cobbles and boulders within the fill and glacial till. Excavation of the underlying bedrock will require line drilling and blasting techniques and should be undertaken by a specialized contractor. Pre-condition survey of surrounding buildings and infrastructure (such as roadways and underground services) should be undertaken at the site prior to start of construction as well as conducting vibration monitoring during blasting and rock excavation and construction operations.

Excavations within the soils for the proposed development will likely have to be undertaken within the confines of a shoring system that may consist of steel H soldier pile and timber lagging, steel interlocking sheeting and/or secant pile shoring system. The shoring system may be tied back by anchors grouted into the sound bedrock.

Excavations within the bedrock may be undertaken with near vertical sides subject to review by a geotechnical engineer. The rock face may require support in the form of rock bolts to maintain the integrity of the rock face. In the upper weathered/fractured zones of the bedrock, rock bolts in combination with wire mesh system and/or shotcrete may be required. This will be best established on-site during excavation.

Surface water and groundwater in open cut excavations as well as in shored excavations 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 anticipated that all fill required for backfilling purposes will have to be imported to the site and conform to the Ontario Provincial Standard Specification (OPSS) requirements for Granular A and B Type II.

Due to the variable depth of the bedrock surface throughout the site, it is recommended that prior to tendering, additional rock probes should be undertaken to collect additional data on the depth to bedrock.

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

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1 Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed Phase One and Phase Two commercial and high-density residential development to be located at the site registered by the street address of 1356 Clyde Avenue, Ottawa, Ontario. Authorization to proceed with this geotechnical investigation was provided by Purchase Order No. CL1216-487 dated December 18, 2019 and issued by Golpro Holdings Inc.

This geotechnical investigation was undertaken concurrently with a Phase One Environmental Site Assessment (ESA) conducted by EXP with the results reported under separate cover.

Design information regarding the proposed development was provided in the architectural drawings prepared by RLA Architecture in Sheet Nos. 01 to 16 (RLA Project No.: 1941) dated March 11 and 12, 2020.

The drawings indicate the proposed development will consist of two (2) low rise (6 storeys) and two (2) high rise (24 and 28 storeys) tower buildings with a three (3) level underground parking garage beneath the buildings. There will be paved outdoor parking lots and access roads in addition to landscaped areas. It is our understanding that the lowest floor slab of the parking garage will be at an approximate 10.5 m depth below existing grade. Information regarding site grade raise was not available at the time of this geotechnical investigation.

The geotechnical investigation was undertaken to:

a) Establish the subsurface soil, bedrock and groundwater conditions at the seven (7) boreholes located on the site;

b) Classify the site for seismic site response in accordance with the requirements of the 2012 Ontario Building Code (OBC) 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 foundation walls of the proposed buildings;

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

h) Provide pipe bedding requirements for proposed underground municipal services;

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

j) Recommend pavement structure thicknesses for paved surface parking lots and access roads; and

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

The comments and recommendations given in this report are 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 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 subject site is located in the northwest corner of the Clyde Avenue and Baseline Road intersection in Ottawa, Ontario. At the time of this geotechnical investigation, the site was occupied by two (2) multi-unit single storey slab-on-grade commercial buildings with outdoor paved parking lots and access roads. The location of the site is shown in Figure 1.

The ground surface elevations at the boreholes located on the site range from Elevation 97.4 m to 96.4 m indicating the topography of the site gradually slopes downward in an easterly direction.

3 Geology of Site

3.1 Surficial Geology

The surficial geology map (Map 1506A – Surficial Geology, Ontario-Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1982) indicates that the site is located in an area of two (2) different soil types; glacial till plain (with relief less than 5 m) in the east portion of the site and offshore marine deposits consisting of clay, silty clay and silt in the western portion of the site.

3.2 Bedrock Geology

The bedrock geology map (Map 1508A – Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1979) indicates the site is underlain by limestone with some shaly partings of the Ottawa formation.

4 Investigation Procedure

4.1 Fieldwork

The fieldwork for the geotechnical investigation was undertaken on February 4 and 5, 2020 and consists of the drilling of seven (7) boreholes (BH Nos. 1 to 7) to termination and auger refusal depths ranging from 0.8 m to 9.2 m below existing grade. The fieldwork was monitored on a full-time basis by EXP. The borehole locations and existing buildings on site are shown in Figure 2. The borehole locations and the proposed building development are shown in Figure 3.

The borehole locations and geodetic elevations were established in the field by a survey crew from EXP and their locations cleared from any underground services by USL-1 cable locators.

The boreholes were drilled with a CME-55 rubber track-mounted drill rig equipped with continuous flight hollow-stem auger equipment and rock coring capabilities. Standard penetration tests (SPTs) was performed in all the boreholes on a continuous basis and at 0.6 m to 1.5 m depth intervals. The soil samples were retrieved by the split-barrel sampler, in accordance with the American Society for Testing and Materials (ASTM). Auger samples were obtained in the upper levels of some of the boreholes. The undrained shear strength of the clayey soil was measured in situ by conducting the vane test at selected depth intervals within the clayey soil. The undrained shear strength was also measured in some of the recovered samples of the clayey soil using a penetrometer. The presence of the bedrock was proven in selected boreholes by conventional coring techniques using NQ-size core barrel. A record of wash water return, colour of wash and any sudden drop of the drill rods were kept during rock coring operations.

A 32 mm diameter monitoring well with a screened section was installed in selected boreholes for long-term monitoring of the groundwater levels and for future sampling of the groundwater, if required. The installation configuration of each monitoring well is documented on the respective borehole log. All boreholes were backfilled upon completion of drilling and sampling operations.

4.2 Laboratory Testing Program

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

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

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

Table I: Summary of Laboratory Testing Program

Type of Test	Number of Tests Completed
Soil Samples	
Moisture Content Determination	21
Unit Weight Determination	4
Grain Size Analysis	3
Atterberg Limit Determination	1
Corrosion Analysis (pH, sulphate, chloride and resistivity)	1
Bedrock Cores	
Unit Weight Determination	8
Unconfined Compressive Strength Test	8
Corrosion Analysis (pH, sulphate, chloride and electrical resistivity)	2

4.3 Seismic Shear Wave Survey

A seismic shear wave survey was conducted on site on March 10, 2020 by Geophysics (GPR) International Inc. The purpose of the survey is to determine the seismic shear wave velocity and the site classification for seismic site response. The procedure for the survey and the survey results are shown in Appendix B.

5 Subsurface Conditions

A detailed description of the subsurface conditions and groundwater levels from the boreholes are given on the attached Borehole Logs, Figure Nos. 4 to 10 inclusive. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

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

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

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

5.1 Pavement Structure

All seven (7) boreholes are located within paved areas. The pavement structure consists of 40 mm to 50 mm thick asphaltic concrete underlain by 300 mm to 1000 mm thick granular fill base. The granular fill samples did not have stains and odors. Based on the standard penetration test (SPT) N-values of 60 to 79, the granular fill base is in a very dense state. The high N-values may be attributed to portions of the granular fill base being frozen. The moisture content of the granular fill base ranges from 4 percent to 12 percent.

5.2 Fill

The pavement structure in Borehole Nos. 1, 2 and 6 is underlain by fill that extends to 1.4 m and 2.3 m depths (Elevation 95.5 m and 94.1 m). The fill consists of silty sand with gravel to a mixture of silty sand and silty clay. The soil samples of the fill did not have stains and odors. The fill is in a compact state based on standard penetration test (SPT) N-values of 12 to 27. The moisture content of the fill is 5 percent to 21 percent.

Grain size analysis of one (1) sample of the fill was conducted and the results are summarized in Table II. The grain size distribution curve is shown in Figure 11.

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

Borehole No. - Sample No.	Depth (m)	Grain-size Analysis (%)			Soil Classification (USCS)
		Gravel	Sand	Fines (Silt and Clay)	
BH 2 – SS2	0.8 – 1.4	6	73	21	FILL: Silty Sand (SM)

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

5.3 Silty Clay

The fill in Borehole Nos. 1 and 2 is underlain by silty clay that extends to 3.9 m depth (Elevation 93.0 m). The silty clay samples did not have stains and odors. The results from penetrometer and in-situ vane tests indicate the undrained shear strength of the silty clay ranges from 77 kPa to 192 kPa, suggesting the consistency of the silty clay is stiff to very stiff. The sensitivity of the silty clay is 3.8 to 5.3, indicating the silty clay is of a medium sensitivity to sensitive. The natural moisture content and unit weight of the silty clay is 29 percent to 44 percent and 18.0 kN/m^3 to 19.3 kN/m^3, respectively.

Grain size analysis and Atterberg limit determination tests were conduced on two (2) samples of the silty clay and the grain size distribution curve is shown in Figure 12 and the test results are summarized in Table III.

Table III: Summary of Lab Test Results – Silty Clay Samples

Borehole/ - Sample No.	Depth (m)	Moisture Content (%)	Grain-size Analysis (%)			Atterberg Limit (%)		
			Gravel	Sand	Fines	LL	PL	PI
BH1 – SS4	2.3 – 2.9	33	0	6	94	-	-	-
BH2 – SS4	2.3 – 2.9	44	-	-	-	33	17	16

LL: Liquid Limit; **PL:** Plastic Limit; **PI**: Plasticity Index

Based on a review of the test results, the soil may be classified as a silty clay of low plasticity (CL) in accordance with the USCS.

5.4 Glacial Till

Glacial till was contacted beneath the silty clay in Borehole Nos. 1 and 2 at a 3.9 m depth (Elevation 93.0 m). The glacial till extends to a 6.3 m depth (Elevation 90.6 m) in Borehole No. 3. The glacial till consists of a sand with gravel, cobbles and boulders. The glacial till samples did not have stains and odors. Based on the SPT N-values of 7 to 17, the glacial till is in a loose to compact state. In Borehole No. 1 the SPT N-value at a 4.8 m depth is 50 followed by refusal. The high N-value and refusal may be attributed to the sampler making contact with a boulder. The natural moisture content of the glacial till ranges from 11 percent to 44 percent.

Grain size analysis on one (1) sample of the glacial till was conducted and the results are summarized in Table IV. The grain size distribution curves are shown in Figures 13.

Table IV: Summary of Results from Grain-size Analysis – Glacial Till Sample

Borehole No. - Sample No.	**Depth (m)**	**Grain-size Analysis (%)**			**Soil Classification (USCS)**
		Gravel	**Sand**	**Fines (Silt and Clay)**	
BH2 – SS7	4.6 – 5.2	19	37	44	Silty Clayey Sand with Gravel (SC-SM)

Based on a review of the results from the grain size analysis, the glacial till may be classified as a silty clayey sand with gravel (SC-SM) in accordance with the USCS. The glacial till contains cobbles and boulders.

5.5 Inferred Boulders or Weathered Bedrock

The fill in Borehole Nos. 3 to 7 is underlain by a layer of inferred boulders or weathered bedrock contacted at 0.4 m to 2.3 m depths (Elevation 96.8 m to 94.1 m). This layer extends to depths ranging from 0.8 m to 2.5 m (Elevation 96.6 m to 93.9 m). This layer is inferred since recovery of samples from the split spoon sampler was insufficient to describe the material. The natural moisture content of the inferred boulders or weathered bedrock from one (1) auger sample is 6 percent.

5.6 Limestone Bedrock

Auger refusal was met in Borehole Nos. 1, 4 and 5 at 0.8 m to 1.6 m depths (Elevation 95.6 m to 91.7 m) on inferred boulders or bedrock and in Borehole Nos. 2, 3, 6 and 7 at 0.8 m to 6.3 m depths (Elevation 96.6 m to 90.6 m). Conventional core drilling techniques were used to advance Borehole Nos. 2, 3, 6 and 7 beyond the auger refusal depths to termination depths of 8.6 m to 9.2 m depths (Elevation 88.3 m to 87.2 m) confirming that auger refusal in these boreholes was met on limestone bedrock. Photographs of the bedrock cores are shown in Appendix C.

A summary of the auger refusal depth (elevation) on inferred boulders or limestone bedrock and the bedrock depths (elevations) is shown in Table V.

Table V: Summary of Bedrock Depths and Elevations in Boreholes

Borehole No.	Ground Surface Elevation (m)	Inferred Bedrock Depth/Bedrock Depth (Elevation) (m)	Inferred Bedrock Elevation/Bedrock Elevation (m)	Bedrock Proven by Coring of Bedrock
1	96.93	5.2	91.7	No (inferred)
2	96.94	6.3	90.6	Yes to 8.6 m depth (Elevation 88.3 m)
3	97.36	0.8	96.6	Yes to 9.2 m depth (Elevation 88.2 m)
4	96.80	1.6	95.2	No (inferred)
5	96.36	0.8	95.6	No (inferred)
6	96.41	2.5	93.9	Yes to 9.2 m depth (Elevation 87.2 m)
7	96.40	1.0	95.4	Yes to 9.2 m depth (Elevation 87.2 m)

A review of the above table indicates that the depth to the bedrock surface ranges from approximately 0.8 m to 6.3 m below existing grade (Elevation 96.6 m to 90.6 m) which indicates the bedrock surface dips in the northerly direction. The depth to the bedrock surface is quite variable throughout the site.

The Total Core Recovery (TCR) of the bedrock is 100 percent. The Rock Quality Designation (RQD) ranges from 42 percent to 98 percent indicating the bedrock is of a fair to excellent quality. Locally in the upper 600 mm of the bedrock in Borehole No. 7, the RQD value is 0 percent indicating a zone of very poor quality of rock; likely due to the high frequency of fractures.

Unit weight determination and unconfined compressive strength tests were conducted on eight (8) rock core sections and the results are summarized in Table VI. A review of the test results indicates the strength of the rock may be classified as very strong in accordance with the Canadian Foundation Engineering Manual (CFEM), Fourth Edition, 2006.

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

Borehole No.-Run No.	Depth (m)	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)	Classification of Rock with respect to Compressive Strength (1)
2	6.4 – 6.6	26.4	143.6	Very Strong
2	8.1 – 8.3	26.6	155.1	Very Strong
3	1.2- 1.4	26.3	113.0	Very Strong
3	4.7 – 4.9	26.6	116.8	Very Strong
6	3.2 – 3.4	26.8	160.4	Very Strong
6	8.8 – 9.0	26.8	127.3	Very Strong
7	5.2 – 5.4	26.9	132.8	Very Strong
7	7.7 – 7.9	26.8	144.4	Very Strong

Note:
(1) Reference: Fourth Edition – Canadian Foundation Engineering Manual (2006)

5.7 Groundwater Level Measurements

A summary of the groundwater level measurements taken on February 18 and 25, 2020 in the monitoring wells installed in Borehole Nos. 1, 3, 6 and 7 is shown in Table VII.

Table VII: Summary of Groundwater Level Measurements

Borehole No. (BH)	Ground Surface Elevation (m)	Date of Measurement	Elapsed Days after Drilling	Groundwater Depth (m)	Groundwater Elevation (m)
1	96.93	February 18, 2020	14 days	3.2	93.7
1	96.93	February 25,2020	21 days	3.2	93.7
3	97.36	February 18, 2020	13 days	Not Accessible Due to Parked Vehicle	N/A
3	97.36	February 25, 2020	20 days	3.4	93.9
6	96.41	February 18, 2020	13 days	3.4	93.0
6	96.41	February 25, 2020	20 days	3.4	93.0
7	96.40	February 18, 2020	13 days	2.8	93.6
7	96.40	February 25, 2020	20 days	2.6	93.8

A review of Table VII indicates groundwater levels at the site vary between 2.6 m and 3.4 m depths (Elevation 93.9 m to 93.0 m). The groundwater level measurements indicate the groundwater flow is anticipated to be in a southerly direction.

Water levels were determined in the boreholes at the times and under the conditions stated in the scope of services. 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 Design Considerations

The recommendations made in this report are based on the assumption that the site can be permanently dewatered without impacting the neighboring structures and underground services. It is recommended that once the final design details for the development are available, the impact permanent dewatering of the site will have on neighboring structures and underground services should be determined.

7 Site Classification for Seismic Site Response and Liquefaction Potential of Soils

7.1 Site Classification for Seismic Site Response

Based on the lowest floor slab level of the parking garage at a 10.5 m depth below existing grade and a review of the borehole information, the proposed buildings will be supported by conventional spread and strip footings founded on the sound limestone bedrock below the 10.5 m floor slab depth.

The results of the seismic shear wave survey are presented in the report in Appendix B. The results of the seismic shear wave survey completed at the site indicates that the average shear wave velocity (Vs) below the 10.5 m lowest floor slab depth where the footings will be located is 2100 m/s and therefore the site class can be classified as Class A for seismic site response as per Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC).

7.2 Liquefaction Potential of Soils

The on-site soils are not considered to be susceptible to liquefaction during a seismic event. However, it is expected that all subsurface soils will be excavated /removed as part of the proposed construction and therefore, there is no concern with liquefaction potential of subsurface soils at the site.

8 Grade Raise Restrictions

A restriction in the grade raise at this site is not applicable from a geotechnical perspective since all of the compressible and settlement sensitive clayey soil will be excavated down to the bedrock and removed from the site for the construction of the proposed development. The excavated area will be backfilled with granular fill.

9 Foundation Considerations

The borehole information indicates the subsurface conditions at the site consist of fill, silty clay, glacial till underlain by limestone bedrock contacted at 0.8 m to 6.3 m depths (Elevation 96.6 m to 90.6 m). The groundwater level ranges from 2.6 m to 3.4 m depths (Elevation 93.9 m to 93.0 m).

Since the lowest floor slab of the parking garage will be at a 10.5 m depth below existing grade, the proposed buildings may be supported by spread and strip footings designed to bear on the competent sound limestone bedrock free of soil filled seams and below any weathered and fractured zones. It has been assumed that the overburden soils will be entirely removed from the building envelopes down to the bedrock.

Spread and strip footings founded on the competent sound limestone bedrock, free of soil filled seams and below the weathered and fractured zones, may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 4500 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.

All the footing beds should be reviewed by a geotechnical engineer to ensure that the bedrock subgrade 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. Any sub-excavation which extends below the underside of the footings would have to be raised using 15 MPa lean mix concrete. Also, if the surface of the excavated bedrock is not level, the bedrock surface may be levelled by the placement of concrete.
The resistance to sliding of the building footings will be provided by friction between the footing concrete and the sound sandstone bedrock. The unfactored ULS coefficient of friction is 0.70.

A minimum of 1.5 m of earth cover should be provided to exterior footings of heated structures to protect them from damage due to frost penetration. The frost cover should be increased to 2.1 m for unheated structures. For this project it is anticipated that the required earth cover for the footings of the proposed buildings will be satisfied, since the footings are anticipated to be at depths greater than 1.5 m below the final grade.

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

10 Floor Slab Construction and Drainage Requirements

The lowest floor slab of the parking garage may be designed as a slab-on-grade. The slab-on-grade should be set on a bed of well packed 19 m clear stone at least 200 mm thick placed on an engineered fill pad at least 300 mm thick placed on the sound sandstone bedrock. The required engineered fill pad should comprise of Ontario Provincial Standard Specification (OPSS)1010 Granular B Type II placed on top of the bedrock in 300 mm thick lifts and each lift compacted to 98 percent standard Proctor maximum dry density (SPMDD). The clear stone will prevent the capillary rise of moisture from the underlying soil to the floor slab. Adequate saw cuts should be provided in the floor slab to control cracking.

It is recommended that perimeter as well as underfloor drains should be provided for the slab-on-grades of the parking garage. The underfloor drainage system may consist of 100 mm diameter perforated pipe or equivalent placed in parallel rows at 5 m to 6 m centres and at least 300 mm below the underside of the floor slab. The drains should be set on 100 mm 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 so that at least one system would be operational should the other fail.

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

11 Lateral Earth Pressure Against Subsurface Walls

If the space between the subsurface walls and the rock face is to be backfilled, the subsurface walls 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. 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 equations (i) and (ii).

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

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

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

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

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:

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

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

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. A schematic of the location of the drainage board is shown in Figure 14. 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, as shown in Figure 14. 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.

12 Excavations and De-Watering Requirements

Excavations for the proposed development are expected to extend to an approximate 11.0 m depth below existing grade with excavation depths in the soil to approximately 6.5 m and to an approximate 10.0 m depth below the bedrock surface. The excavations will extend to approximately 8.5 m below the groundwater level.

12.1 Overburden Soil Excavation

The excavations in the soil may be undertaken using large heavy mechanical equipment capable of removing debris within the fill as well as cobbles and boulders within the fill and underlying 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. The excavation side slopes below the groundwater level are anticipated to slough and should be cut back a 2H:1V. It is anticipated that due to the significant depth of the excavation and the proximity of the excavation to existing buildings and infrastructure, the excavations will likely have to be undertaken within the confines of a shoring system. The shoring system may consist of steel H soldier pile and timber lagging system, interlocking sheeting system and/or secant pile shoring system.

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

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

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

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

12.1.1 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 21 kN/m^3
h = the depth, in metres, at which pressure, P, is being computed
q = the equivalent surcharge acting on the ground surface adjacent to the shoring system

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

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

12.1.2 Secant Pile Shoring System

Groundwater cut-off shoring systems such as steel interlocking sheeting and 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 = estimated unit weight of the soil to be retained = 21 kN/m^3

γ' = submerged unit weight of soil = 11 kN/m^3

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

If the secant walls are incorporated into the design of the structures, they should also be designed to resist soil dynamic thrust and hydrodynamic thrust due to a seismic event.

Secant pile walls consist of overlapping concrete piles that form a stronger watertight barrier compared with the sheeting system. 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.

12.1.3 Additional Comments

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.

12.1.4 Rock Anchors

The shoring system will require lateral restraint by tiebacks in the form of temporary grouted rock anchors. It is noted that if permission is not received from adjacent property owners for the installation of tiebacks, the shoring system may have to be supported by cross bracing or the use of rakers on the inside of the excavation.

Grouted rock anchors may fail in one or more of the following manners:

a) Failure of the grout/tendon bond;
b) Failure of the steel tendon or top anchorage;
c) Failure of the rock/grout bond; or
d) Failure of the rock mass.

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

Failure of the rock/grout bond:

- The unfactored ultimate limit state (ULS) bond strength of the sound rock/grout interface may be taken as 1500 kPa (1.5 MPa). For semi-empirical analysis, the factored ULS bond strength is 450 kPa, using a resistance factor of 0.3. The factored ULS bond strength may be taken as 600 kPa and includes a resistance factor of 0.4 based on conducting proof test on all anchors. The unconfined compressive strength of the grout is assumed to be 30 MPa.

Failure of the rock mass:

- A 90-degree apex (45 degrees from the vertical) should be used to calculate the rock volume using the theoretical rock cone. The apex is located at the middle of the bonded length.

- The submerged unit weight of the bedrock equal to 16.8 kN/m^3 should be used in the calculations.
- The weathered and highly fractured zones of the limestone bedrock should not be included in the bond length when calculating the anchor capacity. The recommended minimum unbonded length is 3.0 m and should include the weathered and highly fractured zone of the bedrock.
- The minimum bonded length is 3 m.
- For groups of rock anchors, the anchor group resistance to rock mass failure should be reduced to reflect the theoretical rock cone overlap.

Pre-production or design performance tests on selected rock anchors should be conducted in accordance with the 2006 CFEM and Ontario Provincial Standard Specification (OPSS) 942. Proof load tests should be conducted on all anchors and should be in accordance with the 2006 CFEM and OPSS 942.

If permanent grouted rock anchors are required to resist uplift forces from the proposed buildings, the design of the rock anchors should include examination of the above noted failure modes. The design, installation and testing of the permanent rock anchors should be in accordance with Ontario Provincial Standard Specification (OPSS) 942.

12.2 Rock Excavation

Excavation of the underlying bedrock will require line drilling and blasting techniques and should be undertaken by a specialized blasting contractor working under the direction of a blasting engineer.

12.2.1 Rock Support

Excavations within the bedrock may be undertaken with near vertical sides subject to review by a geotechnical engineer. The rock face may require support in the form of rock bolts to maintain the integrity of the rock face. In the upper weathered/fractured zones of the bedrock, rock bolts in combination with wire mesh system and/or shotcrete may be required. The excavation will have to be undertaken in a staged approach with the rock excavated in a pre-determined depth interval (for example every 3 m). The exposed rock face in each stage will have to be examined by a geotechnical engineer to determine the number of rock bolts required. The rock bolt system should be installed in this manner to the bottom of the excavation. The design of the rock bolts should be in accordance with the design of the rock anchors in Section 11.1.4 of this report.

12.2.2 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.

12.3 De-Watering Requirements

Surface water and groundwater in open cut excavations as well as in shored excavations 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 has been assumed that the excavation depth at the site will be approximately 11.0 m and would necessitate groundwater removal from the site. It is noteworthy to mention that new legislation came into force in Ontario on March 29, 2016 to regulate groundwater takings for construction dewatering purposes. Prior to March 29, 2016, a Category 2 Permit to Take Water (PTTW) was required from the Ontario Ministry of the Environment and Climate Change (MOECC) for groundwater takings related to construction dewatering, where taking volumes in excess of 50 m^3/day, but less than 400 m^3/day, and the taking duration was no more than 30 consecutive days. The new legislation replaces the Category 2 PTTW for construction dewatering with a new process under the Environmental Activity and Sector Registry (EASR). The EASR is an on-line registry, which allows persons engaged in prescribed activities, such as water takings, to register with the MOECC instead of applying for a PTTW.

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

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

13 Pipe Bedding Requirements

It is anticipated that the subgrade for the proposed underground municipal services will consist of imported granular material and limestone bedrock.

The pipe bedding for the underground municipal services should be in accordance with City of Ottawa specifications, drawings and special provisions. The bedding, pipe cover and backfill materials should be compacted to a minimum of 98 percent standard Proctor maximum dry density (SPMDD).

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

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

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

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

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

The soils to be excavated from the site will comprise of silty sand with gravel fill, silty clay, glacial till and limestone bedrock. From a geotechnical perspective, these soils and the limestone bedrock are not considered suitable for reuse as backfill material in the interior or exterior of the building and should be discarded.

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

- Engineered fill, underfloor fill including backfilling in service trenches inside the building - OPSS 1010 (as amended by SSP110S13) for Granular B Type II (50 mm minus) placed in 300 mm thick lifts with each lift compacted to 98 percent SPMDD beneath the floor slab;
- Backfill against exterior subsurface walls - OPSS 1010 Granular B Type II placed in 300 mm thick lifts and compacted to 95 percent SPMDD; and
- Trench backfill outside building area, and fill placement to subgrade level for pavement - OPSS Granular A and B Type II materials placed in 300 mm thick lifts compacted to minimum 95 percent SPMDD.

15 Access Roads and Parking Areas

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

Table VIII: Recommended Pavement Structure Thicknesses

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

SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698-12e2
MRD denotes Maximum Relative Density, ASTM D2041

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

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

(1) The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum cross fall of 2 percent) to provide effective surface drainage towards catch basins. Surface water should not be allowed to pond adjacent to the outside edges of paved areas.

(2) The granular materials used for pavement construction should conform to Ontario Provincial Standard Specifications (OPSS) for Granular A and Granular B Type II and should be compacted to 100 percent SPMDD. The asphaltic concrete and its placement should meet OPSS 1151 requirements. It should be placed and compacted to OPSS 311 and 313.

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

16 Corrosion Potential of Subsurface Soil and Bedrock

Chemical tests limited to pH, sulphate, chloride and resistivity determinations were undertaken on one (1) soil sample (silty clay) and two (2) sections of bedrock cores. A summary of the results is shown in Table IX. The laboratory certificate of analysis is shown in Appendix A.

Table IX: pH, Sulphate, Chloride and Resistivity Test Results on Soil Sample and Bedrock Cores

Borehole No. (BH); Sample/Run No. (SS)	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
Soil Sample					
BH 2: SS5	3.0 – 3.6	8.03	0.0358	0.1500	340
Bedrock Cores					
BH 2: Run 1	6.3 – 7.7	8.68	0.0051	0.0043	4310
BH 7: Run 6	7.6 – 9.2	8.70	0.0054	0.0049	4030

The sulphate test results indicate the soil sample and bedrock core sections have a negligible sulphate attack on subsurface concrete. The concrete should be in accordance with CSA A.23.1-14.

Based on a review of the resistivity test results, the silty clay sample is considered to be very corrosive to bare steel as per the National Association of Corrosion Engineers (NACE). The limestone bedrock core sections are considered to be mildly corrosive to bare steel as per NACE. Appropriate measures should be undertaken to protect buried steel elements from corrosion.

17 Additional Study

Due to the variable depth of the bedrock surface throughout the site, It is recommended that prior to tendering, additional rock probes should be undertaken to collect additional data regarding the depth to bedrock surface throughout the site.

The recommendations made in this report are based on the assumption that the site can be permanently dewatered without impacting the neighboring structures and underground services. It is recommended that once the final design details for the development are available, the impact permanent dewatering of the site will have on neighboring structures and underground services should be determined.

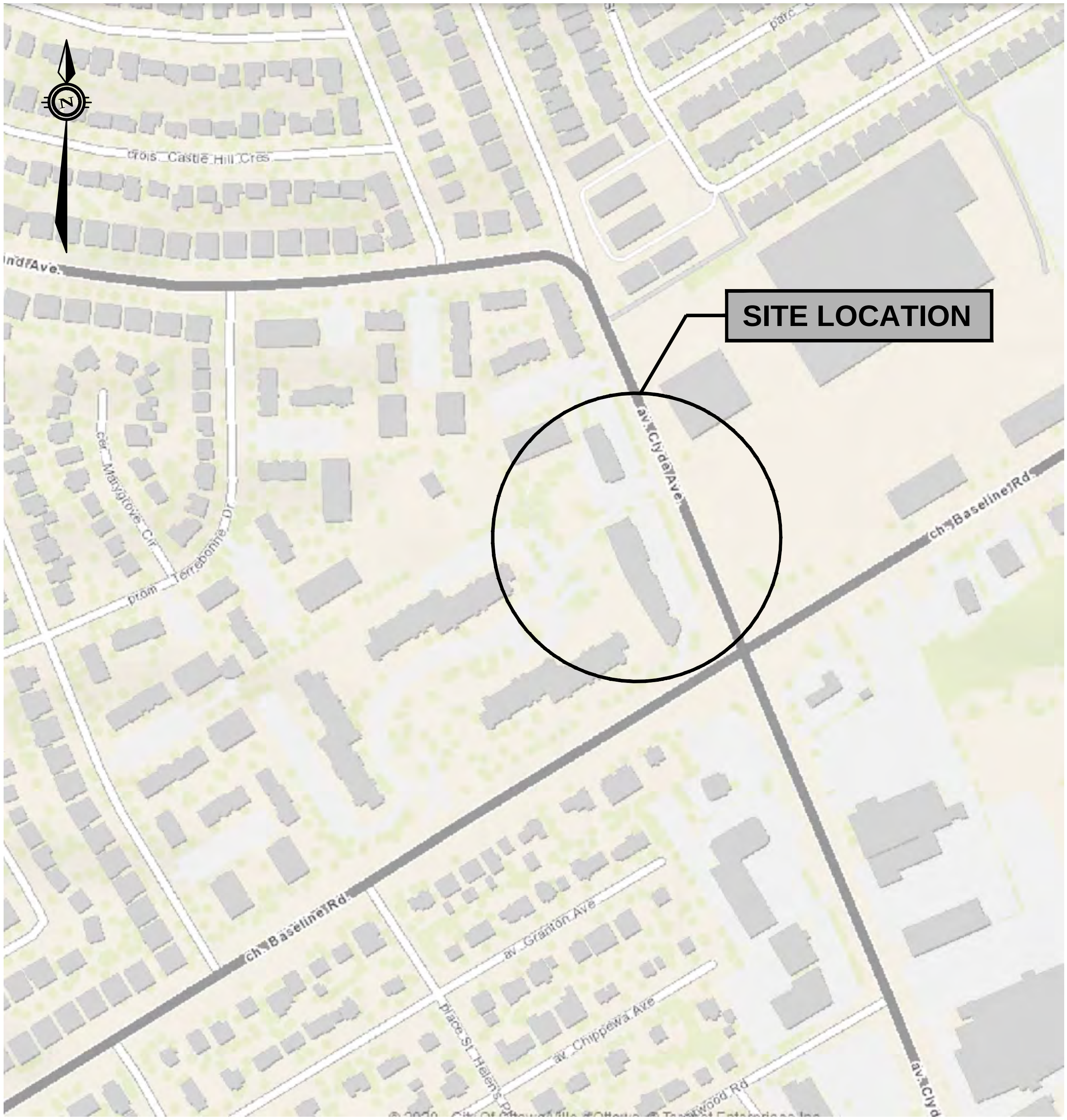
18 General Comments

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

The information contained in this report is not intended to reflect on environmental aspects of the soils. Reference is made to the Phase One Environmental Site Assessment completed by EXP and reported under separate cover.

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

Figures



BACKGROUND MAP DATA FROM CITY OF
OTTAWA © 2019, TERANET ENTERPRISES

0 40m 80m 160m
HORIZONTAL 1:4000

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scale AS SHOWN
date FEB. 2020
drawn by G.C.

CLIENT:
GOLPRO HOLDINGS INC.
PROP. COMMERCIAL & RESIDENTIAL DEVELOPMENT
1356 CLYDE AVENUE, OTTAWA, ONTARIO

TITLE:
SITE LOCATION PLAN

project no.
OTT-00257901-A0

FIG 1

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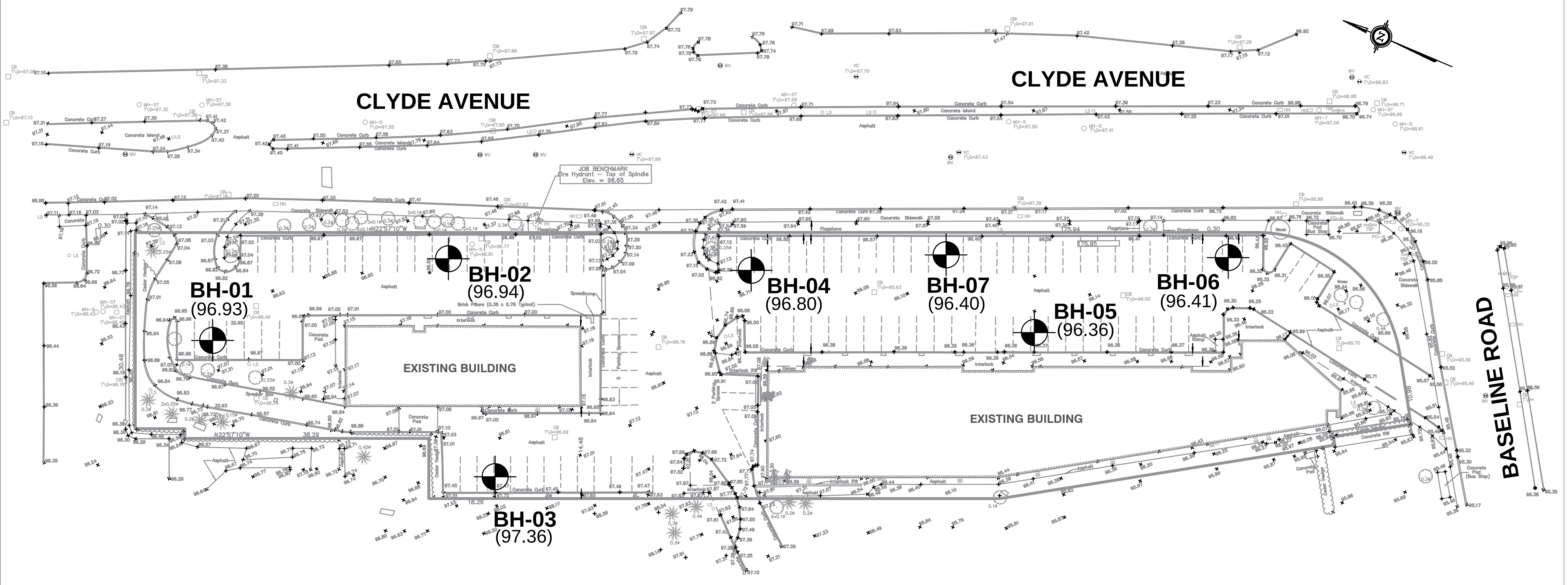

NOTE: 1. THIS DRAWING SHOWS THE BOREHOLE LOCATIONS AND EXISTING BUILDINGS ON SITE AS OF MARCH 2020.

0 10m 20m 40m
HORIZONTAL 1:1000

NOTES:
1. THE BOUNDARIES, SOIL AND ROCK TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND ROCK CORES WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
4. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
5. BASE PLAN OBTAINED FROM ANNIS, O'SULLIVAN, VOLLEBEKK LTD. ONTARIO LAND SURVEYORS, PROJECT NO. 17986-19, DATED JULY 02, 2019.

LEGEND

⊕ **BH-01** (96.93) BOREHOLE NUMBER, LOCATION AND GEODETIC ELEVATION IN METERS

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scale: AS SHOWN
date: MAR. 2020
drawn by: G.C.
CLIENT: GOLPRO HOLDINGS INC.
PROPOSED COMMERCIAL & RESIDENTIAL DEVELOPMENT
1356 CLYDE AVENUE, OTTAWA, ONTARIO
TITLE: BOREHOLE LOCATION PLAN
project no. OTT-00257901-A0
FIG 2

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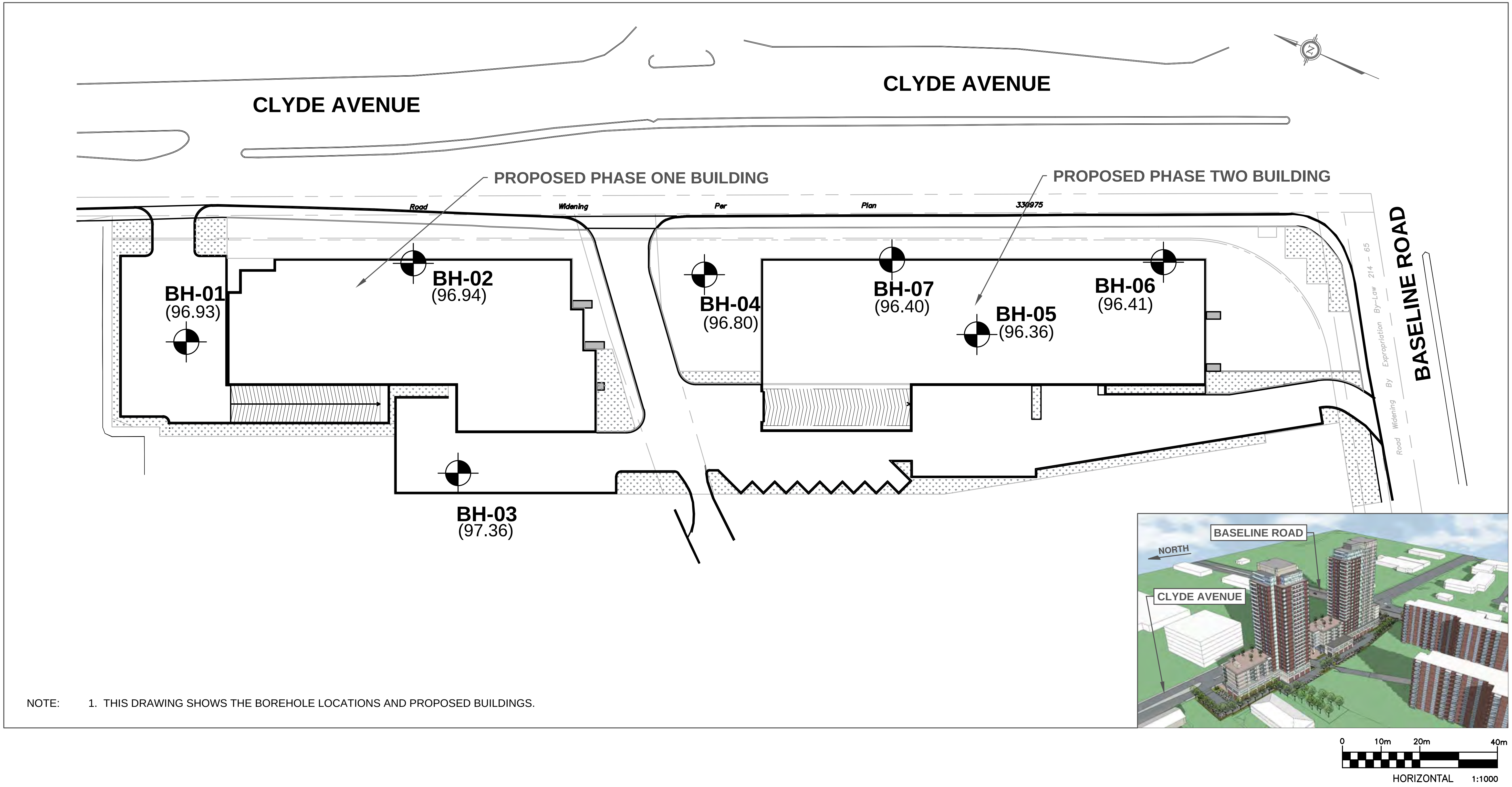

NOTE: 1. THIS DRAWING SHOWS THE BOREHOLE LOCATIONS AND PROPOSED BUILDINGS.

NOTES:
1. THE BOUNDARIES, SOIL AND ROCK TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND ROCK CORES WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
4. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
5. BASE PLAN OBTAINED FROM RLA ARCHITECTURE, PROJECT NO. 1941, DRAWING NO. 4, "GROUND FLOOR PLAN", DATED FEBRUARY 27, 2020

LEGEND

⊕ **BH-01** (96.93)	BOREHOLE NUMBER, LOCATION AND GEODETIC ELEVATION IN METERS

exp Services Inc.
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2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6
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• BUILDINGS • EARTH & ENVIRONMENT • ENERGY •
• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

scale AS SHOWN
date MAR. 2020
drawn by G.C.

CLIENT: GOLPRO HOLDINGS INC.
PROPOSED COMMERCIAL & RESIDENTIAL DEVELOPMENT
1356 CLYDE AVENUE, OTTAWA, ONTARIO

TITLE: BOREHOLE LOCATION PLAN

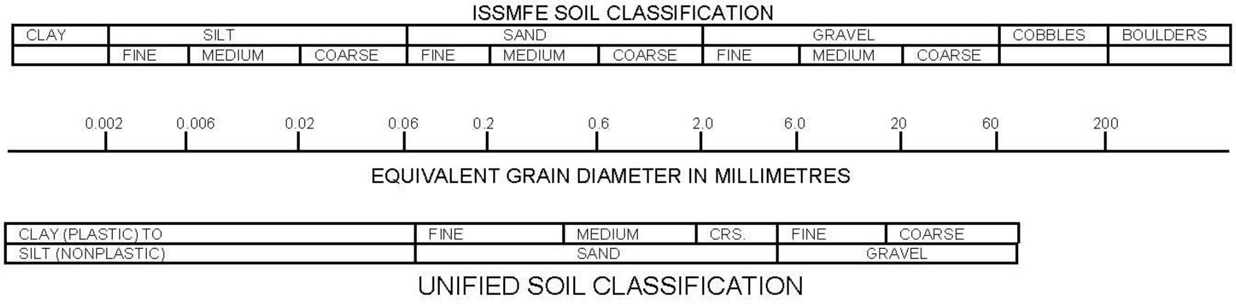
project no. OTT-00257901-A0

FIG 3

Filename: e:\ott\ott-00257901-a0\60 execution\65 drawings_geot\fig-2 bh location plan (1356 clyde).dwg
Last Saved: 3/6/2020 2:53:46 PM
Last Plotted: 3/6/2020 2:54:07 PM Plotted by: CuiG Pen Table:: exp-64.ctb

Notes On Sample Descriptions

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



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

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

Log of Borehole BH1

Project No: OTT-00257901-A0

Figure No. 4

Project: Proposed Commercial and Residential Development

Page. 1 of 1

Location: 1356 Clyde Avenue, Ottawa, ON

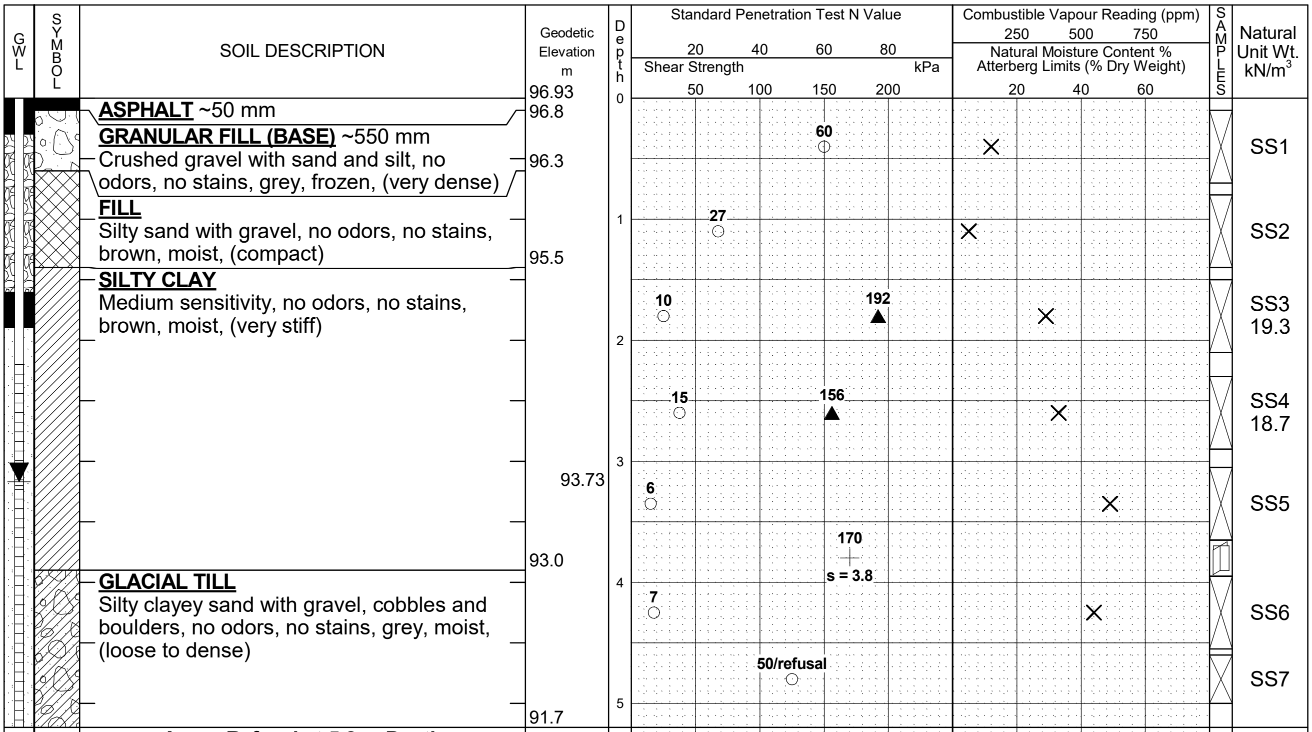
Date Drilled: 'February 4, 2019

Drill Type: CME 55 Rubber Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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	▲



SOIL DESCRIPTION	Geodetic Elevation m
	96.93
ASPHALT ~50 mm	96.8
GRANULAR FILL (BASE) ~550 mm Crushed gravel with sand and silt, no odors, no stains, grey, frozen, (very dense)	96.3
FILL Silty sand with gravel, no odors, no stains, brown, moist, (compact)	95.5
SILTY CLAY Medium sensitivity, no odors, no stains, brown, moist, (very stiff)	93.0
GLACIAL TILL Silty clayey sand with gravel, cobbles and boulders, no odors, no stains, grey, moist, (loose to dense)	91.7

Auger Refusal at 5.2 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00257901-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	2.5	3.0
14 days	3.2	
21 days	3.2	

CORE DRILLING RECORD

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

Log of Borehole BH2

Project No: OTT-00257901-A0

Project: Proposed Commercial and Residential Development

Location: 1356 Clyde Avenue, Ottawa, ON

Figure No. 5

Page. 1 of 1

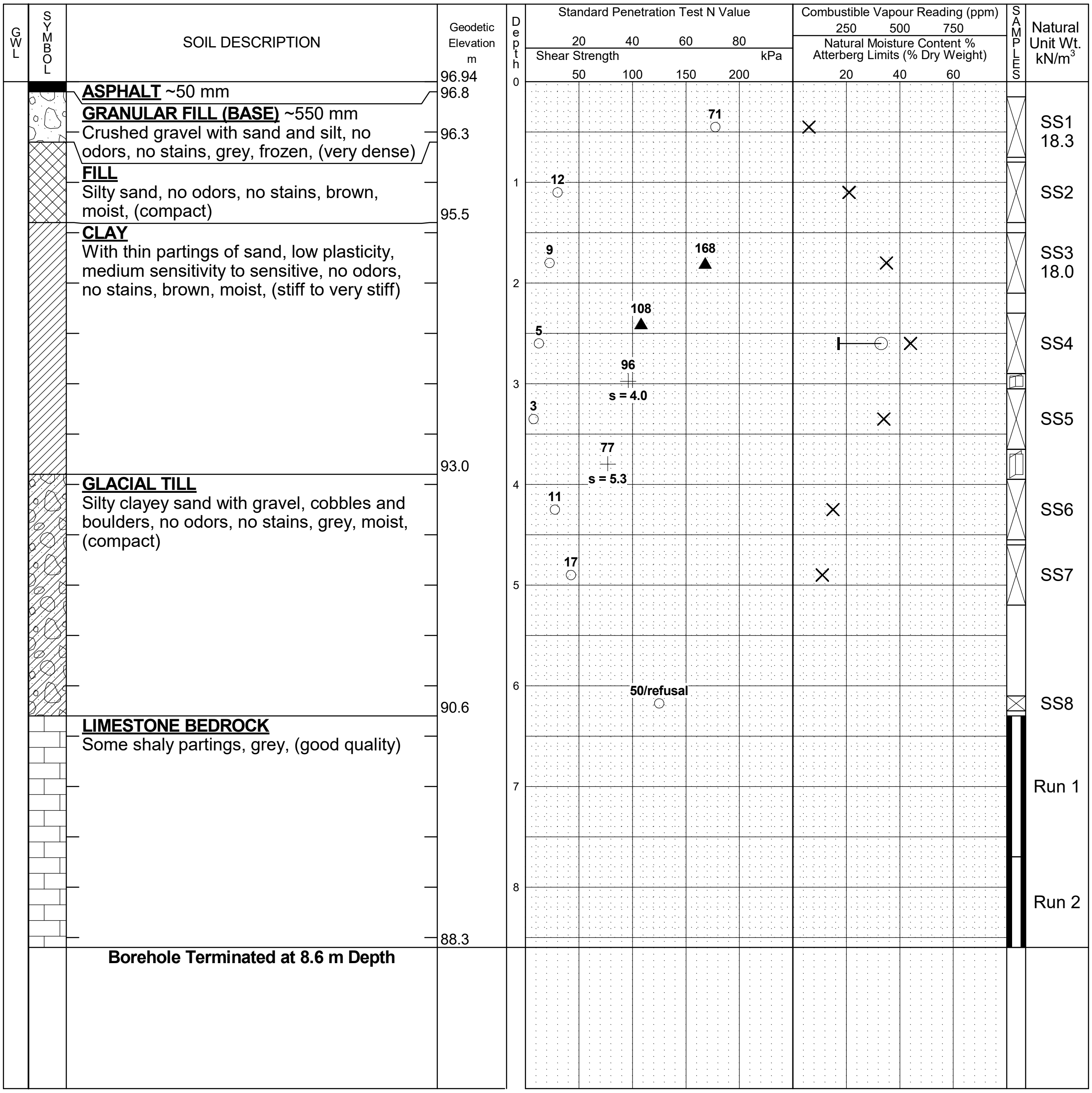
Date Drilled: 'February 4, 2019

Drill Type: CME 55 Rubber Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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	▲



ASPHALT ~50 mm

GRANULAR FILL (BASE) ~550 mm
Crushed gravel with sand and silt, no odors, no stains, grey, frozen, (very dense)

FILL
Silty sand, no odors, no stains, brown, moist, (compact)

CLAY
With thin partings of sand, low plasticity, medium sensitivity to sensitive, no odors, no stains, brown, moist, (stiff to very stiff)

GLACIAL TILL
Silty clayey sand with gravel, cobbles and boulders, no odors, no stains, grey, moist, (compact)

LIMESTONE BEDROCK
Some shaly partings, grey, (good quality)

Borehole Terminated at 8.6 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00257901-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.3 - 7.7	100	88
2	7.7 - 8.6	100	80

Log of Borehole BH3

Project No: OTT-00257901-A0

Project: Proposed Commercial and Residential Development

Location: 1356 Clyde Avenue, Ottawa, ON

Figure No. 6

Page. 1 of 1

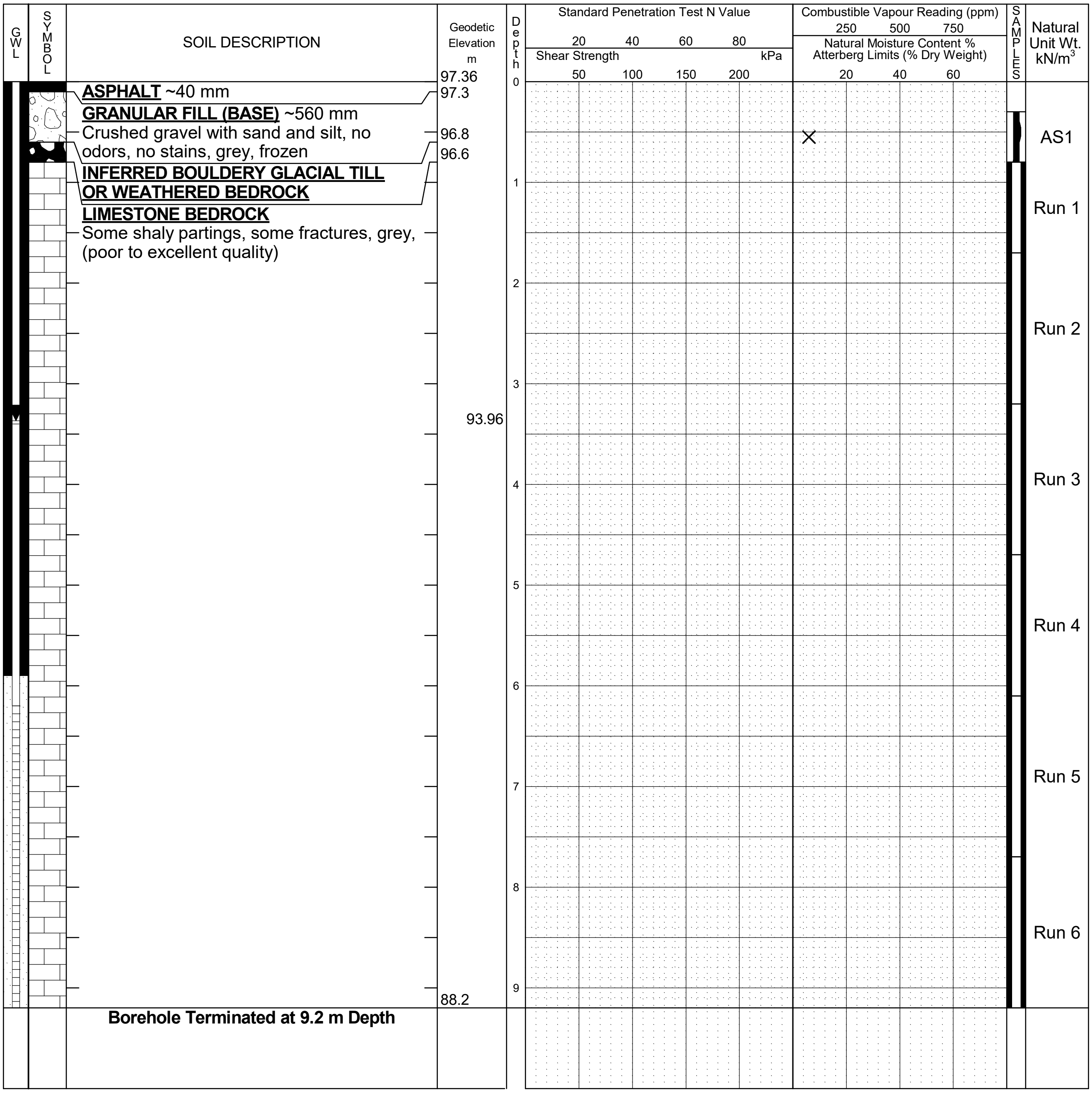
Date Drilled: 'February 5, 2019

Drill Type: CME 55 Rubber Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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	▲



97.36

ASPHALT ~40 mm — 97.3

GRANULAR FILL (BASE) ~560 mm
Crushed gravel with sand and silt, no odors, no stains, grey, frozen — 96.8

INFERRED BOULDERY GLACIAL TILL OR WEATHERED BEDROCK — 96.6

LIMESTONE BEDROCK
Some shaly partings, some fractures, grey, (poor to excellent quality)

93.96

88.2

Borehole Terminated at 9.2 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00257901-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
20 days	3.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	0.8 - 1.7	100	42
2	1.7 - 3.2	100	42
3	3.2 - 4.7	100	63
4	4.7 - 6.1	100	74
5	6.1 - 7.7	100	82
6	7.7 - 9.2	100	95

Log of Borehole BH4

Project No: OTT-00257901-A0

Project: Proposed Commercial and Residential Development

Location: 1356 Clyde Avenue, Ottawa, ON

Figure No. 7

Page. 1 of 1

Date Drilled: 'February 4, 2019

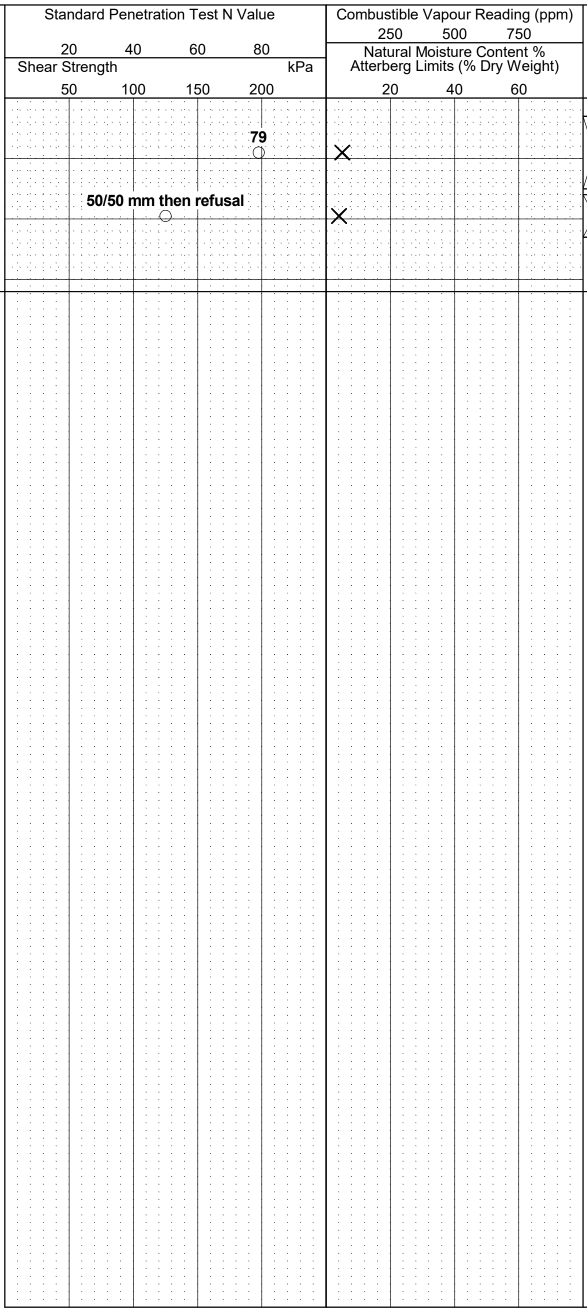
Drill Type: CME 55 Rubber Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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	SAMPLES	Natural Unit Wt. kN/m^3
			96.8	0		
		ASPHALT ~40 mm	96.7			
		GRANULAR FILL (BASE) ~ 1000 mm Crushed gravel with sand and silt, no odors, no stains, grey; upper 700 mm frozen, (very dense)	95.7	1	☒ SS1 ☒ SS2	
		INFERRED BOULDERY GLACIAL TILL OR WEATHERED BEDROCK	95.2			
		Auger Refusal at 1.6 m Depth				



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00257901-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

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

Log of Borehole BH5

Project No: OTT-00257901-A0

Project: Proposed Commercial and Residential Development

Location: 1356 Clyde Avenue, Ottawa, ON

Figure No. 8

Page. 1 of 1

Date Drilled: 'February 4, 2019

Drill Type: CME 55 Rubber Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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		

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
			96.36	0				
		ASPHALT ~50 mm	96.3		50/75 mm then refusal ○	×	☒	SS1
		GRANULAR FILL (BASE) ~ 300 mm Crushed gravel with sand and silt, no odors, no stains, grey, frozen	96.0					
		INFERRED BOULDERY GLACIAL TILL OR WEATHERED BEDROCK	95.6					
		Auger Refusal at 0.8 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00257901-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

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

Log of Borehole BH6

Project No: OTT-00257901-A0

Figure No. 9

Page. 1 of 1

Project: Proposed Commercial and Residential Development

Location: 1356 Clyde Avenue, Ottawa, ON

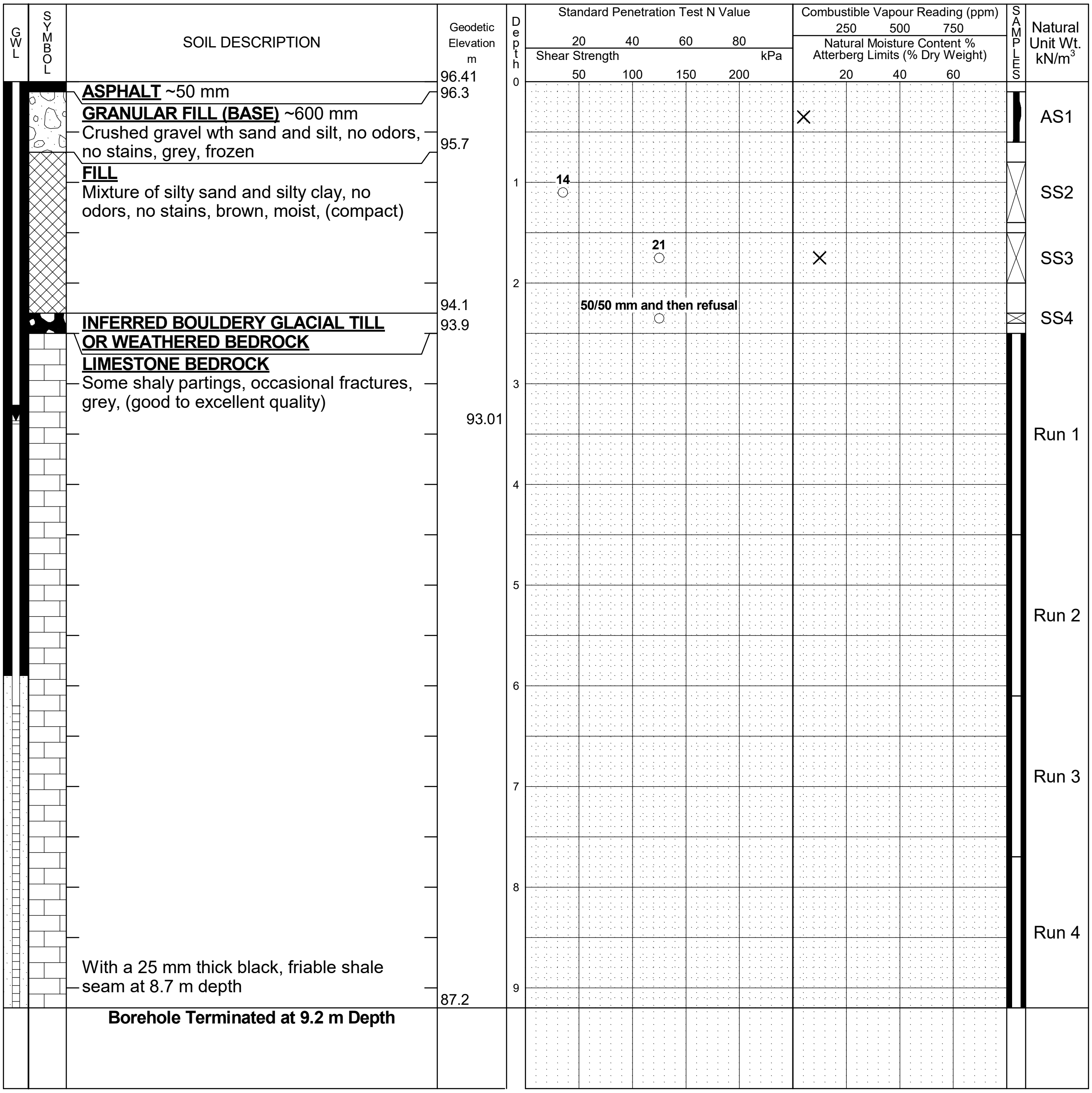
Date Drilled: 'February 5, 2019

Drill Type: CME 55 Rubber Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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	▲



Geodetic Elevation (m)	Soil Description
96.41	
96.3	**ASPHALT** ~50 mm
95.7	**GRANULAR FILL (BASE)** ~600 mm Crushed gravel wth sand and silt, no odors, no stains, grey, frozen
94.1	**FILL** Mixture of silty sand and silty clay, no odors, no stains, brown, moist, (compact)
93.9	**INFERRED BOULDERY GLACIAL TILL OR WEATHERED BEDROCK**
93.01	**LIMESTONE BEDROCK** Some shaly partings, occasional fractures, grey, (good to excellent quality)
87.2	With a 25 mm thick black, friable shale seam at 8.7 m depth

Borehole Terminated at 9.2 m Depth

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00257901-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
13 days	3.4	
20 days	3.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	2.5 - 4.5	100	90
2	4.5 - 6.1	100	98
3	6.1 - 7.7	100	98
4	7.7 - 9.2	100	89

Log of Borehole BH7

Project No: OTT-00257901-A0

Project: Proposed Commercial and Residential Development

Location: 1356 Clyde Avenue, Ottawa, ON

Figure No. 10

Page. 1 of 1

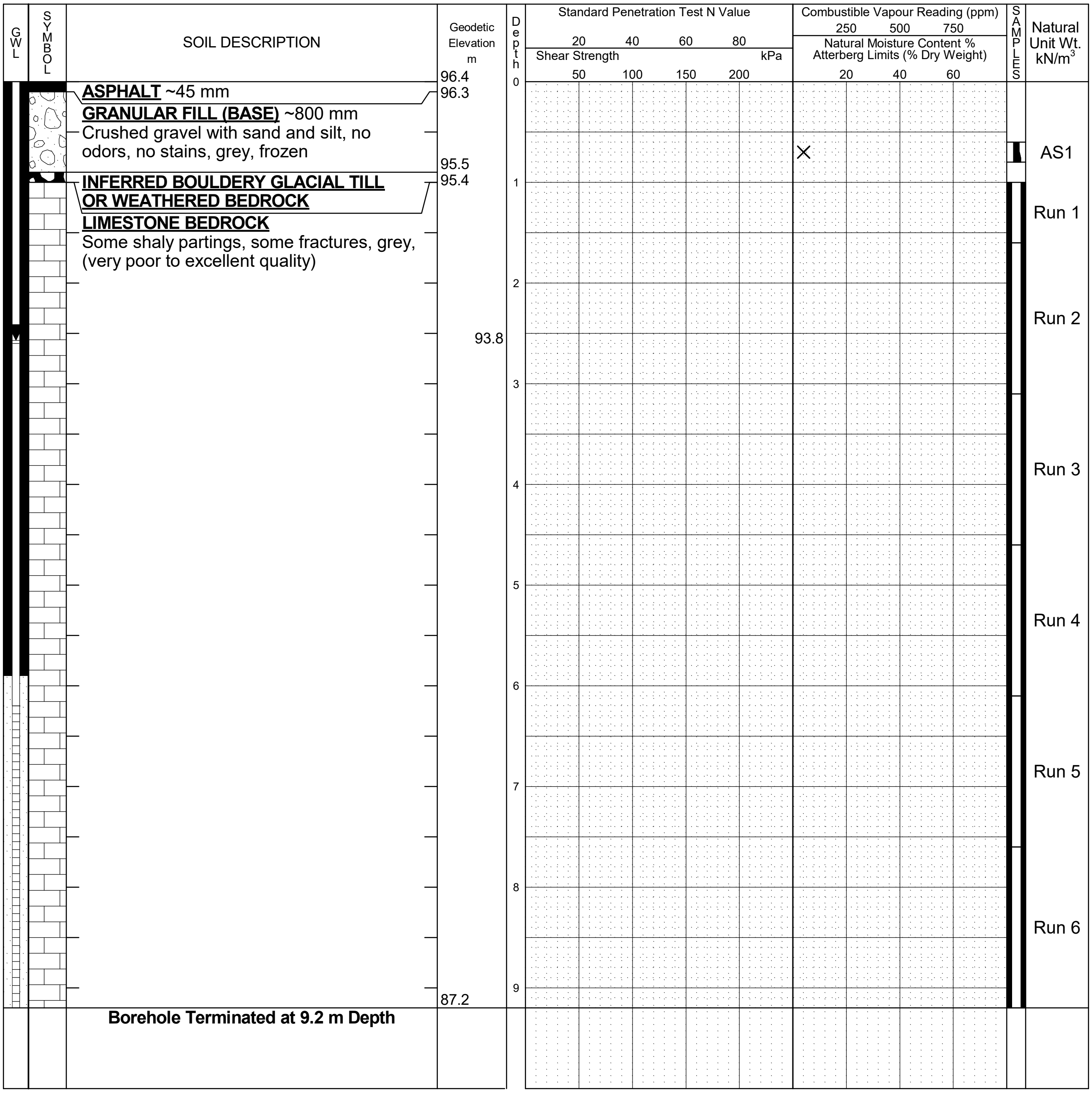
Date Drilled: 'February 5, 2019

Drill Type: CME 55 Rubber Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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	▲



Geodetic Elevation m	SOIL DESCRIPTION
96.4 96.3	**ASPHALT** ~45 mm
	GRANULAR FILL (BASE) ~800 mm Crushed gravel with sand and silt, no odors, no stains, grey, frozen
95.5 95.4	**INFERRED BOULDERY GLACIAL TILL OR WEATHERED BEDROCK**
	LIMESTONE BEDROCK Some shaly partings, some fractures, grey, (very poor to excellent quality)
93.8	
87.2	

Borehole Terminated at 9.2 m Depth

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00257901-A0

WATER LEVEL RECORDS

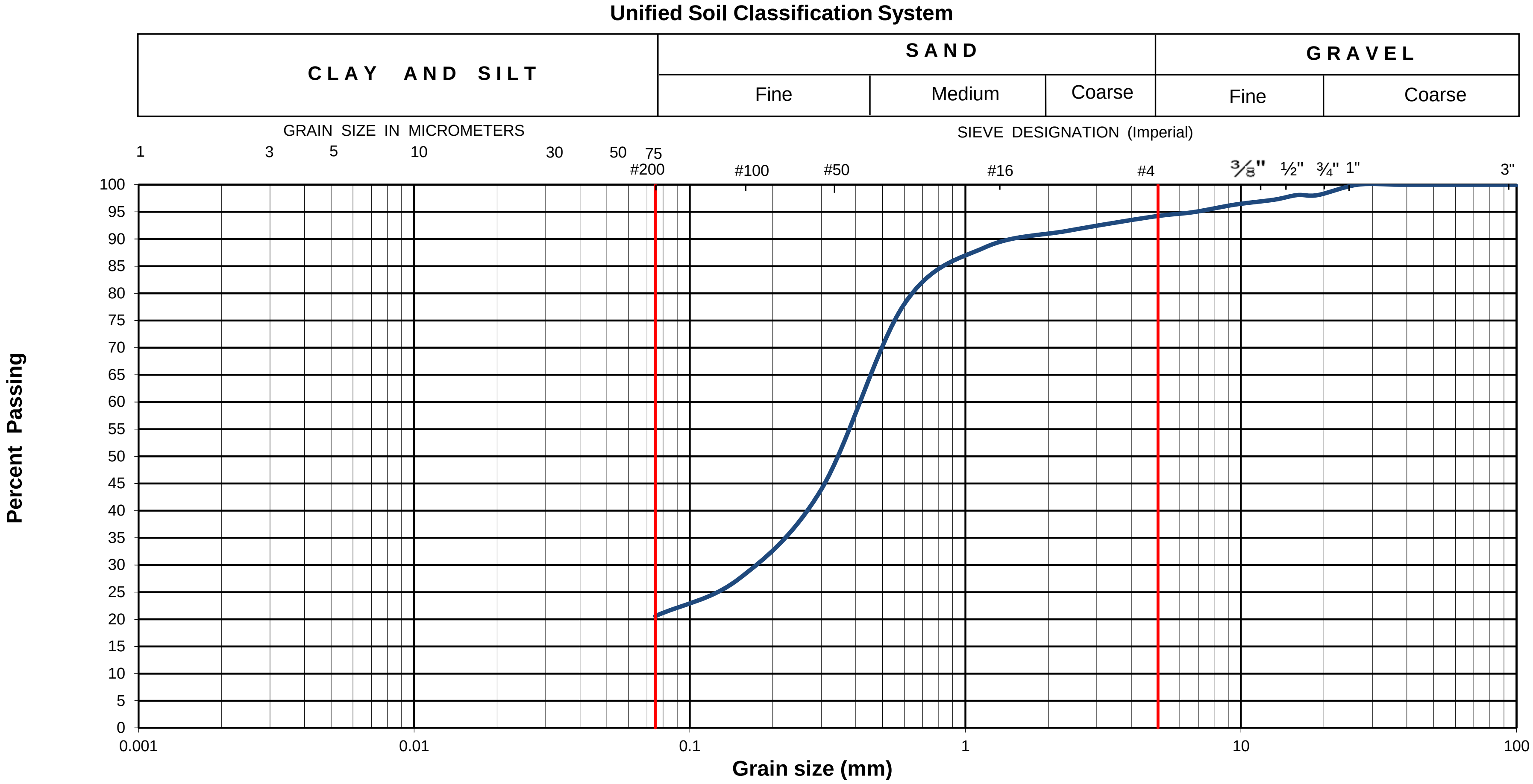
Date	Water Level (m)	Hole Open To (m)
13 days	2.8	
20 days	2.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	1 - 1.6	100	0
2	1.6 - 3.1	100	98
3	3.1 - 4.6	100	96
4	4.6 - 6.1	100	86
5	6.1 - 7.6	100	69
6	7.6 - 9.2	100	87

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

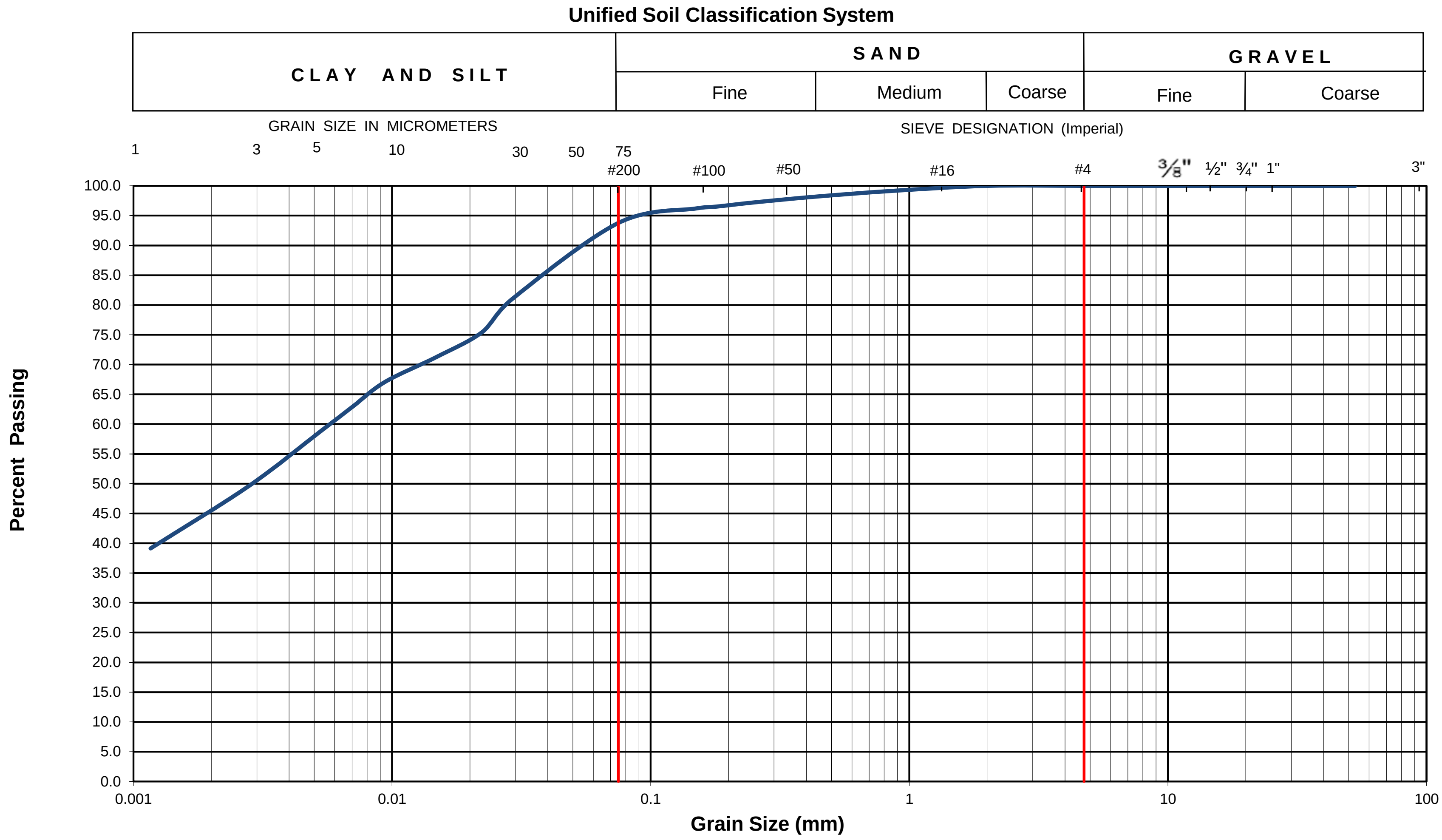
Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136



EXP Project No.:	**OTT-00257901-A0**	**Project Name :**	**Proposed Commercial and Residential Development**				
Client :	**Golpro Holdings Inc.**	**Project Location :**	**1356 Clyde Avenue, Ottawa, ON**				
Date Sampled :	**February 4, 2020**	**Borehole No:**	**BH2**	**Sample:**	**SS2**	**Depth (m) :**	**0.8-1.4**
Sample Composition :		**Gravel (%)**	**6**	**Sand (%)**	**73**	**Silt & Clay (%)**	**21**
Sample Description :	**FILL: Silty Sand (SM)**					**Figure :**	**11**

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

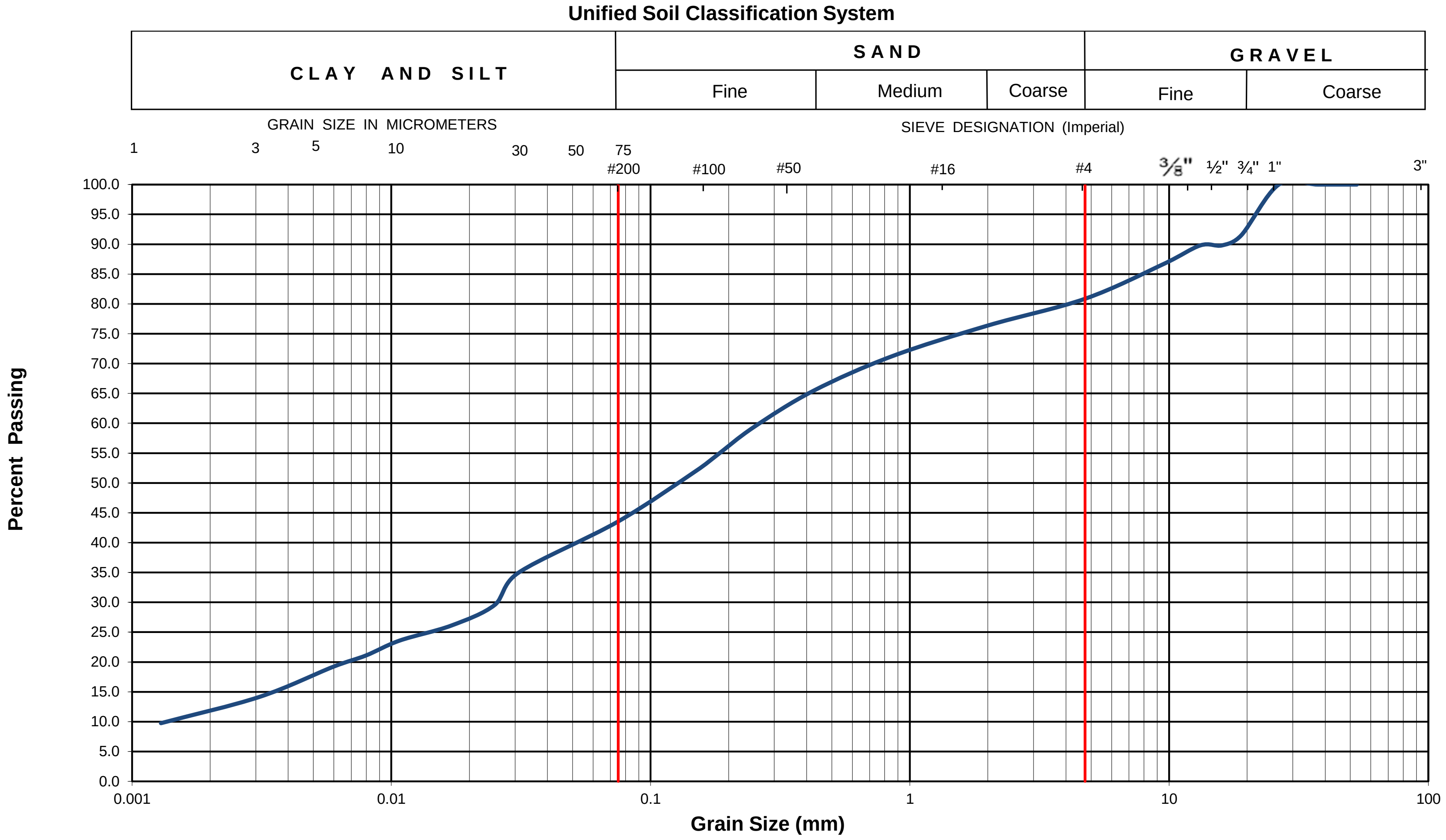
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



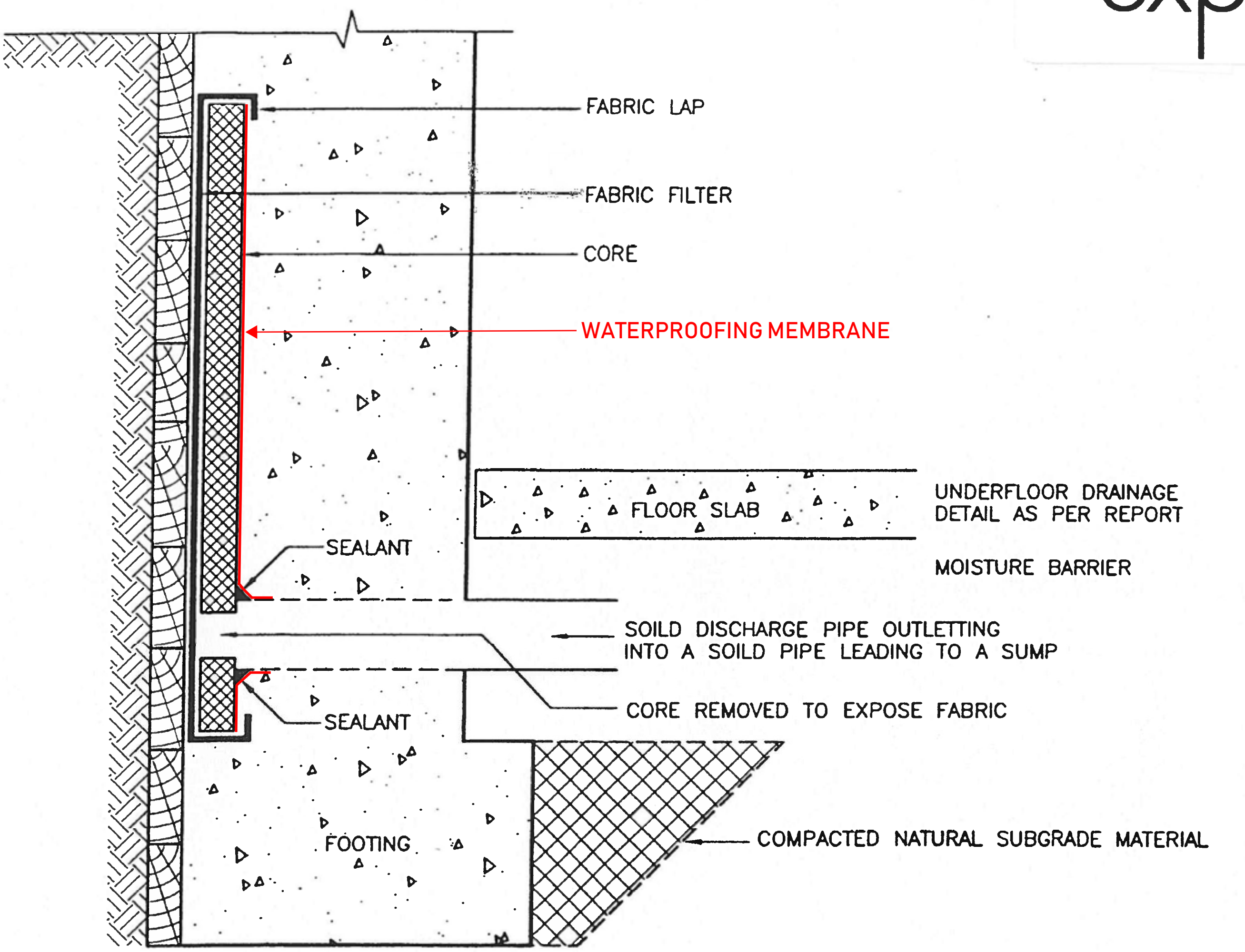
EXP Project No.:	OTT-00257901-A0	**Project Name :**	Proposed Commercial and Residential Development				
Client :	Golpro Holdings Inc.	**Project Location :**	1356 Clyde Avenue, Ottawa, ON				
Date Sampled :	February 4, 2020	**Borehole No:**	BH1	**Sample No.:**	SS4	**Depth (m) :**	2.3-2.9
Sample Description :		**% Silt and Clay**	94	**% Sand**	6	**% Gravel**	0
Sample Description :		Silty Clay of Low Plasticity (CL)				**Figure :**	12

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-00257901-A0**	**Project Name :**	**Proposed Commercial and Residential Development**						
Client :	**Golpro Holdings Inc.**	**Project Location :**	**1356 Clyde Avenue, Ottawa, ON**						
Date Sampled :	**February 4, 2020**	**Borehole No:**	**BH2**	**Sample No.:**	**SS7**	**Depth (m) :**	**4.6-5.2**		
Sample Description :		**% Silt and Clay**	**44**	**% Sand**	**37**	**% Gravel**	**19**	**Figure :**	**13**
Sample Description :	**GLACIAL TILL: Silty, Clayey Sand with Gravel (SC-SM)**								



SECTION AT DISCHARGE PIPE

NOTES:

1. DRAINAGE CORE AND CLOTH TO BE TERRADRAIN 200 OR EQUIVALENT.
2. INSTALLATION INSTRUCTIONS AS PER MANUFACTURES SPECIFICATION.
3. TO BE FULL WIDTH UNLESS OTHERWISE RECOMMENDED BY THE ENGINEER.
4. FINAL DETAIL MUST BE APPROVED BEFORE SYSTEM IS CONSIDERED ACCEPTABLE.
5. TERRADRAIN 200 SHOULD BE KEPT A MINIMUM OF 1.2 m BELOW EXTERIOR FINISHED GRADE.

SUGGESTED EXTERIOR DRAINAGE AGAINST SOLDIER PILE AND LAGGING SHORING SYSTEM

Appendix A: 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: Susan Potyondy

PROJECT: OTT-257901-AO

AGAT WORK ORDER: 20Z578602

SOIL ANALYSIS REVIEWED BY: Nivine Basily, Inorganics Report Writer

DATE REPORTED: Mar 04, 2020

PAGES (INCLUDING COVER): 5

VERSION*: 1

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

*Notes

Disclaimer:

- *All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.*
- *All samples will be disposed of within 30 days following analysis, unless expressly agreed otherwise in writing. Please contact your Client Project Manager if you require additional sample storage time.*
- *AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.*
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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: 20Z578602

PROJECT: OTT-257901-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: Susan Potyondy

SAMPLING SITE:1356 Clyde Ave

SAMPLED BY:EXP

Inorganic Chemistry (Soil)

DATE RECEIVED: 2020-02-27

DATE REPORTED: 2020-03-04

			SAMPLE DESCRIPTION:	BH2 SS5 10'-12'		BH2 Run 1 20'8"-25'4"	BH7 Run 6 24'10"-30'
			SAMPLE TYPE:	Soil		Soil	Soil
			DATE SAMPLED:	2020-02-04		2020-02-04	2020-02-05
Parameter	**Unit**	**G / S**	**RDL**	**972894**	**RDL**	**972896**	**972897**
Chloride (2:1)	µg/g		8	1500	2	43	49
Sulphate (2:1)	µg/g		8	358	2	51	54
pH (2:1)	pH Units		NA	8.03	NA	8.68	8.70
Resistivity (2:1) (Calculated)	ohm.cm		1	340	1	4310	4030

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

972894 EC, 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.

Elevated RDLs indicate the degree of sample dilutions prior to the analysis to keep analytes within the calibration range or reduce matrix interference.

972896-972897 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By: Nivine Basily

Quality Assurance

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-257901-AO
SAMPLING SITE:1356 Clyde Ave

AGAT WORK ORDER: 20Z578602
ATTENTION TO: Susan Potyondy
SAMPLED BY:EXP

Soil Analysis

RPT Date: Mar 04, 2020			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Inorganic Chemistry (Soil)															
Chloride (2:1)	976879		4710	4720	0.3%	< 2	92%	70%	130%	108%	80%	120%	NA	70%	130%
Sulphate (2:1)	976879		223	203	9.4%	< 2	108%	70%	130%	106%	80%	120%	107%	70%	130%
pH (2:1)	972879		8.12	8.18	0.7%	NA	100%	90%	110%						

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

Matrix spike: Spike level < native concentration. Matrix spike acceptance limits do not apply.

Certified By: Nivine Basily

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
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Method Summary

CLIENT NAME: EXP SERVICES INC **AGAT WORK ORDER: 20Z578602**

PROJECT: OTT-257901-AO **ATTENTION TO: Susan Potyondy**

SAMPLING SITE:1356 Clyde Ave **SAMPLED BY:EXP**

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Chloride (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Resistivity (2:1) (Calculated)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION

AGAT Laboratories

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

Chain of Custody Record

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

Laboratory Use Only

Work Order #: 20Z578602

Cooler Quantity: one - no ice

Arrival Temperatures: 15.9 | 16.5 | 16.1
LT- 6.9 | 7.7 | 7.4

Custody Seal Intact: ☐ Yes ☐ No ☐ N/A

Notes: onice

Report Information:

Company: ExP
Contact: Susan Polyondy
Address: 2650 Queensview drive Suite 100
Ottawa ON K2B 8H6
Phone: 613-688-1899 Fax:
Reports to be sent to:
1. Email: Susan.Polyondy@exp.com
2. Email:

Regulatory Requirements:

☐ No Regulatory Requirement

(Please check all applicable boxes)

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

☐ Sewer Use
☐ Sanitary
☐ Storm
Region ______ *Indicate One*
☐ MISA

☐ Regulation 558
☐ CCME
☐ Prov. Water Quality Objectives (PWQO)
☐ Other
______ *Indicate One*

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

Report Guideline on *Certificate of Analysis* ☐ Yes ☐ No

Turnaround Time (TAT) Required:

Regular TAT ☒ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

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

OR Date Required (Rush Surcharges May Apply): ______

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

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

Project Information:

Project: OTT-257901-A0
Site Location: 1356 Clyde Ave
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
BH 2 SS5 10'-12'	Feb4/20				
BH 2 Run 1 20'8"-25'4"	Feb4/20				
BH 7 Run 6 24'10"-30'	Feb5/20				

Sample Identification	Field Filtered - Metals, Hg, CrVI Y / N	O. Reg 153 Metals and Inorganics	☐ All Metals ☐ 153 Metals (excl. Hydrides) ☐ Hydride Metals ☐ 153 Metals (Incl. Hydrides)	ORPs: ☐ B-HWS ☐ Cl ☐ CN ☐ Cr^{6+} ☐ EC ☐ FOC ☐ Hg ☐ pH ☐ SAR	Full Metals Scan	Regulation/Custom Metals	Nutrients: ☐ TP ☐ NH_3 ☐ TKN ☐ NO_3 ☐ NO_2 ☐ NO_3+NO_2	Volatiles: ☐ VOC ☐ BTEX ☐ THM	PHCs F1 - F4	ABNs	PAHs	PCBs: ☐ Total ☐ Aroclors	Organochlorine Pesticides	TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	Sewer Use	pH	Sulphate	Chloride	Electro Resistivity		Potentially Hazardous or High Concentration (Y/N)
BH 2 SS5 10'-12'																✓	✓	✓	✓		
BH 2 Run 1 20'8"-25'4"																✓	✓	✓	✓		
BH 7 Run 6 24'10"-30'																✓	✓	✓	✓		

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Ryan DiGiuseppe	Feb 26/20	6:30pm	W Berthelet	2020-02-27	9h15
[illegible] FedEx	2020/02/27	14h00			
			Shaumin	Feb 28/2020	10 Am

Page ______ of ______

Nº: T 093824

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

Appendix B: Seismic Shear Wave Survey Report

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

March 10th, 2020

Transmitted by email: Ismail.Taki@exp.com
Our Ref.: GPR-20-02040

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

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
1356 Clyde Avenue, Ottawa (ON)

[Your Ref.: OTT-00257901-A0]

Dear Sir,

Geophysics GPR International inc. has been requested by **exp** Services inc. to carry out seismic shear wave surveys on a property located at 1356 Clyde Avenue, in Ottawa. The geophysical investigations used the Multi-channel Analysis of Surface Waves (MASW), the Extended SPatial AutoCorrelation (ESPAC) methods, and seismic refraction. The results aimed to determine the Site Class.

The surveys were carried out on March 5th, 2020, by Mr. Dominic Déraps, tech. and Emmanuel Truchot, tech. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the test methods, and the results in graphic and table format.

MASW PRINCIPLE

The *Multi-channel Analysis of Surface Waves* (MASW) and the *Extended SPatial AutoCorrelation* (ESPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface waves (“ground roll”). The MASW is considered an “active” method, as the seismic signal is induced at known location and time in the geophones spread axis. Conversely, the ESPAC 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 dispersion properties are expressed as a change of phase velocities with 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/ESPAC records, the corresponding spectrogram analysis and resulting 1D V_S model. The ESPAC method allows deeper Vs soundings, but generally with a lower resolution for the surface portion. Its dispersion curve can then be merged with the higher frequency one from the MASW to calculate a more complete inversion.

INTERPRETATION

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

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

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

SURVEY DESIGN

The main seismic acquisition spreads were laid east (front) of the building (cf. Figure 2). The geophone spacing was 3.0 metres for the main spread, using 24 geophones. Two shorter spreads, with 0.5 and 1 metre spacing, were dedicated to the shallow materials.

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

The shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length. The seismic records were produced with a seismograph Terraloc MK6 (from ABEM Instrument), and the geophones were 4.5 Hz. A 9 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic spreads.

RESULTS

From seismic refraction, the rock V_S was calculated between 2040 and 2065 m/s for its sounder shallow portion (cf. Figure 5).

From the MASW results, the calculated seismic shear wave velocities are illustrated at the Figure 6 and the $\overline{V}_{S30}$ calculation is presented at the Table 1.

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 up to 30 metres, as:

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

(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 for the site is 920.8 m/s (cf. Table 1), corresponding to the Site Class “B”. However, the Site Classes A and B should not be used if there is 3 metres or more of unconsolidated materials between the rock surface and the bottom of the foundation.

In the case the foundation would be less than 3 metres from the rock surface, the minimal $\overline{V}_{S30}$* value would be 1311.2 m/s (cf. Table 2), allowing to use the Site Class "B". If there would be 2.1 metres or less between the rock surface and the bottom of the foundation, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, and the Site Class "A" could be used.

CONCLUSION

As a complementary part of a geotechnical study, **exp** Services inc. mandated Geophysics GPR International inc. to conduct some seismic surveys on a property located at 1356 Clyde Avenue, in Ottawa (ON), to determine the Site Class. Geophysical surveys were carried out using the seismic MASW and ESPAC analysis methods, and the seismic refraction. The $\overline{V}_{S30}$ calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 921 m/s, corresponding to the Site Class “B” ($760 < \overline{V}_{S30} \leq 1500$ m/s). However, the Site Classes “A” and “B” are not to be used if there is 3 metres or more of unconsolidated materials between the rock surface and the underside of the footing or mat foundation.

In the case the foundation would be less than 3 metres from the rock surface, the Site Class “B” could be used ($\overline{V}_{S30}{}^{*} \geq 1311$ m/s). In the case the foundation would be less than 2.1 metres from the rock surface, the Site Class “A” could be used ($\overline{V}_{S30}{}^{*} > 1500$ m/s).

It must be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, quick and highly sensitive clays, high moisture content etc. 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 be to your satisfaction, we remain yours truly,

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

Figure 1: Regional location of the Site

(source: *OpenStreetMap©*)

Figure 2: Location of the seismic spreads

(source: geoOttawa©)

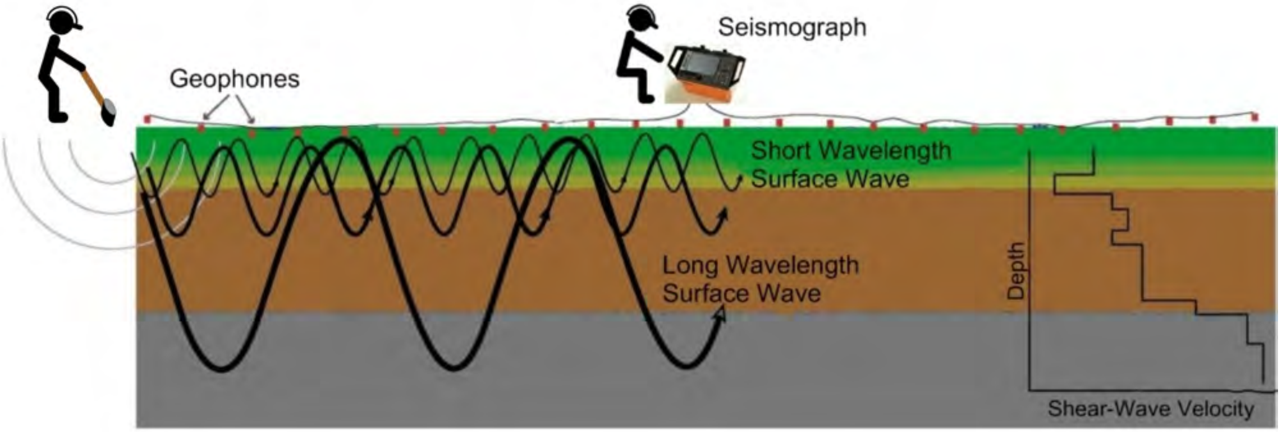
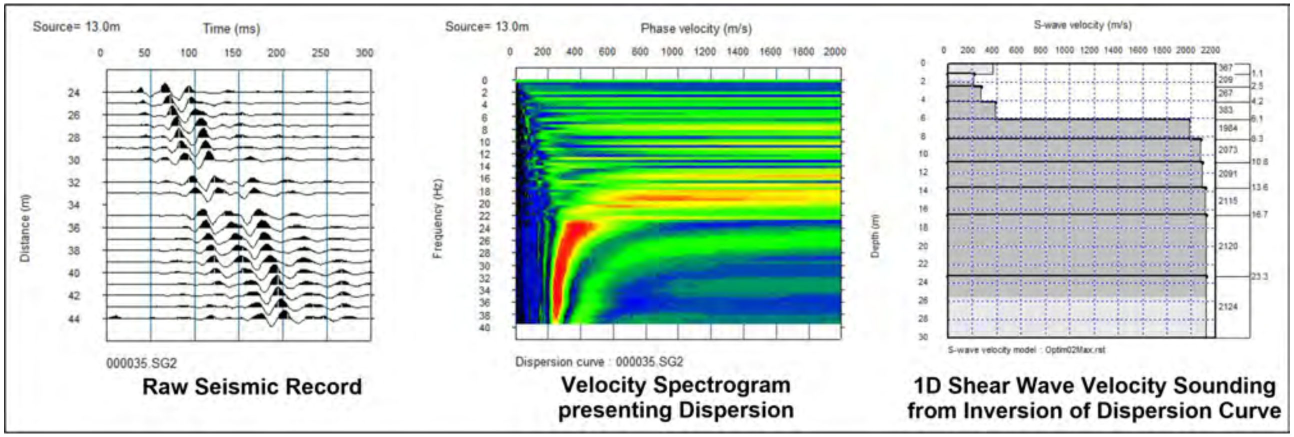


Figure 3: MASW Operating Principle

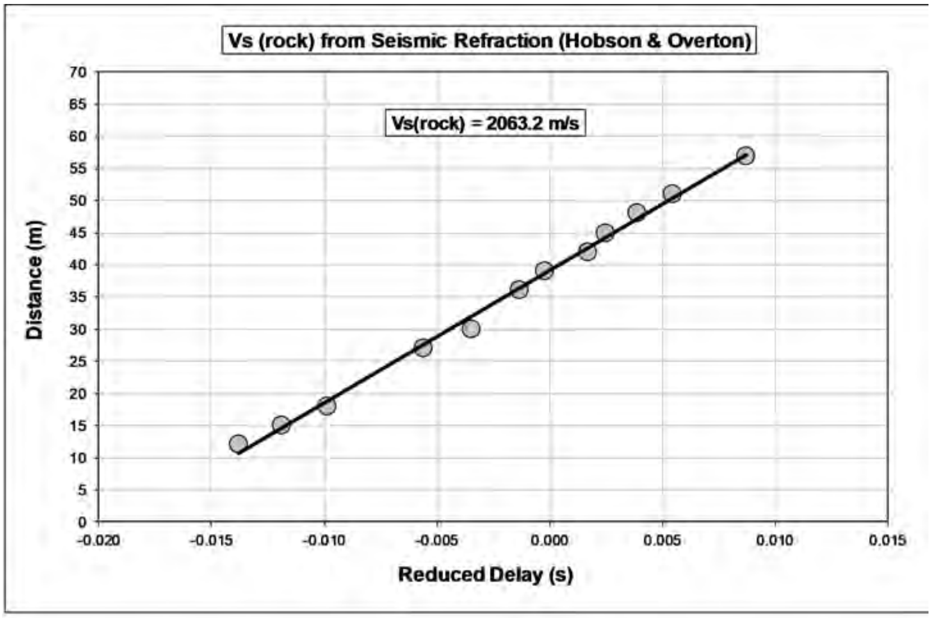


Raw Seismic Record

Velocity Spectrogram presenting Dispersion

1D Shear Wave Velocity Sounding from Inversion of Dispersion Curve

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



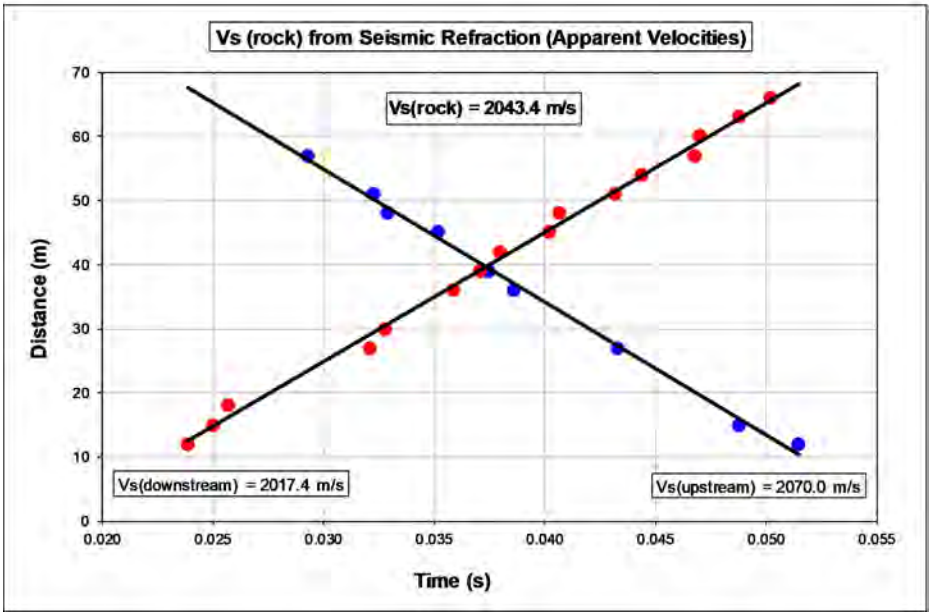


Figure 5: Rock V_S from Seismic Refraction

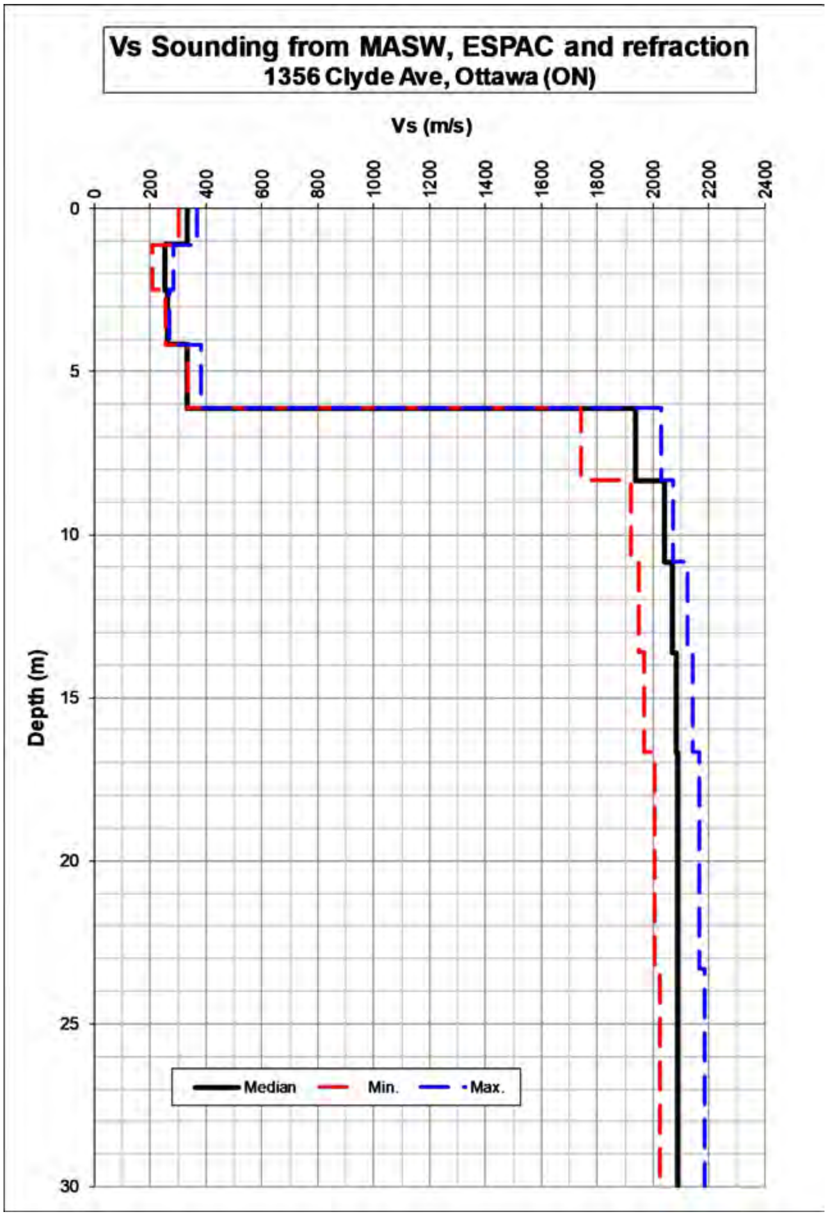


Figure 6: MASW Shear-Wave Velocity Sounding

TABLE 1

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

Depth	Vs			Thickness	Cumulative Thickness	Med. Vs Delay	Cumulative Delay	Avg. Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	299.5	331.6	367.2					
1.11	210.0	253.5	284.7	1.11	1.11	0.003351	0.003351	331.6
2.50	253.2	262.4	269.2	1.39	2.50	0.005478	0.008829	283.2
4.17	332.6	333.6	383.3	1.67	4.17	0.006353	0.015182	274.4
6.11	1742.4	1938.5	2027.2	1.94	6.11	0.005828	0.021010	290.9
8.33	1918.7	2040.9	2073.2	2.22	8.33	0.001146	0.022156	376.1
10.83	1948.8	2067.1	2125.0	2.50	10.83	0.001225	0.023381	463.3
13.61	1969.7	2081.8	2139.9	2.78	13.61	0.001344	0.024725	550.5
16.67	2003.2	2086.4	2163.7	3.06	16.67	0.001468	0.026193	636.3
23.33	2024.9	2089.5	2184.1	6.67	23.33	0.003195	0.029388	794.0
30				6.67	30.00	0.003191	0.032579	920.8

V_{S30} (m/s)	**920.8**
Class	**B** (1)

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

TABLE 2

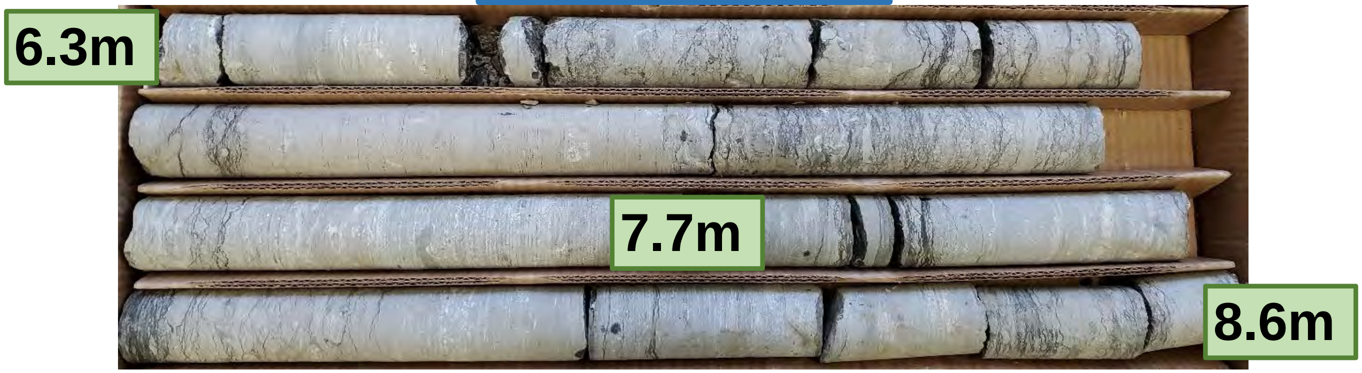
V_{S30}* Calculation for the Site Class (less than 3 metres of overburden)

Depth	Vs			Thickness	Cumulative Thickness	Med. Vs Delay	Cumulative Delay	Avg. Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	299.5	331.6	367.2	**Considering less than 3 metres of unconsolidated material between the rock and the bottom of the foundation**				
1.11	210.0	253.5	284.7					
2.50	253.2	262.4	269.2					
3.12	253.2	262.4	269.2					
4.17	332.6	333.6	383.3	1.05	1.05	0.003989	0.003989	262.4
6.11	1742.4	1938.5	2027.2	1.94	2.99	0.005828	0.009818	304.7
8.33	1918.7	2040.9	2073.2	2.22	5.21	0.001146	0.010964	475.5
10.83	1948.8	2067.1	2125.0	2.50	7.71	0.001225	0.012189	632.8
13.61	1969.7	2081.8	2139.9	2.78	10.49	0.001344	0.013533	775.2
16.67	2003.2	2086.4	2163.7	3.06	13.55	0.001468	0.015000	903.1
23.33	2024.9	2089.5	2184.1	6.67	20.21	0.003195	0.018196	1110.9
33.12				9.79	30.00	0.004684	0.022880	1311.2

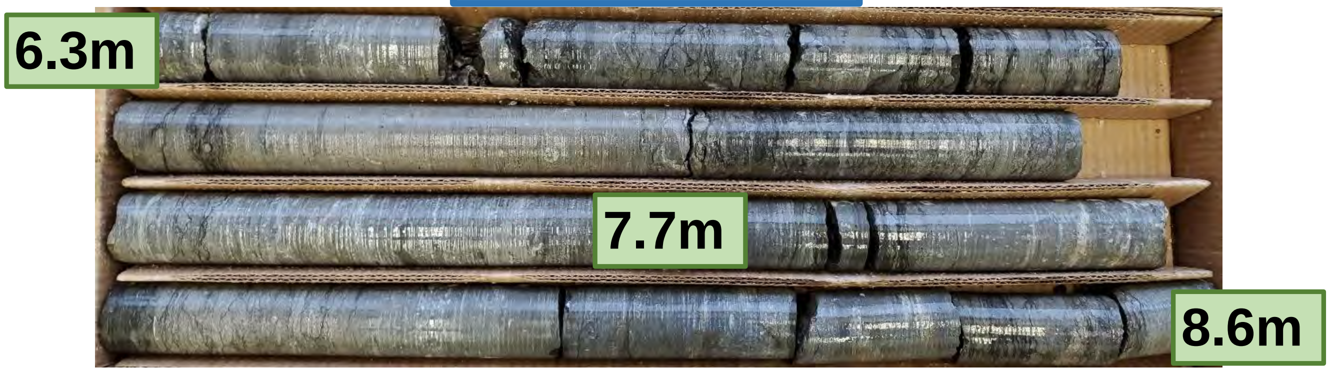
V_{S30} (m/s)	**1311.2**
Class	**B**

Appendix C: Bedrock Core Photographs

DRY BEDROCK CORES



WET BEDROCK CORES



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Ottawa, ON K2B 8H6
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• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

borehole no.
BH2

date cored
Feb 04, 2020

core runs
Run 1: 6.3 m - 7.7 m
Run 2: 7.7 m - 8.6 m

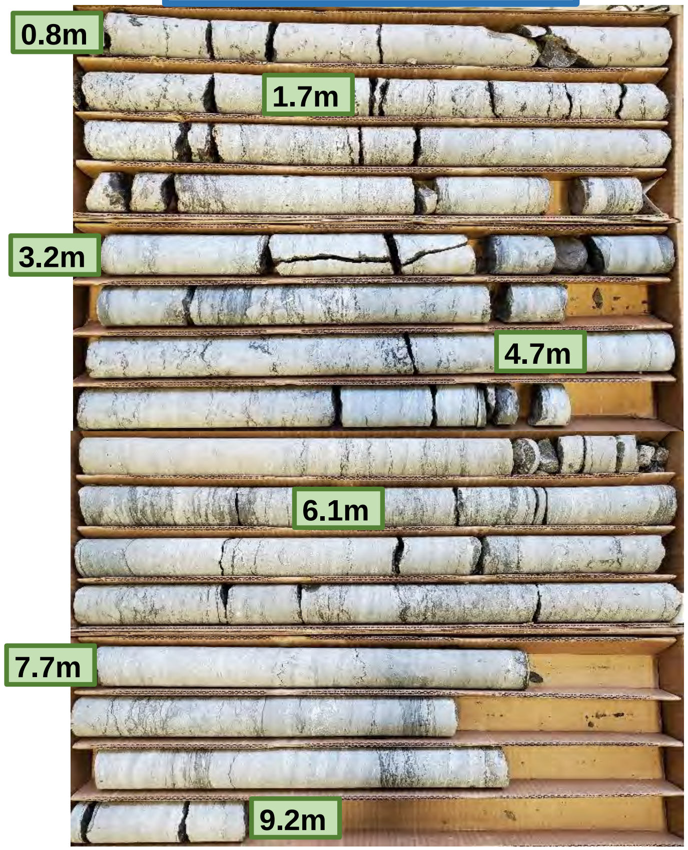
PROJECT
Geotechnical Investigation
1356 Clyde Avenue, Ottawa, ON

ROCK CORE PHOTOGRAPHS

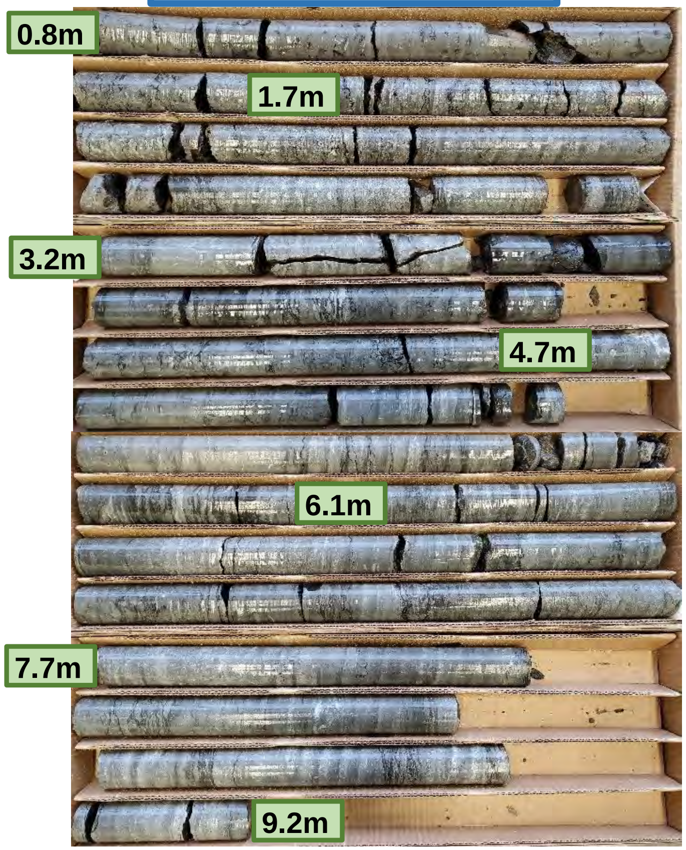
project no.
OTT-00257901-A0

FIG. C-1

DRY BEDROCK CORES



WET BEDROCK CORES



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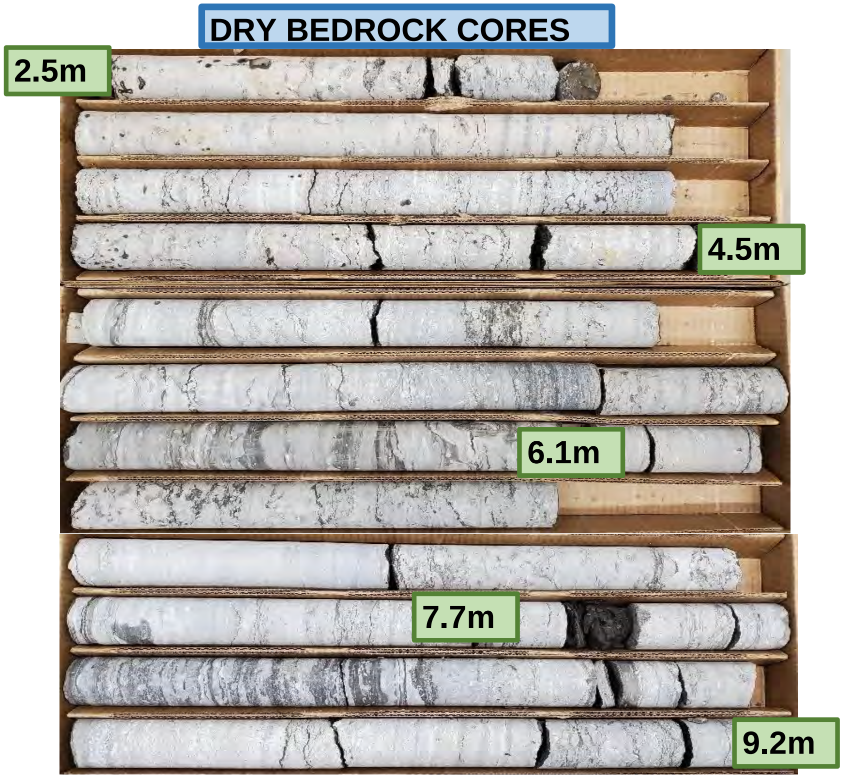
borehole no. **BH3**	core runs Run 1: 0.8 m - 1.7 m Run 2: 1.7 m - 3.2 m Run 3: 3.2 m - 4.7 m Run 4: 4.7 m - 6.1 m Run 5: 6.1 m - 7.7 m Run 6: 7.7 m - 9.2 m	PROJECT	Geotechnical Investigation 1356 Clyde Avenue, Ottawa, ON	project no. OTT-00257901-A0
date cored Feb 05, 2020			ROCK CORE PHOTOGRAPHS	FIG. C-2



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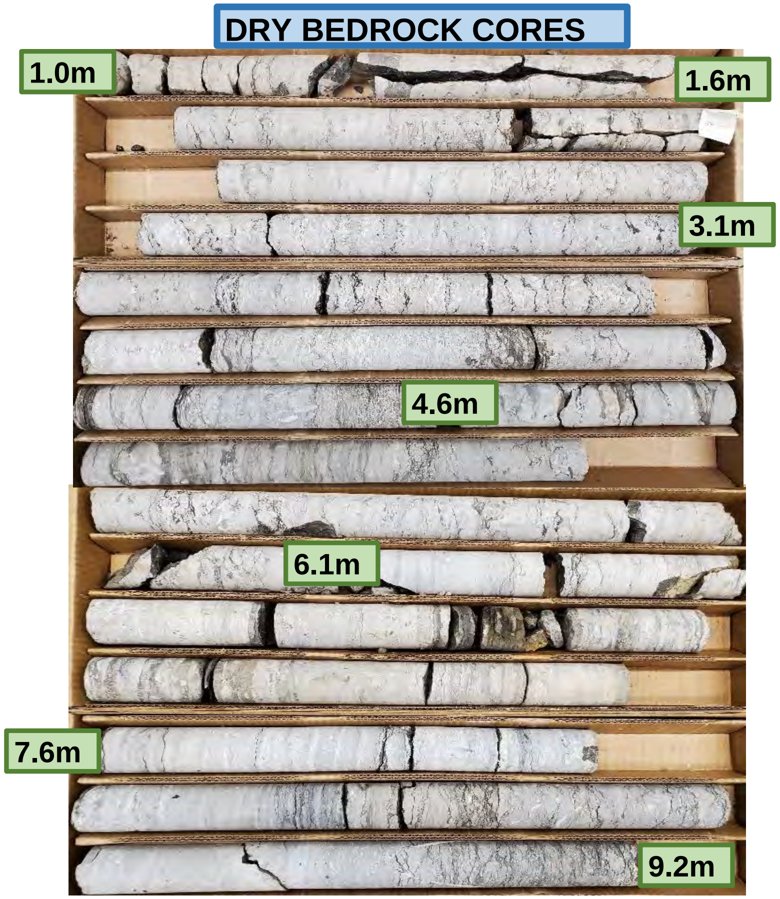
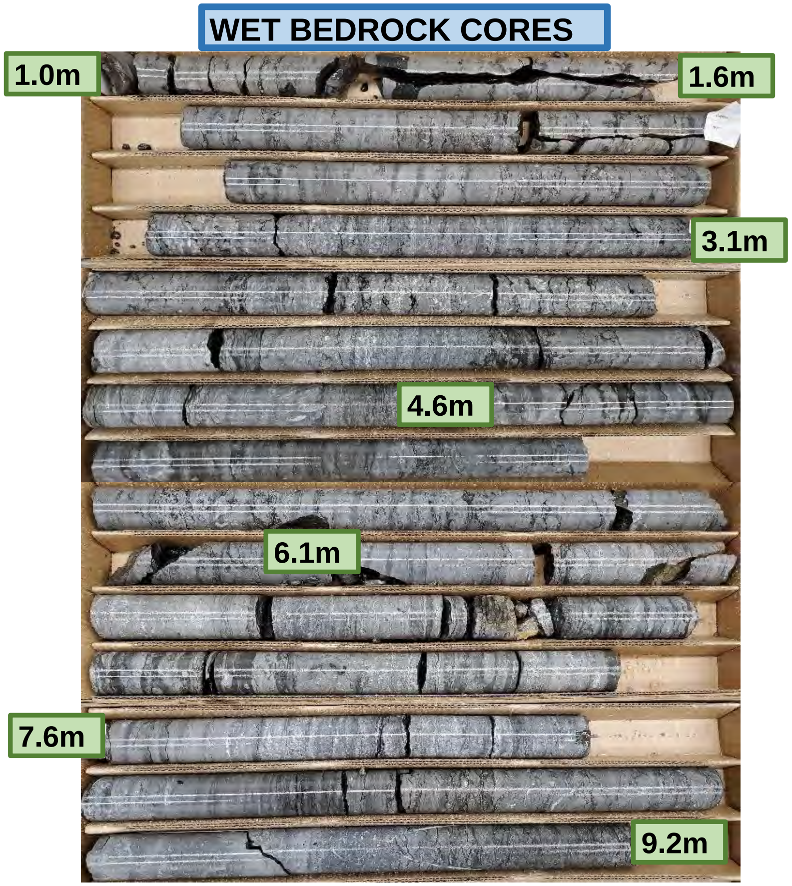
borehole no.	core runs	PROJECT	project no.
BH6	Run 1: 2.5 m - 4.5 m Run 2: 4.5 m - 6.1 m Run 3: 6.1 m - 7.7 m Run 4: 7.7 m - 9.2 m	Geotechnical Investigation 1356 Clyde Avenue, Ottawa, ON	OTT-00257901-A0
date cored Feb 05, 2020		ROCK CORE PHOTOGRAPHS	FIG. C-3





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borehole no.	core runs	PROJECT	project no.
BH7	Run 1: 1.0 m - 1.6 m Run 2: 1.6 m - 3.1 m Run 3: 3.1 m - 4.6 m	Geotechnical Investigation 1356 Clyde Avenue, Ottawa, ON	OTT-00257901-A0
date cored Feb 05, 2020	Run 4: 4.6 m - 6.1 m Run 5: 6.1 m - 7.6 m Run 6: 7.6 m - 9.2 m	ROCK CORE PHOTOGRAPHS	FIG. C-4

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