- **Windmill Dream Ontario Holdings LLP c/o Zibi Canada**

Geotechnical Investigation

Type of Document:
Final (supersedes April 17, 2019 final report)

Project Name:
Proposed Office Building
Block 211 and Parcel of Land North of Block 211
Chaudière Island
Ottawa, Ontario

Project Number:
OTT-00250193-C0

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Date Submitted:
June 19, 2019

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Legal Notification

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Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed office building to be located within Zibi's complex in Block 211 on Chaudière Island, Ottawa, Ontario. Authorization to proceed with this geotechnical investigation was provided via Windmill Dream Ontario Holding Purchase Order Number: 910876 dated March 13, 2019.

The proposed development includes a new multi-storey office building with two (2) level underground parking garage. The design elevation of the top of the lowest floor slab of the proposed building will be at Elevation 44.5 m. The elevation of the anticipated maximum depth of the excavation for the proposed building will be at Elevation 42.0 m.

EXP's scope of work also includes environmental investigations of Block 211 and the parcel of land north of Block 211. The findings from these investigations are reported under separate covers.

The fieldwork for the geotechnical investigation was undertaken from February 20 to 25, 2019 and consists the drilling of eight (8) boreholes (Borehole Nos. 1 to 8) advanced to auger refusal and termination depths ranging from 5.5 m to 17.8 m below existing grade (Elevation 47.0 m to 35.9 m). The fieldwork was supervised on a full-time basis by a representative from EXP. Boreholes completed by others on the subject site are also referred to as part of this study as additional information.

A review of the borehole logs indicates that within Block 211, below fill and glacial till, limestone bedrock was contacted at 0.5 m to 4.0 m depths (Elevation 52.4 m to 48.5 m). In Borehole No. 7 located within the footprint of the crevasse identified within Block 211, the bedrock surface was contacted at a greater depth of 13.0 m (Elevation 40.7 m). In the parcel of land situated north of Block 211, the surface of the limestone bedrock was contacted at 3.0 m to 7.0 m depths (Elevation 49.4 m to 44.8 m). Groundwater levels range from 4.5 m to 10.6 m depths (Elevation 47.0 m to 42.5 m).

The site may be classified for seismic site response as **Class A** for foundations founded on the bedrock. It is anticipated that all subsurface soil on site including the fill and underlying glacial till will be excavated and removed from site as part of the environmental remediation. Therefore, in this case, liquefaction of the soils will not apply.

Significant grade raise is not anticipated for the proposed building development. Also, it is anticipated that all the existing soil comprising of fill and glacial till will be removed down to the bedrock as part of the proposed construction. Therefore, grade raise is not a concern at the subject site. However, EXP should be consulted to review the final grading plan.

The top of the lowest slab of the proposed building will be at Elevation 44.5 m and the maximum depth of the bottom of the excavation for the proposed building will be at Elevation 42.0 m. The excavation will be within the bedrock and below the groundwater level and the water level in the Ottawa River. Due to the presence of the groundwater level within the overburden soil of the buried deep crevasse that extends into the proposed building footprint from the Ottawa River and the close proximity of the site to the Ottawa River, significant groundwater flows into the excavation and the newly constructed building may occur during construction and in the long term over the life of the proposed building.

The proposed building may be supported by conventional spread and strip footings founded on the sound bedrock with the lowest slab of the building designed as a slab on grade, provided the building is equipped with permanent perimeter and underfloor drainage systems designed to handle the estimated groundwater flow of 150 m^3/day presented in the EXP hydrogeological report.

The hydrogeological investigation prepared by EXP indicates the groundwater flow into the excavation is estimated at 360 m^3/day. To control the estimated groundwater flows into the excavation and the proposed building, it is recommended that a permanent groundwater barrier system of very low permeability (coefficient of permeability of 10×10^{-10} m/s), such as a clay berm or equivalent, or a groundwater cut-off system such as a secant pile wall shoring system or equivalent be constructed along the full length of the north wall. The permanent groundwater barrier or cut off wall systems will minimize groundwater flows into the excavation and the proposed building from the buried deep crevasse and the Ottawa River. A contingency should be made for extending the barrier wall or cut-off wall system in the south direction at the east and west ends of the north wall, in case groundwater cannot be effectively controlled by the north barrier system or cut off wall alone. The length of the north barrier system or cut-off wall may be shortened and the need for extending the barrier system or wall in the south direction may be reduced or eliminated, if the excavation east and west of the crevasse reveals the surface of the bedrock is higher, the bedrock quality is good and groundwater control does not appear to be problematic. The secant pile wall shoring system should be socketed approximately 1 m into the bedrock and the elevation of the top of the secant pile wall system should match that of the regulatory flood level in the Ottawa River of Elevation of 49.0 m to 49. 5 m.

The permanent groundwater barrier system or cut-off wall system must be designed and installed by an experienced contractor on a performance and end results type of specification.

Along the east and west sides of the excavation beyond the groundwater barrier system or secant pile wall shoring system and along the south side of the excavation, the side slopes of the overburden soil may be cut back in accordance with the Occupational Health and Safety Act (OHSA). If the required side slopes cannot be achieved due to space restrictions on site, the soil may be supported by a shoring system that is socketed into the bedrock. If movement of the shoring system, adjacent structures and infrastructure can be tolerated along the east, west and south sides of the excavation, the excavation side slopes within the overburden soil may be supported by a soldier pile and timber lagging shoring system installed to the bedrock surface. Surface water flows into the excavation, that may be significant during precipitation events, may occur that will require removal from the excavation by sump pumping techniques and large capacity pumps.

Excavation within the bedrock below the bottom of the secant pile wall system and soldier pile timber lagging shoring system may be undertaken with near vertical sides subject to review by a geotechnical engineer. The exposed rock face in the excavation will require a support system consisting of rock bolts to maintain the integrity of the exposed rock face. Significant groundwater flows through the rock face into the excavation may occur in localized areas and these flows can be reduced by grouting permeable seepage zones along the rock face and removing any water that enters the excavation by sump pumping method.

For the spread and strip footing design to support the building and slab-on-grade design of the lowest floor slab of the proposed building, permanent perimeter and underfloor drainage systems are required and should be designed to handle the estimated groundwater flow of 150 m^3/day presented in the EXP hydrogeological report. Since this building design relies completely on the satisfactory performance of the building drainage systems, it is crucial that the permanent perimeter and underfloor drainage systems operate on a continuous basis. The perimeter and underfloor drainage systems should be connected to separate sumps and the design should include back up pumps and generators, in case of mechanical failure and/or power outage.

Details regarding the above are discussed in greater detail in the main body of this report.

This geotechnical engineering report should be read in conjunction with the hydrogeological investigation report dated June 14, 2019 and prepared by EXP.

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1 Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed office building to be located within Zibi's complex in Block 211 on Chaudière Island, Ottawa, Ontario. Authorization to proceed with this geotechnical investigation was provided via Windmill Dream Ontario Holding Purchase Order Number: 910876 dated March 13, 2019.

The proposed development includes a new multi-storey office building with two (2) level underground parking garage. The design elevation of the top of the lowest floor slab of the proposed building will be at Elevation 44.5 m. The elevation of the anticipated maximum depth of the excavation for the proposed building will be at Elevation 42.0 m.

This geotechnical investigation was undertaken to:

a) Establish the subsurface soil, bedrock and groundwater conditions at the eight (8) borehole locations;

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 stratum and comment on the anticipated total and differential settlements of the recommended foundation type;

e) Provide rock parameters for rock anchor design;

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

g) Provide lateral earth pressure parameters (for static and seismic conditions) for the subsurface foundation walls of the proposed building;

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

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 bedrock to buried metal structures/members.

This geotechnical engineering report should be read in conjunction with the hydrogeological investigation report dated June 14, 2019 and prepared by EXP.

EXP's scope of work for this project also includes environmental investigations of Block 211 and the land north of Block 211. The environmental investigations are reported under separate covers.

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 situated on Chaudière Island located in the Ottawa River between Ottawa, Ontario to the south and Hull, Quebec to the north. Access to the site is by Booth Street which divides the island into two (2) parcels of land; east and west parcels. The subject site is located in the northwest portion of the east parcel of the island. The site location is shown in Figure 1.

Chaudière Island is an island that rises above the Ottawa River. The east portion of the island is bounded by the Ottawa River along the north and east sides and the east leg of the Buchanan channel on the south side. The island consists of earth and rock slopes along the north and east sides having a height of +/- 10 m that slope down to the shoreline of the Ottawa River. The south side of the island is +/- 5 m above the east leg of the Buchanan channel with the channel shoreline developed with Building No. 535, a tunnel and retaining walls of the Buchanan channel. The Buchanan channel is an open concrete channel with a concrete base and sides. Based on a review of drawings made available by Zibi Canada, the tunnel floor is at Elevation 49.5 m and the bottom of the east leg of the Buchanan channel is at Elevation +/- 45.0 m.

The proposed new office building will be located in the west vacant land identified as Block 211 and in a portion of the vacant land north of Block 211. The site of the proposed building is bounded by Booth Street to the west, the remaining portion of the vacant parcel of land north of Block 211 along the north side, the industrial building to the east and the gravel road to the south and the existing Building No. 535, tunnel, retaining walls and east leg of the Buchanan channel, south of the gravel road. The land north of Block 211 is currently vacant and is bounded on the north side by the earth (soil) and rocky slope along the Ottawa River, Block 211 to the south, the existing industrial building to the east and Booth Street to the west.

It is known that an historic filled in former channel or crevasse extends from the north shoreline, crosses the land north of Block 211 and extends into Block 211 beneath the location of the proposed building by approximately 30 m. The crevasse appears to be widest at the river shoreline, gradually narrowing to the south. It appears that the north portion of the proposed office building envelope will be located within a part of the crevasse footprint. The crevasse along the north shoreline slope of the island is an earth slope with the remaining portion of the north slope being a rocky slope.

It is our understanding that erosion of a part of the earth (soil) slope along the north shoreline occurred during the spring 2017 flood event and that additional erosion of this slope has occurred during the spring 2019 flood event.

The footprint of the west portion of the adjacent existing industrial building was located within the east portion of the proposed new office building area. The west portion of the existing building was recently demolished with all below grade slabs, foundation walls and footings excavated and removed from the site and the excavated area backfilled with loose soil.

Based on the ground surface elevation at the borehole locations, the topography of the site is relatively flat with borehole ground surface elevations ranging from Elevation 53.7 m to 52.2 m.

3 Available Information

The following geotechnical reports were made available by Zibi Canada and used as reference material in the preparation of this geotechnical engineering report:

- *Preliminary Geotechnical Investigation, Proposed Mixed Use Development, Chaudière and Albert Islands, Ottawa, Ontario*, dated August 11, 2015 and prepared by Paterson Group Inc.
- *Preliminary Geotechnical Investigation, Proposed Development, Chaudière and Albert Islands, Ottawa, Ontario*, dated April 4, 2014 and prepared by Paterson Group Inc.

The referenced reports include borehole information (2006) in the vicinity of Block 211 and the parcel of land north of Block 211 from DST Consulting Engineers Inc. The reports also provide geotechnical engineering comments for the proposed development and a slope stability analysis of the north slope of the parcel of land north of Block 211.

The location of the boreholes from the above referenced geotechnical reports completed by others and located within the subject site are shown in the borehole location plan, Figure 2.

Water levels in the Ottawa River near the site, from the same dates as groundwater level measurements in the monitoring wells installed in the boreholes, were extracted from the Environment and Climate Change Real-time Hydrometric Data web site. The water levels in the Ottawa River are from Station 02LA028 located in the Ottawa River Main Channel below Chaudière Falls. The extracted water levels are summarized in Table I.

Table I: Water Levels in the Ottawa River – Station 02LA028

Date	Minimum Water level (m)	Maximum Water Level (m)
March 5, 2019	42.50	42.85
March 6, 2019	42.51	42.74
May 13, 2019	44.74	45.49
May 22, 2019	43.94	44.62
May 29, 2019	43.48	44.28
May 30, 2019	43.40	44.20

Note:

1) Water levels assumed to be above mean sea level

The 2014 flood risk mapping data for the Ottawa River from the Rideau Valley Conservation Authority (RVCA) and provided by Zibi Canada for a station (Station No. 1045.2) located downstream from the site are summarized in Table II.

Table II: Flood Risk Mapping Data

Return Period (years)	**Water Level (m)**
Regulatory Flood Level (1)	49.46
500	48.15
200	47.53
100	47.08
50	46.62
20	45.97
10	45.45
5	44.87
2	43.95

Notes:

(1) – 100-year Return Energy Grade Line
(2) Water levels assumed to be above mean sea level

It should be noted that the slope along the north shoreline of the Chaudière Island has been eroding and is failing. it is our understanding that a shoreline stabilization and land reclamation project will be undertaken in this area as part of the overall site development and as part of/prior to the construction of the proposed building.

4 Geology of Site

4.1 Surficial Geology

The surficial geology map (Map 1506A – Surficial Geology, Ontario-Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1982) indicates that beneath any fill, Chaudière Island is covered with a thin veneer of unconsolidated Quaternary sediments over bedrock.

4.2 Bedrock Geology

The bedrock geology map (Map 1508A – Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1979) indicates the bedrock at Chaudière Island consists of limestone of the Ottawa formation. The limestone contains shaley partings and interbeds with some sandstone in the basal part. The map indicates that faults are located approximately north, northeast and southwest of Chaudière Island.

5 Procedure

5.1 Borehole Fieldwork

The fieldwork for the geotechnical investigation was undertaken from February 20 to 25, 2019 and consists of eight (8) boreholes (Borehole Nos. 1 to 8) advanced to auger refusal and termination depths ranging from 5.5 m to 17.8 m below existing grade (Elevation 47.0 m to 35.9 m). The borehole fieldwork was supervised on a full-time basis by a representative from EXP.

A summary of the borehole drilling program is shown in Table III. The borehole locations are shown in Figure 2.

Table III: Summary of Borehole Drilling Program

Borehole No. (BH)	Approximate Borehole Location	Borehole Auger Refusal and Termination Depths Below Existing Grade (Elevations), m
1	Parcel of Land North of Block 211 – within the footprint of the proposed building and crevasse	11.4 (41.7) (Auger Refusal)
2	Parcel of Land North of Block 211	11.5 (40.7) (Terminated)
3	Parcel of Land North of Block 211 – within the footprint of the proposed building	11.7 (40.7) (Terminated)
4	Block 211 - within the footprint of the proposed building	5.5 (47.0) (Terminated)
5	Block 211 - within the footprint of the proposed building	16.2 (36.5) (Terminated)
6	Block 211 - within the footprint of the proposed building	11.7 (41.5) (Terminated)
7	Block 211 – within the footprint of the proposed building and crevasse	17.8 (35.9) (Terminated)
8	Block 211 – south wall of proposed building	11.6 (41.7) (Terminated)

The borehole locations and 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-75 truck-mounted drill rig equipped with continuous flight hollow-stem auger equipment and rock coring capabilities. Standard penetration tests (SPTs) was performed in all the boreholes on a continuous basis to 0.75 m depth interval. 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 from 0.0 m to 0.6 m depths in seven (7) of the eight (8) boreholes. The presence of the bedrock was proven in seven (7) of the eight (8) 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.

All boreholes with the exception of Borehole No. 4 were equipped with either 32 mm or 51 mm diameter PVC monitoring well (with slotted section), for long-term monitoring of the groundwater levels and sampling of the groundwater. The installation configuration of each monitoring well is documented on the respective borehole log.

5.2 Laboratory Testing Program

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

The soil samples and rock cores were visually examined in the laboratory by a senior geotechnical engineer. The soil samples were classified in accordance with the Unified Soil Classification System (USCS). 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 sample and rock core laboratory testing program is shown in Table IV. The laboratory testing program for selected soil samples and rock cores were undertaken in accordance with ASTM.

Table IV: Summary of Laboratory Testing Program

Type of Test	Number of Tests Completed
Soil Samples	
Moisture Content Determination	53
Unit Weight Determination	1
Grain Size Analysis	4
Bedrock Cores	
Unit Weight Determination	21
Unconfined Compressive Strength Test	21
Corrosion Analysis (pH, sulphate, chloride and electrical resistivity)	5

5.3 Hydraulic Conductivity Testing

The procedure for the hydraulic conductivity testing in selected boreholes is discussed in the previously referenced EXP hydrogeological investigation report.

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

A multi-channel analysis of surface waves (MASW) survey was conduced on site on May 3, 2019 by Geophysics (GPR) International Inc. The MASW survey consists of two (2) survey lines across the site. The purpose of the MASW survey is to measure the shear wave velocity at the site, determine the site classification for seismic site response based on the shear wave velocity and attempt to better delineate the slope profiles of the crevasse crossing the site.

6 Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface conditions and groundwater levels from the boreholes are given on the attached Borehole Logs, Figure Nos. 3 to 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.

6.1 Fill

Fill was encountered at ground surface in the eight (8) boreholes and extends to depths ranging from 0.6 m to 11.0 m below existing grade (Elevation 52.6 m to 42.7 m). The deepest fill was encountered in the boreholes located within the crevasse, i.e. Borehole Nos. 1 and 7. It is not known if the fill in Borehole No. 1 extends below the auger refusal of 11.4 m depth (Elevation 41.7 m), since it is not known if auger refusal was met on debris within the fill, a boulder or on bedrock. In the remaining boreholes not located in the crevasse, the fill depth is shallower; 0.6 m to 4.0 m depths below existing grade (Elevation 52.6 m to 48.5 m).

The fill consists of a heterogeneous mixture of gravel, sand, silt and clay in varying amounts with organics, cobbles and boulders. The fill contains varying amounts of asphalt, rubber and tar-like debris in addition to brick, cinders and wood debris. In Borehole No. 7, the fill contains an approximate 75 mm thick peat layer at 4.8 m depth (Elevation 48.9 m). The fill has reddish brown and black stains and exhibits tar-like odour at localized depths.

Based on the SPT N-values of 1 to 83, the fill is in a very loose to very dense state. In some boreholes, the SPT N-value is 50 for less than 300 mm sampler penetration, suggesting that the sampler may have contacted debris, cobble or boulder within the fill. The natural moisture content of the fill ranges from 2 percent to 44 percent. The high moisture content of 161 percent at 10.7 m depth (Elevation 42.4 m) in Borehole No. 1 may be a result of decayed wood debris observed at this depth. The natural unit weight of the fill is 20.6 kN/m^3.

Grain size analysis was conducted on four (4) samples of the fill and the results are summarized in Table V. The grain size distribution curves are shown in Figures 11 to 14.

Table V: Summary of Results from Grain-size Analysis – Fill Samples

Borehole No. - Sample No.	Depth (m)	Grain-size Analysis (%)			Soil Classification (USCS)
		Gravel	Sand	Fines (Silt and Clay)	
BH 3 – SS3	1.5-2.1	18	59	23	Silty Sand with Gravel (SM)
BH 5 – AS1	0.0-0.6	22	59	19	Silty Sand with Gravel (SM)
BH 7 – SS3	1.2-1.8	34	50	16	Silty Sand with Gravel (SM)
BH 8 – AS1	0.0-0.6	23	67	10	Poorly Graded Sand with Silt and Gravel (SP-SM)

Based on a review of the results from the grain size analysis, the fill may be classified as a silty sand with gravel (SM) to poorly graded sand with silt and gravel (SP-SM) in accordance with the Unified Soil Classification System (USCS).

6.2 Glacial Till

The fill in Borehole No. 7 is underlain by glacial till that extends from 11.0 m to 13.0 m depths (Elevation 42.7 m to 40.7 m). The glacial till consists of silty sand and gravel with cobbles and boulders and organic-like stains. The SPT N- values are 24 and 26 indicating the glacial till is in a compact state. The natural moisture content of the glacial till is 15 percent.

6.3 Possible Concrete, Boulders or Weathered Bedrock

Possible concrete, boulders or weathered bedrock exist beneath the fill in Borehole Nos. 5 and 6 located within the footprint of the proposed building. The possible concrete, boulders or weathered bedrock was contacted at 0.6 m and 2.3 m depths (Elevation 52.6 m and 50.4 m) and extends to 1.5 m and 3.1 m depths (Elevation 51.7 m and 49.6 m) in Borehole Nos. 6 and 5, respectively.

6.4 Limestone Bedrock

Auger refusal was met in Borehole Nos. 1 to 4, 7 and 8 beneath the fill and glacial till and beneath the possible concrete, boulders or weathered bedrock in Borehole Nos. 5 and 6 at 1.5 m to 13.0 m depths (Elevation 51.8 m to 40.7 m).

Conventional core drilling techniques were used to advance Borehole Nos. 2 to 8 beyond the refusal depths to termination depths of 5.5 m to 17. 8 m (Elevation 47.0 m to 35.9 m), confirming that auger refusal was met on bedrock. In Borehole No. 1, it is not known whether refusal was met on debris, boulders or bedrock.

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

Table VI: Summary of Auger Refusal and Bedrock Depths (Elevations) in Boreholes

Borehole No.	Approximate Borehole Location	Ground Surface Elevation (m)	Auger Refusal Depth (Elevation) on Inferred Debris, Boulders or Bedrock (m)	Bedrock Depth (Elevation) (m)
1	Parcel of Land North of Block 211 – within the footprint of the proposed building and crevasse	53.06	11.4 (41.7)	N/A
2	Parcel of Land North of Block 211	52.16	3.2 (49.0)	3.2 (49.0)
3	Parcel of Land North of Block 211 -within the footprint of the proposed building	52.39	3.0 (49.4)	3.0 (49.4)
4	Block 211 - within the footprint of the proposed building	52.50	4.0 (48.5)	4.0 (48.5)
5	Block 211 - within the footprint of the proposed building	52.66	3.1 (49.6)	3.1 (49.6)
6	Block 211 - within the footprint of the proposed building	53.19	1.5 (51.7)	1.5 (51.7)
7	Block 211 – within the footprint of the proposed building and crevasse	53.69	13.0 (40.7)	13.0 (40.7)
8	Block 211 – south wall of proposed building	53.31	1.5 (51.8)	1.5 (51.8)

The information from the five (5) boreholes completed previously by DST Consulting Engineers Ltd. (DST) and included in the previously referenced 2014 and 2015 Paterson Group Inc. reports are summarized in Table VII. The borehole logs from DST are shown in Appendix A.

Table VII: Summary of Auger Refusal and Bedrock Depths (Elevations) in Boreholes By Others

Borehole/Monitoring Well No.	Approximate Borehole Location	Ground Surface Elevation (m)	Bedrock Depth (Elevation) (m)
BHMW 16	Parcel of Land North of Block 211	51.81	7.0 (44.8)
BHMW 17	Parcel of Land North of Block 211 – north wall of proposed building	52.43	3.4 (49.0)
BHMW 18	Parcel of Land North of Block 211 – northeast corner of proposed building	53.26	6.7 (46.6)
BHMW 19	Block 211 – within the footprint of proposed building	53.31	3.0 (50.3)
BHMW 26	Block 211 – Head Street south of proposed building	52.94	0.5 (52.4)

A review of the above tables indicates that within Block 211, limestone bedrock was contacted at 0.5 m to 4.0 m depths (Elevation 52.4 m to 48.5 m). In EXP Borehole No. 7 located within the footprint of the crevasse in Block 211, the bedrock surface was contacted at a greater depth of 13.0 m (Elevation 40.7 m). EXP Borehole No. 1 located within the former crevasse, met auger refusal at an 11.4 m depth (Elevation 41.7 m) on debris within the fill, a boulder or on bedrock.

In the parcel of land situated north of Block 211, limestone bedrock surface was contacted at 3.0 m to 7.0 m depths (Elevation 49.4 m to 44.8 m).

The limestone bedrock contains shaley partings and seams, mud and fractured rock seams. The Total Core Recovery (TCR) of the bedrock is 82 percent to 100 percent. The Rock Quality Designation (RQD) of 10 percent to 58 percent in the upper 1.2 m of the bedrock in Borehole Nos. 2, 3 and 6 indicates the rock quality is very poor to fair. The RQD values below the 1.2 m depth from the bedrock surface in Borehole Nos. 2, 3, and 6 and from the bedrock surface in the remaining boreholes, Borehole Nos. 4,5, 7 and 8, ranges from 62 percent to 100 percent indicating the bedrock has a fair to excellent quality. Locally in Borehole No. 7, at a 14.5 m to 16.3 m depth (Elevation 39.2 m to 37.4 m), the bedrock has an RQD value of 42 percent and is of a poor quality. Photographs of the rock cores are shown in Appendix B.

Observations during coring of the bedrock and examination of the bedrock cores in the laboratory did not indicate any evidence of faults.

Unit weight determination and unconfined compressive strength tests were conducted on twenty-one (21) rock core sections and the results are summarized in Table VIII. A review of the test results indicates the strength of the rock may be classified as strong to very strong in accordance with the Canadian Foundation Engineering Manual (CFEM), Fourth Edition, 2006.

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

Borehole No.- Run No.	Depth (Elevation) (m)	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)	Classification of Rock with respect to Strength (1)
BH 4 – Run1	4.7 – 4.8	26.3	147.9	Very Strong
BH 5 – Run 1	3.8 – 4.0	26.0	134.9	Very Strong
BH 5 -Run 2	4.8 – 4.9	26.3	144.6	Very Strong
BH 5 – Run 3	6.3 – 6.4	26.3	134.8	Very Strong
BH 5 - Run 4	7.6 – 7.8	26.2	131.2	Very Strong
BH 5 – Run 5	9.2 – 9.3	26.1	129.9	Very Strong
BH 5 – Run 6	10.6 – 10.7	26.2	130.3	Very Strong
BH 5 – Run 7	12.2 – 12.4	26.2	85.7	Strong
BH 5 – Run 8	13.9 – 14.1	26.2	138.1	Very Strong
BH 5 - Run 9	15.0 – 15.1	26.2	137.5	Very Strong
BH 6 - Run 1	2.3 – 2.4	26.0	85.0	Strong
BH 6 - Run 2	3.3 – 3.4	26.2	160.9	Very Strong
BH 6 – Run 4	6.1 – 6.2	25.8	115.3	Very Strong
BH 6 – Run 7	10.6 – 10.7	23.9	155.7	Very Strong
BH 7 – Run 1	13.8 – 14.0	26.1	85.7	Strong
BH 7 -Run 2	15.5 – 15.7	26.3	152.8	Very Strong
BH 7 – Run 3	17.0 – 17.2	23.8	92.9	Strong
BH 8 – Run 1	1.9 – 2.1	26.1	103.8	Very Strong
BH 8 – Run 2	3.5 – 3.6	26.1	93.1	Strong
BH 8 – Run 4	6.2 – 6.3	26.0	185.7	Very Strong
BH 8 – Run 6	9.6 – 9.8	23.7	126.1	Very Strong

Note:

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

6.5 Groundwater Level Measurements

Six (6) sets of groundwater level measurements were taken in the boreholes equipped with monitoring wells on March 5 and 6, May 13,22, 29 and 30, 2019. A summary of the of groundwater level measurements is shown in Table IX.

Table IX: Summary of Groundwater Level Measurements

Borehole No. (BH)	Location of Borehole	Ground Surface Elevation (m)	Date of Measurement (elapsed time in days from date of installation)	Groundwater Depth Below Ground Surface (Elevation), m	Groundwater Located Within Fill Soil or Bedrock
1	Parcel of Land North of Block 211 – within the footprint of the crevasse	53.06	March 5, 2019 (12 days)	9.9 (43.2)	Fill
			March 14, 2019 (21 days)	10.6 (42.5)	Fill
			May 13, 2019 (81 days)	7.1 (46.0)	Fill
			May 22, 2019 (90 days)	7.8 (45.3)	Fill
			May 29, 2019 (97 days)	8.1 (45.0)	Fill
			May 30, 2019 (98 days)	8.1 (45.0)	Fill
2	Parcel of Land North of Block 211	52.16	March 5, 2019 (11 days)	7.2 (45.0)	Bedrock
			May 13, 2019 (80 days)	6.4 (45.8)	Bedrock
			May 22, 2019 (89 days)	6.8 (45.4)	Bedrock
			May 29, 2019 (96 days)	Monitoring Well Decommissioned on May 29, 2019	N/A
			May 30, 2019 (97 days)	Monitoring Well Decommissioned on May 29, 2019	N/A
3	Parcel of Land North of Block 211	52.39	March 5, 2019 (11 days)	7.4 (45.0)	Bedrock
			May 13, 2019 (80 days)	6.3 (46.0)	Bedrock
			May 22, 2019 (90 days)	6.7 (45.7)	Bedrock
			May 29, 2019 (97 days)	6.7 (45.7)	Bedrock
			May 30, 2019 (98 days)	6.7 (45.7)	Bedrock

Table IX: Summary of Groundwater Level Measurements

Borehole No. (BH)	Location of Borehole	Ground Surface Elevation (m)	Date of Measurement (elapsed time in days from date of installation)	Groundwater Depth Below Ground Surface (Elevation), m	Groundwater Located Within Fill Soil or Bedrock
5	Block 211 - within the footprint of the proposed office building	52.66	March 6, 2019 (12 days)	7.5 (45.2)	Bedrock
			May 13, 2019 (82 days)	Could not Find Monitoring Well (possibly located under a fill stockpile)	N/A
			May 22, 2019 (91 days)	Could not Find Monitoring Well (possibly located under a fill stockpile)	N/A
			May 29, 2019 (98 days)	Could not Find Monitoring Well (possibly located under a fill stockpile)	N/A
			May 30, 2019 (99 days)	Could not Find Monitoring Well (possibly located under a fill stockpile)	N/A
6	Block 211 - within the footprint of the proposed office building	53.19	March 5, 2019 (8 days)	7.6 (45.6)	Bedrock
			May 13, 2019 (77 days)	6.3 (46.9)	Bedrock
			May 22,2019 (86 days)	6.2 (47.0)	Bedrock
			May 29, 2019 (93 days)	6.2 (47.0)	Bedrock
			May 30, 2019 (94 days)	6.2 (47.0)	Bedrock

Table IX: Summary of Groundwater Level Measurements

Borehole No. (BH)	Location of Borehole	Ground Surface Elevation (m)	Date of Measurement (elapsed time in days from date of installation)	Groundwater Depth Below Ground Surface (Elevation), m	Groundwater Located Within Fill Soil or Bedrock
7	Block 211 – within the footprint of the crevasse	53.69	March 6, 2019 (14 days)	10.6 (43.1)	Fill
			May 13, 2019 (82 days)	7.8 (45.9)	Fill
			May 22, 2019 (91 days)	8.5 (45.2)	Fill
			May 29, 2019 (98 days)	8.7 (45.0)	Fill
			May 30, 2019 (99 days)	8.7 (45.0)	Fill
8	Block 211	53.31	March 6, 2019 (9 days)	4.5 (48.8)	Bedrock
			May 13, 2019 (77 days)	9.4 (44.0)	Bedrock
			May 22, 2019 (86 days)	9.2 (44.2)	Bedrock
			May 29, 2019 (93 days)	9.0 (44.3)	Bedrock
			May 30, 2019 (94 days)	9.0 (44.3)	Bedrock

The first set of groundwater level measurements taken on March 5 and 6, 2019 are not considered to be stabilized groundwater levels, since this set of measurements was taken shortly after the completion of the boreholes.

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.

7 Design Considerations

Based on an overall review of the borehole information, the following geotechnical aspects should be considered for the design and construction of the proposed new multi-storey office building with two (2) level underground parking garage at Block 211 and north of Block 211:

1. Based on the results of the MASW survey, the site may be classified for seismic site response as **Class A** for foundations founded on the bedrock.
2. It is our understanding that the proposed building development at the site will include reclamation of land north of the top of the bank along the Ottawa River. The design of the proposed building should take into consideration the final location of the boundary of the reclaimed land along the Ottawa River.
3. The top of the lowest slab of the proposed building will be at Elevation 44.5 m and the maximum depth of the bottom of the excavation for the proposed building will be at Elevation 42.0 m. The excavation will be within the bedrock and below the groundwater level and the water level in the Ottawa River. Due to the presence of the groundwater level within the overburden soil of the buried deep crevasse that extends into the proposed building footprint from the Ottawa River and the close proximity of the site to the Ottawa River, significant groundwater flows into the excavation and the newly constructed building may occur during construction and in the long term over the life of the proposed building.

   The proposed building may be supported by conventional spread and strip footings founded on the sound bedrock with the lowest slab of the building designed as a slab on grade, provided the building is equipped with permanent perimeter and underfloor drainage systems designed to handle the estimated groundwater flow of 150 m^3/day presented in the EXP hydrogeological investigation report.

   The hydrogeological investigation prepared by EXP indicates the groundwater flow into the excavation is estimated at 360 m^3/day. To control the estimated groundwater flows into the excavation and the proposed building, it is recommended that a permanent groundwater barrier system of very low permeability (coefficient of permeability of 10×10^{-10} m/s), such as a clay berm or equivalent, or a groundwater cut-off system such as a secant pile wall shoring system or equivalent be constructed along the full length of the north wall. The permanent groundwater barrier or cut off wall systems will minimize groundwater flows into the excavation and the proposed building from the buried deep crevasse and the Ottawa River. A contingency should be made for extending the barrier wall or cut-off wall system in the south direction at the east and west ends of the north wall, in case groundwater cannot be effectively controlled by the north barrier system or cut off wall alone. The length of the north barrier system or cut-off wall may be shortened and the need for extending the barrier system or wall in the south direction may be reduced or eliminated, if the excavation east and west of the crevasse reveals the surface of the bedrock is higher, the bedrock quality is good and groundwater control does not appear to be problematic. The secant pile wall shoring system should be socketed approximately 1 m into the bedrock and the elevation of the top of the secant pile wall system should match that of the regulatory flood level in the Ottawa River of Elevation of 49.0 m to 49. 5 m.

The permanent groundwater barrier system or cut-off wall system must be designed and installed by an experienced contractor on a performance and end results type of specification.

Along the east and west sides of the excavation beyond the groundwater barrier system or secant pile wall shoring system and along the south side of the excavation, the side slopes of the overburden soil may be cut back in accordance with the Occupational Health and Safety Act (OHSA). If the required side slopes cannot be achieved due to space restrictions on site, the soil may be supported by a shoring system that is socketed into the bedrock. If movement of the shoring system, adjacent structures and infrastructure can be tolerated along the east, west and south sides of the excavation, the excavation side slopes within the overburden soil may be supported by a soldier pile and timber lagging shoring system installed to the bedrock surface. Surface water flows into the excavation, that may be significant during precipitation events, may occur that will require removal from the excavation by sump pumping techniques and large capacity pumps. The feasibility of using a soldier pile timber lagging system along the east side of the excavation will have to be assessed by conducting an additional geotechnical investigation to determine the subsurface conditions in the east half of the proposed building location that was inaccessible during this geotechnical investigation.

Excavation within the bedrock below the bottom of the secant pile wall system and soldier pile timber lagging shoring system may be undertaken with near vertical sides subject to review by a geotechnical engineer. The exposed rock face in the excavation will require a support system consisting of rock bolts to maintain the integrity of the exposed rock face. Significant groundwater flows through the rock face into the excavation may occur in localized areas and the flows can be reduced by grouting permeable seepage zones along the rock face and removing any water that enters the excavation by sump pumping method.

For the spread and strip footing design to support the building and slab-on-grade design of the lowest floor slab of the proposed building, permanent perimeter and underfloor drainage systems are required and should be designed to handle the estimated groundwater flow of 150 m^3/day presented in the EXP hydrogeological report. Since this building design relies completely on the satisfactory performance of the building drainage systems, it is crucial that the permanent perimeter and underfloor drainage systems operate on a continuous basis. The perimeter and underfloor drainage systems should be connected to separate sumps and the design should include back up pumps and generators, in case of mechanical failure and/or power outage.

4. An additional geotechnical investigation consisting of boreholes should be undertaken in the east half of the footprint of the proposed building which could not be accessed by a drill rig during this geotechnical investigation and within the south section of the crevasse. Test pits should be excavated at the foundations of existing adjacent buildings to determine the founding level of the foundations for underpinning and shoring requirements.

The above items are discussed in detail in the following sections of this report.

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

8.1 Site Classification for Seismic Site Response

The MASW survey report is shown in Appendix C. The results of the MASW survey indicates that the average seismic shear wave velocity is 1870 m/s. Based on a review of Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC), for an average shear wave velocity of 1870 m/s and for foundations founded directly on the sound bedrock surface, the site may be classified as **Class A** for seismic site response.

Delineation of the rock slopes of the filled in crevasse could not be determined due to disturbance from background noise during the MASW survey.

8.2 Liquefaction Potential of Soils

It is anticipated that all subsurface soil on site including the fill and underlying glacial till will be excavated and removed from site as part of the environmental remediation. Therefore, in this case, liquefaction of the soils will not apply.

9 Grade Raise Restrictions

Significant grade raise is not anticipated for the proposed building development. Also, it is anticipated that all the existing soil comprising of fill and glacial till will be removed down to the bedrock as part of the proposed construction. Therefore, grade raise is not a concern at the subject site. However, EXP should be consulted to review the final grading plan.

10 Foundation Considerations

10.1 Footings

Spread and strip footings founded on the sound bedrock, competent and free of soil filled seams may be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 3000 kPa and 5000 kPa depending on the amount of inspection and testing undertaken during construction. The factored ULS value includes a resistance factor of 0.5. Factored geotechnical resistance at ULS of 3000 kPa would require only visual inspection of the footing beds. The use of factored geotechnical resistance at ULS of 5000 kPa would require star drilling and probing of all the spread and strip footings (minimum 50 mm diameter hole may be used, with its depth equal to at least twice the footing width). The strip footings should be star drilled and probed at 3 m intervals. 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 presence of the soil-filled crevasse at the footing design founding level should be anticipated along the north side of the building. In this the case, the footings in these areas may have to be excavated to a deeper depth than the design founding level to reach the bedrock and the excavation will have to be backfilled with concrete to the design founding level. The concrete should have a compressive strength of 30 MPa.

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

10.2 Additional Comments - Foundations

An additional geotechnical investigation consisting of boreholes should be undertaken in the east half of the footprint of the proposed building which could not be accessed by a drill rig during this geotechnical investigation and within the south section of the crevasse.

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

10.3 Sliding Resistance

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

11 Floor Slab and Drainage Requirements

The lowest level floor may be constructed as a slab-on-grade provided it is set on a bed of well compacted bed of 19 mm clear stone at least 300 mm thick placed on the approved bedrock subgrade. The clear stone would prevent the capillary rise of moisture 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 proposed building. 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.

For the spread and strip footing design to support the building and slab-on-grade design of the lowest floor slab of the proposed building, permanent perimeter and underfloor drainage systems are required and should be designed to handle the estimated groundwater flow of 150 m^3/day as indicated in the EXP hydrogeological investigation report. Since this building design relies completely on the satisfactory performance of the building drainage systems, it is crucial that the permanent perimeter and underfloor drainage systems operate on a continuous basis. The perimeter and underfloor drainage systems should be connected to separate sumps and the design should include back up pumps and generators, in case of mechanical failure and/or power outage.

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

12 Lateral Earth and Water Pressures Against Subsurface Walls

12.1 Building with Permanent Drainage Systems

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) below 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 applied at the ground surface should be considered as surcharge when computing lateral pressure using equation (ii).

Lateral static earth pressure, p:

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

where

k = lateral earth pressure coefficient for ‘at rest’ condition = 0.50

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

h = depth of interest below ground surface (m)

q = any surcharge acting at ground surface (kPa)

Lateral static earth pressure 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 earth force (dynamic thrust) due to seismic loading may be computed from the equation given below:

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

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

h = height of basement wall against soil above the bedrock surface (m)

γ = unit weight of soil = 22 kN/m^3

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

F_b = thrust factor = 1.0

The dynamic thrust acts approximately at 0.63h.

All subsurface walls should be waterproofed.

12.2 Lateral Earth Pressures Due to Subsurface Walls Cast Directly Against Bedrock

Lateral stress relief may occur in the limestone after excavation, resulting in horizontal movement of the bedrock towards the excavation. Experience indicates that the stress relief movements increase as depth of the excavation increases and that the distance that movements may occur from the excavation is dependent on the geometry of the excavation.

A simple method to mitigate the effects of rock squeeze, if any, is to delay the installation of the subsurface walls until sufficient rock deformation has occurred, so that any further deformation after construction is within manageable limits. Lo et al. (1987) suggested that for a construction period of two months, approximately 50 percent stress reduction can be achieved. Alternatively, the subsurface walls of the proposed structure cast directly against rock may be designed in one of the two ways:

1.) Provide adequate cushion between the subsurface walls and the rock face to accommodate movement of the rock due to stress relief and prevent stressing of the subsurface walls; or

2.) Design the subsurface walls for lateral pressure that may be exerted by the rock.

It is considered that a 50 mm thick compressible rigid insulation material (such as Styrofoam Brand HI-60 or equivalent)) may be provided between the rock face and the subsurface wall to provide stress relief in case there is any squeezing of the rock towards the structure. It is noted that this precautionary measure was recommended by EXP for local projects and has been recommended by others on some projects in the Toronto area (Lo et al. 1987).

The alternative to providing a compressible foam material cushion to relieve the stress generated by any rock movement towards the structure, is to design the subsurface walls to withstand this pressure. We recommend that a horizontal pressure of 0.5 MPa may be used in the design of the subsurface walls.

For basement walls cast directly against bedrock, a vertical drainage membrane or board such as Terradrain 200 or equivalent should be installed on the face of the bedrock that leads to a solid discharge pipe connecting to a sump. The top of the drainage board should be covered with a fabric filter to prevent the loss of overlying soil into the drainage board. For guidance, reference is made to Figure 15 regarding the vertical drainage board against a soldier pile and timber lagging shoring system that is supporting the adjacent soil. The drainage board against the bedrock face should be installed in a similar manner as is shown in Figure 15.

The subsurface (basement) walls should be waterproofed to prevent seepage of water into the underground parking levels of the building.

13 Excavations and De-Watering Requirements

13.1 Excavations

It is anticipated that with the proposed two (2) level underground parking garage, the excavation depth will be 10 m to 11 m below existing grade. The excavation will terminate within the limestone bedrock and be below the measured groundwater levels.

All shoring systems should be designed in accordance with the Occupational Health and Safety Act (OHSA) and the 2006 Canadian Foundation Engineering Manual (Fourth Edition). The design and installation of the shoring systems should be the responsibility of the shoring designer and contractor.

Details regarding soil and rock excavations and shoring systems are discussed in the following sections of this report.

13.1.1 Overburden Soil Excavation

The excavations in the soil may be undertaken by conventional equipment capable of removing possible debris within the fill, buried concrete structures, cobbles, boulders and large rock pieces within the fill and underlying native soils.

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. Below the groundwater level, the subsurface soils are anticipated to slough and eventually stabilize at a slope ranging from 2H:1V to 3H:1V. If the above side slopes cannot be achieved due to space restrictions on site, the excavation would have to be undertaken within the confines of an engineered support system.

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

- Proximity of the excavation to existing structures. It is anticipated that the north side of the excavation will be next to the vacant land north of the Block 211 and will cross the existing crevasse where the bedrock depth is deep at 13.0 m (Elevation 40.7 m). The east and south sides of the excavation are expected to be next to existing buildings, underground structures and underground municipal services. The west side of the excavation will be next to Booth Street and will include the roadway and underground services.
- Type of foundations of the existing buildings and the difference in founding levels between the foundations of new structures and existing structures; and
- The geotechnical and groundwater conditions encountered.

A conventional shoring system consisting of soldier pile and timber lagging is more flexible compared to the secant pile shoring system but does not provide a positive cut-off to groundwater flows into the excavation. In areas where there is a concern for settlement of the existing adjacent structures, lateral yielding of the soils and to provide a positive cut-off to groundwater seepage, the use of secant pile system is recommended. In areas where the potential of settlement of the nearby structures is low, soldier pile and timber lagging system may be used. However, the soldier pile and timber lagging does not provide a positive cut-off to groundwater seepage.

13.1.1.1 Soldier Pile and Timber Lagging System

The shoring system should be designed in accordance with OHSA. 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 = lateral earth pressure, at any depth, h, below the ground surface
k = applicable earth pressure coefficient
γ = unit weight of soil to be retained, estimated at 22 kN/m^3
h = the depth, in metres, at which pressure, P, is being computed
q = the equivalent surcharge acting on the ground surface adjacent to the shoring (kPa)

The earth pressure coefficient, k, may vary between the following limits:

0.25	where adjacent building footings or settlement-sensitive services lie below a 45-degree line drawn up from the toe of the excavation.
0.35	where adjacent building footings or settlement-sensitive services lie below a 60-degree line to the horizontal drawn up from the toe of the excavation.
0.45	where adjacent building footings or settlement-sensitive services lie above a 60-degree line to the horizontal drawn up from the base of the toe of the excavation.

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 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 0.7 m of rock ledge around the perimeter, 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.

13.1.1.2 Secant Pile Shoring System

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

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

where:

P_0 = ‘at rest’ earth and water thrusts acting against secant pile wall (kN/m)

K_0 = ‘at rest’ lateral earth pressure coefficient = 0.50

q = surcharge acting adjacent to the excavation (kPa)

h_1 = height of shoring from the ground surface to groundwater table (m)

h_2 = height of shoring from groundwater table to the bottom of excavation (m)

γ = unit weight of the soil = 22 kN/m^3

$\gamma^{'}$ = submerged unit weight of soil = 11 kN/m^3

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

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

The shoring systems should be tied back by rock anchors grouted into the sound bedrock. The factored ULS grout to rock bond of 700 kPa may be used for design of the anchors. This value assumes a grout with a minimum strength of 30 MPa is used and that the sides of the drilled holes are cleaned prior to the grouting operation. It is anticipated that the bedrock may contain near vertical seams and some horizontal fractures and therefore some grout loss when grouting anchors in the bedrock should be anticipated. The grout loss is expected to be higher in the fractured bedrock and lower in the sound bedrock. Difficulties may be encountered during the installation of the rock anchors due to the presence of debris (such as brick and wood) and boulders/cobbles within the fill and glacial till soils.

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

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

13.1.2 Rock Excavation

Excavation of the underlying bedrock will require line drilling and blasting techniques and must be undertaken by a specialized blasting contractor.

Rock excavations for the proposed building may be undertaken within exposed near vertical cut rock face walls with rock bolts and a shoring system supporting the overburden soils. The exposed rock face walls will be subject to review by a geotechnical engineer. Considering the close proximity of the existing building, a high quality of excavation support system consisting of rock bolts is required to ensure the integrity of the rock beneath the existing adjacent building foundations and the integrity of the rock face of the excavation. 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 may 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 10.5 of this report.

13.1.3 Vibration Control

Blasting of the bedrock must take in such a manner as not to cause damage to the nearby existing buildings and infrastructure and the secant pile shoring wall for the proposed building.

Blasting should be undertaken in accordance with City of Ottawa Special Provisions S.P. No.: F-1201. The maximum peak particle velocity values provided in the City of Ottawa Special Provisions is shown in Table X.

Table X: Maximum Peak Particle Velocity Values

Element	Frequency (Hz)	Peak Particle Velocity (PPV) (mm/s)
Structures and Pipelines	≤ 40	20
	> 40	50
Concrete and Grout < 72 hours from placement	N/A	10

If the adjacent building and structures are considered to be sensitive or are classified as heritage structures, the following vibration limits for unrestored masonry structures in accordance with DIN 4150 (Table XI below) and for restored masonry structures (Table XII below) in accordance with USBM Z-Curve (USBM R18507) and shown in Figure 16 are recommended.

Table XI: Unrestored Masonry Vibration Limits

Dominant Frequency Range (Hz)	Peak Particle Velocity (mm/s)
< 10	3
10 to 40	3 to 17.5*
> 40	17.5

Note: * on a linear scale relative to dominant frequency range.

Table XII: Restored Masonry Vibration Limits

Dominant Frequency Range (Hz)	Peak Particle Velocity (mm/s)
< 4	< 5.0
4 to 10	< 12.5
10 to 40	12.5 to 50*
> 40	≤50

Note: * on a linear scale relative to dominant frequency range.

13.1.4 Additional Comments

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. Prior to the commencement of blasting, a detailed blast methodology should be submitted by the Contractor. Vibration monitoring be conducted prior to, during and on completion of construction operations.

The shoring system as well as adjacent settlement sensitive structures should be monitored for movement on a periodic basis prior to, during and following construction operations.

It is anticipated that test pit excavations at the site will be required to establish the founding level of foundations of some of the existing adjacent structures for underpinning/shoring requirements.

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.

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

This section of the report should be read in conjunction with the previously referenced hydrogeological investigation report prepared by EXP that provides the estimated groundwater flow into the excavation of 360 m^3/day.

Seepage of water into the shored excavation is anticipated and should be possible to collect the water in the excavation at low points and remove by sump pumping techniques. Significant groundwater flows through localized areas along the rock face into the excavation is anticipated and may be reduced by grouting permeable seepage zones along the rock face and removing any water that enters the excavation by sump pumping method. It is anticipated that continuous pumping may be required to maintain a dry excavation.

The maximum excavation depth at the site will be approximately 11 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.

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

All fill soil and glacial till will be excavated and removed down to the bedrock surface as part of the environmental remediation of the site. These soils are not suitable for re-use as backfill material from a geotechnical point of view.

Therefore, backfill required against foundation walls, in service trenches and as underfloor fill would have to be imported and should conform to Ontario Provincial Standard Specification (OPSS) 1010 requirements for Granular B Type II. The backfill should be placed in 300 mm thick lifts compacted to 95 percent standard Proctor maximum dry density (SPMDD) outside the building and to 98 percent SPMDD inside the building.

For the portion of the basement wall poured against the temporary shoring (such as soldier pile and timber lagging shoring system), vertical drainage board must be installed on the face of the excavation wall or lagging to provide the necessary drainage. A vertical drainage board such as Terradrain 200 or equivalent may be used for this purpose. Reference is made to Figure 15.

For the case where the upper portion of the wall is backfilled with granular material, the vertical drainage membrane should extend up along the backfill to provide drainage of the backfill. The vertical drains could be connected to a solid drain placed inside the building close to the foundation wall and leading to a storm sump in the interior of the building. The lateral pressure acting on the vertical drains may be calculated using a ‘k’-value for at rest condition of 0.50.

For the portion of the foundation wall that extends along the exposed rock face of the excavation and against the face of the secant pile shoring system, the drainage board may be installed in a similar manner as shown in Figure 15.

15 Access Roads and Parking Areas

The subgrade at the site is anticipated to primarily consist of imported granular fill that will be used as backfill following the removal of the existing fill from the site. The imported granular fill will have to be prepared as engineered fill compacted to 95 percent SPMDD. Preliminary pavement structure thicknesses required for the access roads and parking areas set on the prepared engineered fill were computed and are shown on Table XIII.

Table XIII: Recommended Pavement Structure Thicknesses

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

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

The above pavement structures are preliminary in nature and should be reviewed once the final design grades and grading plan are available.

16 Corrosion Potential of Subsurface Bedrock

Chemical tests limited to pH, sulphate, chloride and resistivity were undertaken on five (5) selected bedrock core sections. A summary of the results is shown in Table XIV. The laboratory certificate of analysis is shown in Appendix D.

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

Borehole- Run No.	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
Threshold Values		**<5**	**>0.1**	**>0.04**	
BH 5 -Run 8	13.8 – 13.9	8.63	0.0010	0.0038	5260
BH 5 – Run 9	15.7 – 16.0	8.72	0.0010	0.0029	5430
BH 7 – Run 1	13.7 – 13.8	8.38	0.0067	0.0066	3730
BH 7 - Run 2	15.8 – 16.0	8.51	0.0044	0.0045	4550
BH 7 – Run 3	18.0 – 18.1	8.53	0.0035	0.0039	5920

The results indicate the soils have a negligible chloride and 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 bedrock is considered mildly corrosive to bare steel as per the National Association of Corrosion Engineers (NACE) International. Appropriate measures should be undertaken to protect buried steel elements from corrosion.

17 Additional Studies

An additional geotechnical investigation consisting of boreholes should be undertaken in the east half of the footprint of the proposed building which could not be accessed by a drill rig during this geotechnical investigation and within the south section of the crevasse.

Test pits should be excavated at the foundations of existing adjacent buildings to determine the founding level of the foundations for underpinning and shoring requirements.

18 General Comments

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

The information contained in this report is not intended to reflect on environmental aspects of the soils. Reference is made to the environmental investigations under taken by EXP and reported under separate covers.

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

Figures

BACKGROUND MAP DATA © 2010, MAP ART PUBLISHING CORP.

0 100m 200m 400m
HORIZONTAL 1:10,000

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scale AS NOTED	CLIENT: ZIBI CANADA BLOCK 211 AND PARCEL OF LAND NORTH OF BLOCK 211, CHAUDIÈRE ISLAND, OTTAWA, ON	project no. OTT-00250193-C0
date MARCH 2019		
drawn by M.N.	TITLE: SITE LOCATION PLAN	FIG 1

Filename: p:\projects\geotechnical\250000\250000\250193 c- geo and env block 211\o- drawings\250193-c0 fig 1 loc plan block 211.dwg
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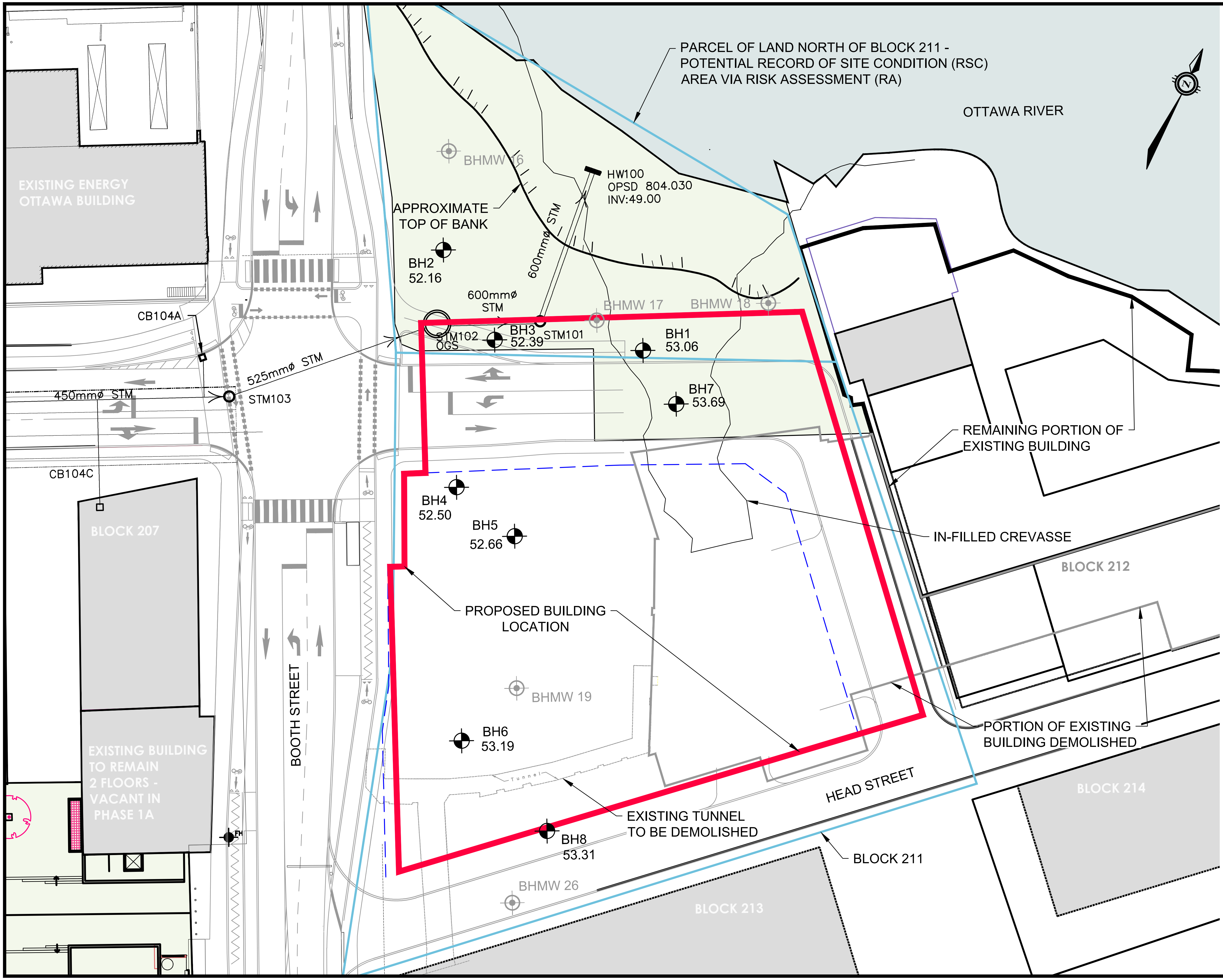

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LEGEND

⊕ **BH1** 53.01	BOREHOLE NUMBER, LOCATION AND ELEVATION
⊕ BHMW 16	BOREHOLE / MONITORING WELL BY DST CONSULTING ENGINEERING INC. INCLUDED IN 2014 AND 2015 PATERSON GROUP INC. REPORTS.

NOTES:

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL 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 ZIBI CANADA

SCALE
0 5m 10m 20m
HORIZONTAL 1:500

CLIENT
ZIBI CANADA

PROJECT
BLOCK 211 AND
PARCEL OF LAND NORTH OF BLOCK 211
CHAUDIÈRE ISLAND, OTTAWA, ON

TITLE
BOREHOLE LOCATION PLAN

date	APRIL 2019	project no.	OTT-00250193-C0
design by	S.P.	drawing no.	FIG 2
prepared by	M.N.		
reviewed by	I.T.		

Filename: p:\projects\geotechnical\250000\250000\250193 c- geo and env block 211\o- drawings\received\2019-05-08 block 211 garage and boundary\250193-c0 fig 2 bh plan block 211 with boundaries.dwg
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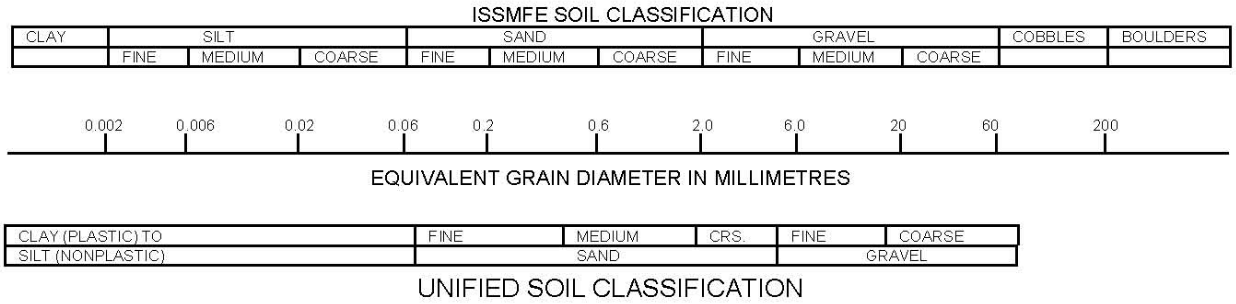
Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Society 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 BH 1

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Location: Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.

Date Drilled: 'February 20 and 21, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic

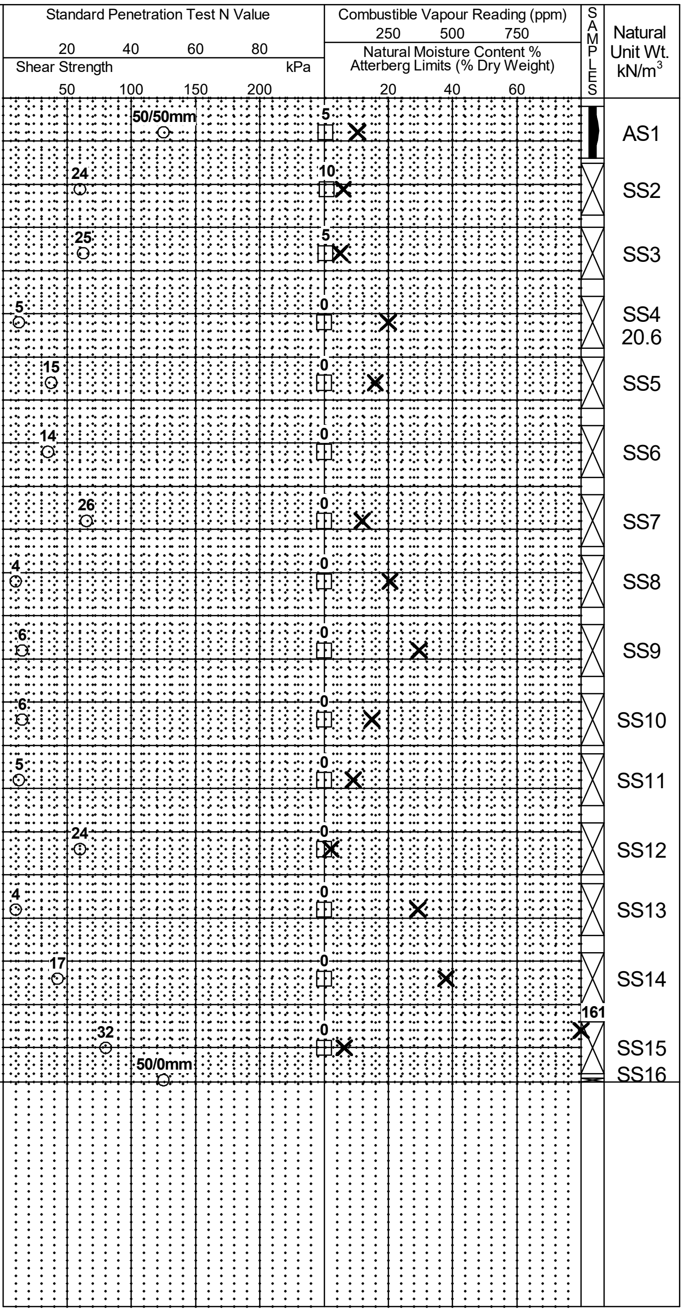
Logged by: M.D Checked by: I.T.

Figure No. 3

Page. 1 of 1

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

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
			53.06	0
		FILL Silty sand with gravel, brown and grey, damp to moist, no odours, no stains, (loose to dense)		
		silty clay and brick debris from 2.3 m to 2.9 m depths		
		tar-like odour and black stains from 3.8 m to 6.7 m depths		
		black stains from 6.9 m to 7.5 m depths		
			44.96	8
		tar-like odour and black stains from 8.4 m to 9.0 m depths		
		brick debris from 9.1 m to 9.7 m depths		
		wood debris and black stains from 9.9 m to 10.9 m depths		
		cobbles, boulders and black stains from 10.7 m to 11.4 m depths		
			41.7	
		Auger Refusal at 11.4 m Depth		



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 51 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
1 day	12.1	-
12 days	9.9	-
81 days	7.1	
90 days	7.8	
97 days	8.1	

CORE DRILLING RECORD

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

Log of Borehole BH 2

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Location: Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.

Figure No. 4

Page. 1 of 1

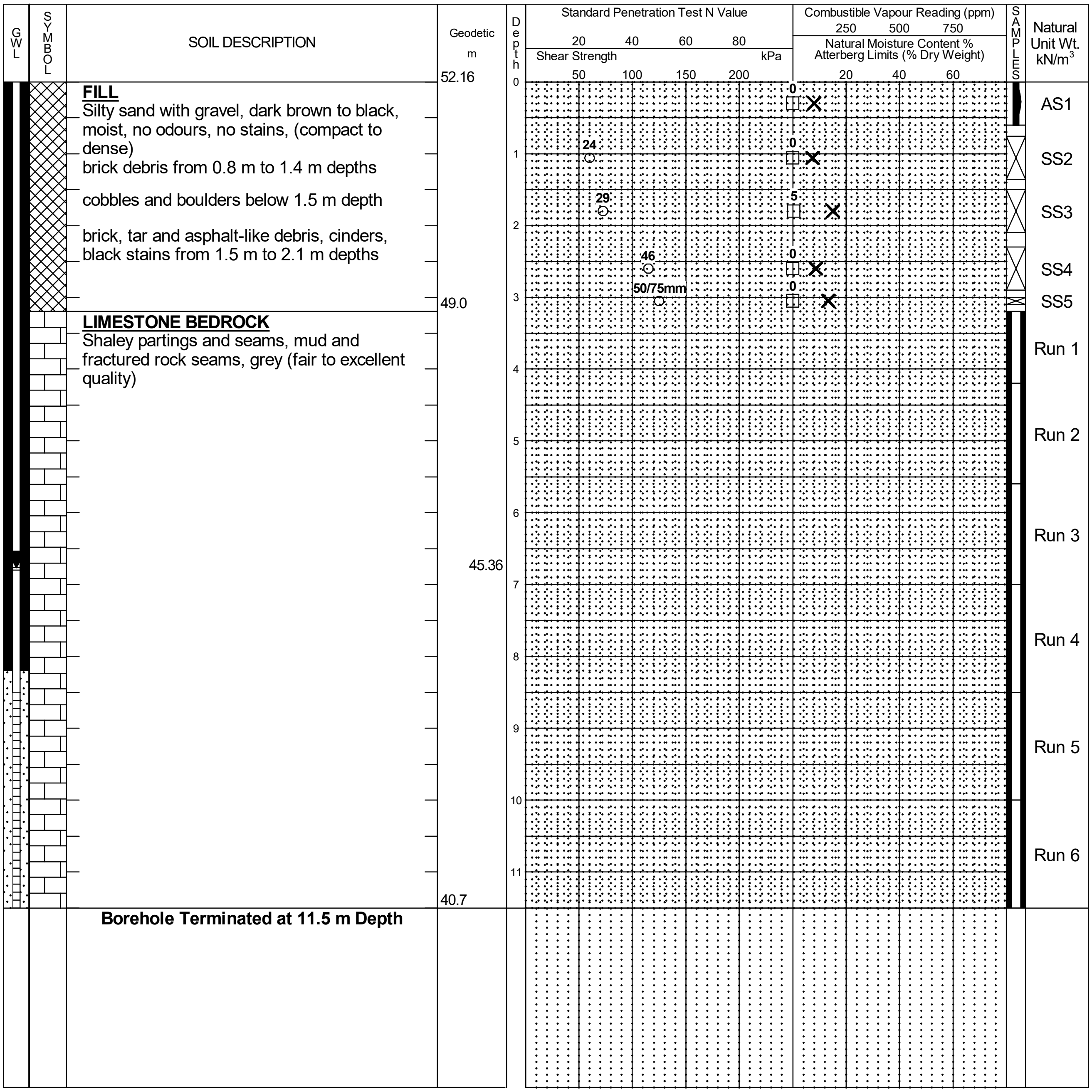
Date Drilled: 'February 21and 22, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic

Logged by: M.D 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	▲



FILL
Silty sand with gravel, dark brown to black, moist, no odours, no stains, (compact to dense)
brick debris from 0.8 m to 1.4 m depths

cobbles and boulders below 1.5 m depth

brick, tar and asphalt-like debris, cinders, black stains from 1.5 m to 2.1 m depths

LIMESTONE BEDROCK
Shaley partings and seams, mud and fractured rock seams, grey (fair to excellent quality)

Borehole Terminated at 11.5 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 32 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
11 days	7.2	-
80 days	6.4	
89 days	6.8	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3.2 - 4.2	89	58
2	4.2 - 5.6	100	87
3	5.6 - 7	100	96
4	7 - 8.5	97	97
5	8.5 - 10	100	91
6	10 - 11.5	100	95

Log of Borehole BH 3

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Location: Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.

Figure No. 5

Page. 1 of 1

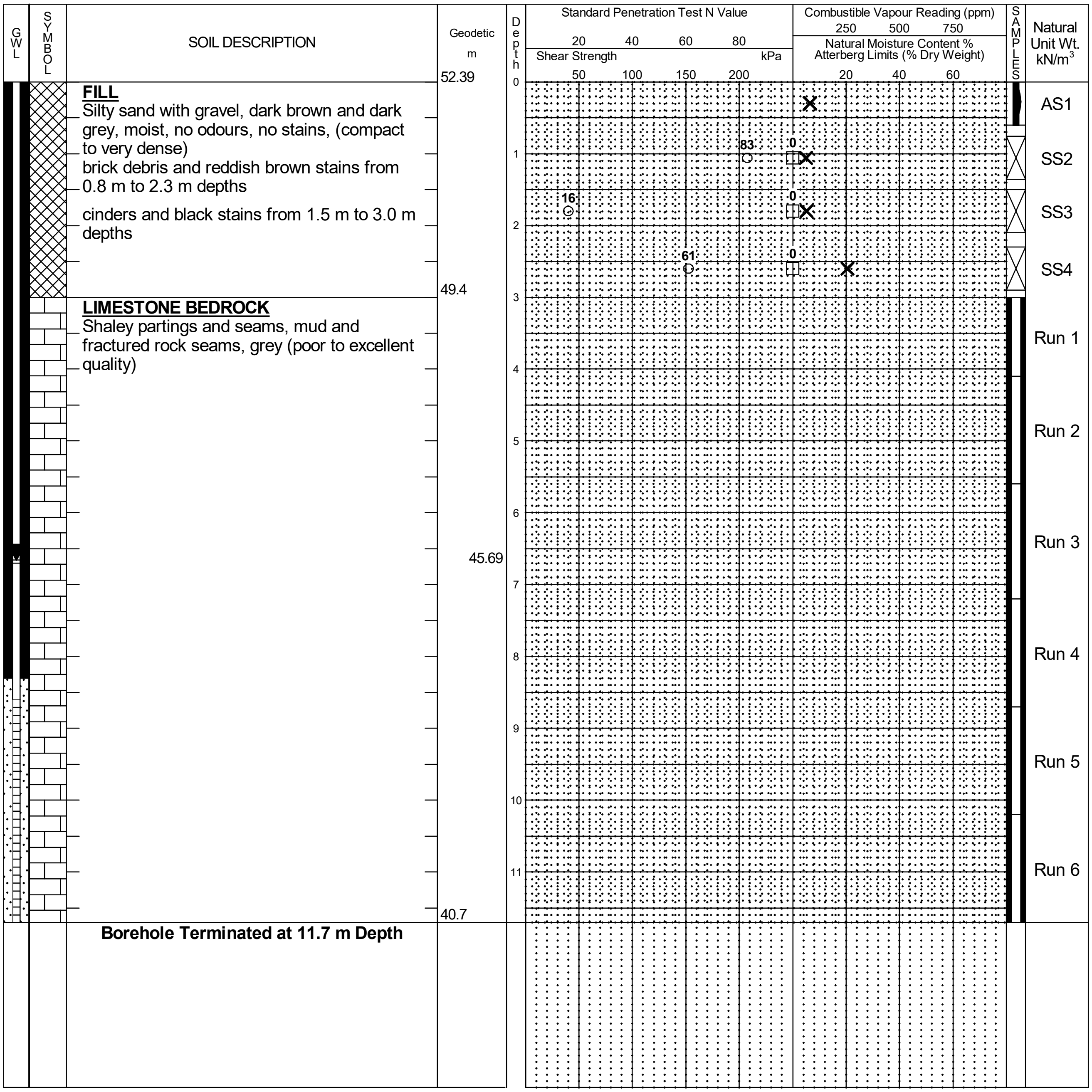
Date Drilled: 'February 21 and 22, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic

Logged by: M.D   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		



FILL
Silty sand with gravel, dark brown and dark grey, moist, no odours, no stains, (compact to very dense)
brick debris and reddish brown stains from 0.8 m to 2.3 m depths

cinders and black stains from 1.5 m to 3.0 m depths

LIMESTONE BEDROCK
Shaley partings and seams, mud and fractured rock seams, grey (poor to excellent quality)

Borehole Terminated at 11.7 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 32 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
11 days	7.4	-
80 days	6.4	-
90 days	6.7	
98 days	6.7	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	3 - 4.1	82	47
2	4.1 - 5.6	100	93
3	5.6 - 7.2	100	92
4	7.2 - 8.7	100	99
5	8.7 - 10.2	100	100
6	10.2 - 11.7	100	100

Log of Borehole BH 4

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Location: Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.

Figure No. 6

Page. 1 of 1

Date Drilled: 'February 21, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic

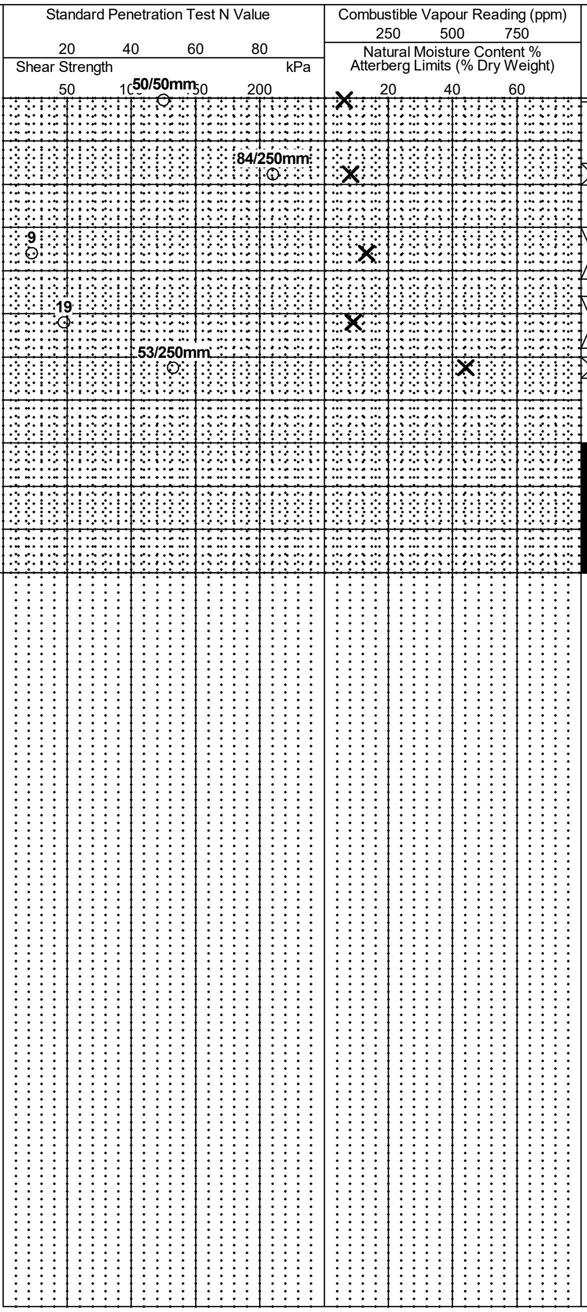
Logged by: M.D

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 m	Depth	SAMPLES	Natural Unit Wt. kN/m^3
		FILL Silty sand with gravel, brown, damp to moist, no odours, no stains, (loose to compact) brick and rubber-like debris, tar-like odour, black stains from 0.8 m to 4.0 m depths	52.5	0	AS1 SS2 SS3 SS4 SS5	
		LIMESTONE BEDROCK Shaley partings and seams, fractured rock seams, grey (fair quality)	48.5	4	Run 1	
			47.0	5		

Borehole Terminated at 5.5 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	N/A	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	4 - 5.5	97	87

Log of Borehole BH 5

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Location: Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.

Date Drilled: 'February 21 and 22, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

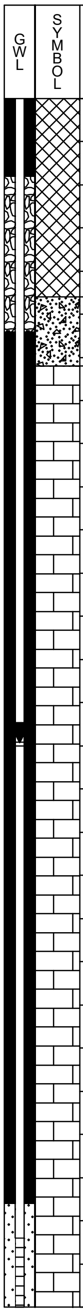
Datum: Geodetic

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

Figure No. 7

Page. 1 of 2

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 m

52.66

FILL
Silty sand with gravel, brown, moist, no odours, no stains, (dense)

brick debris from 0.8 m to 1.4 m depths

difficult augering; cobbles and boulders from 0.8 m to 3.1 m depths

50.4

POSSIBLE CONCRETE, BOULDERS or WEATHERED BEDROCK

49.6

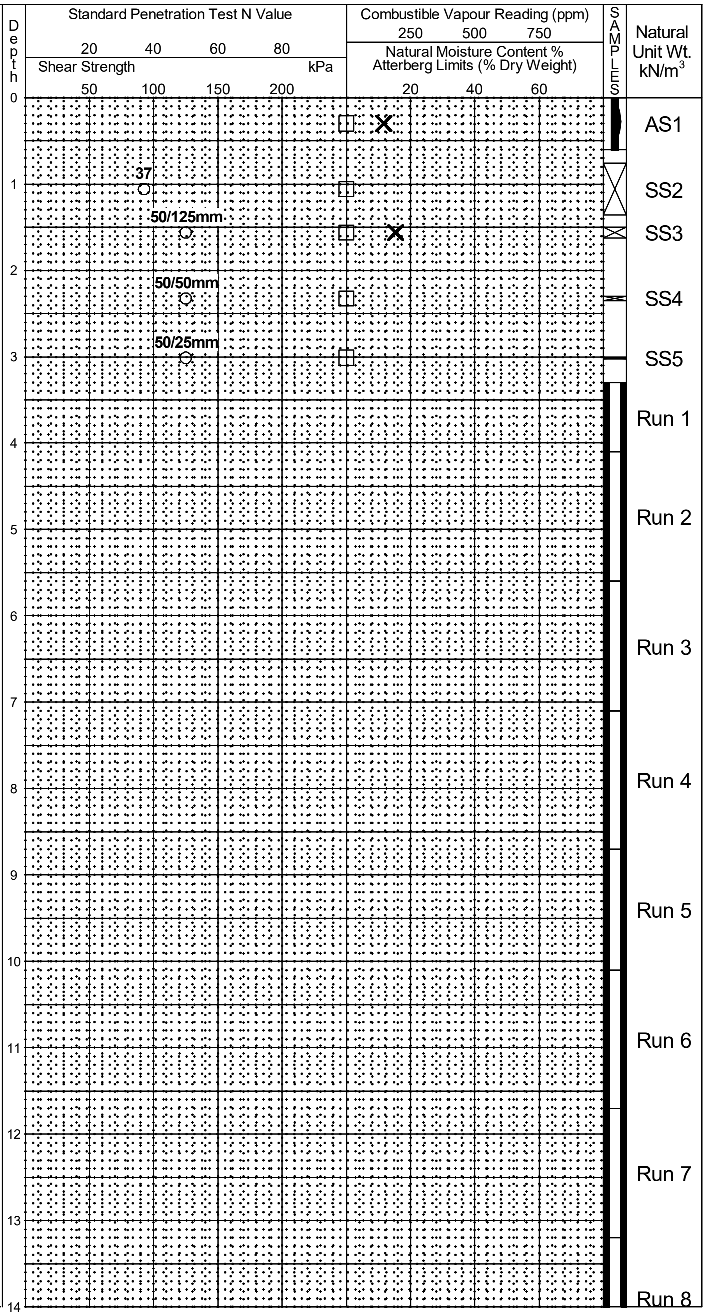
LIMESTONE BEDROCK
Shaley partings and seams, mud and fractured rock seams, grey (good to excellent quality)

Depth (m)	TCR (%)	RQD (%)
3.3-4.1	94	88
4.1-5.6	100	90
5.6-7.1	100	93
7.1-8.7	100	100
8.7-10.1	100	93
10.1-11.7	100	93
11.7-13.2	100	97
13.2-14.6	97	91
14.6-16.2	97	88

45.16

TCR - Total Core Recovery (%)

RQD - Rock Qualtiy Designation (%)



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NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 32 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
3 days	4.0	-
12 days	7.5	-

CORE DRILLING RECORD

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

Log of Borehole BH 5

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Figure No. 7

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic 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
			38.66	14 15 16				Run 9
			36.5					
		Borehole Terminated at 16.2 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 32 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
3 days	4.0	-
12 days	7.5	-

CORE DRILLING RECORD

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

Log of Borehole BH 6

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Location: Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.

Figure No. 8

Page. 1 of 1

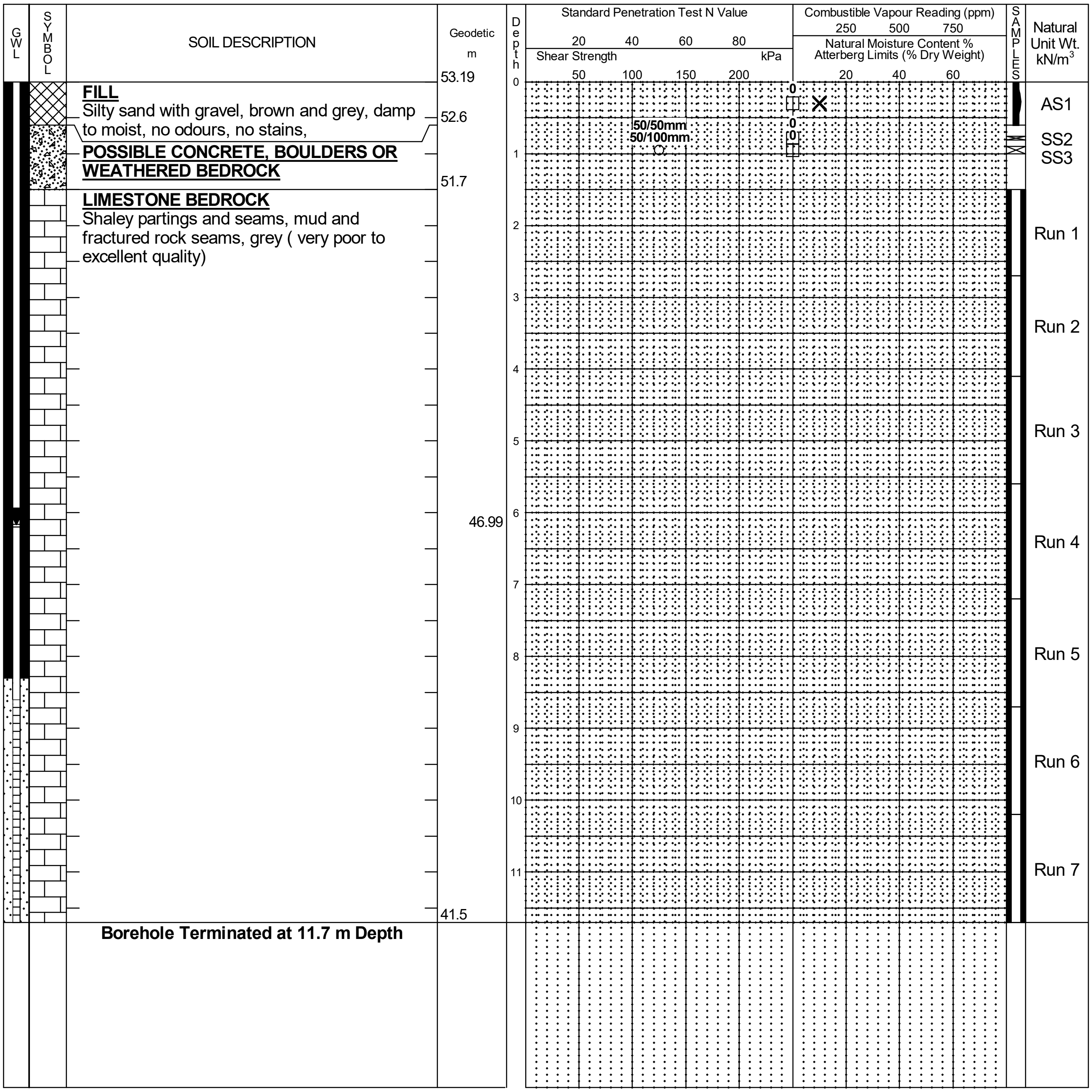
Date Drilled: 'February 25, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic

Logged by: M.D 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		



FILL
Silty sand with gravel, brown and grey, damp to moist, no odours, no stains,

POSSIBLE CONCRETE, BOULDERS OR WEATHERED BEDROCK

LIMESTONE BEDROCK
Shaley partings and seams, mud and fractured rock seams, grey (very poor to excellent quality)

Borehole Terminated at 11.7 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 32 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
8 days	7.6	-
77 days	6.3	-
86 days	6.2	
94 days	6.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	1.5 - 2.7	100	10
2	2.7 - 4.1	100	64
3	4.1 - 5.6	100	92
4	5.6 - 7.2	100	97
5	7.2 - 8.7	100	100
6	8.7 - 10.2	100	90
7	10.2 - 11.7	100	100

Log of Borehole BH 7

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Location: Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.

Date Drilled: 'February 20, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

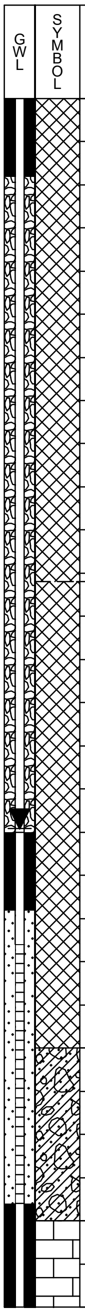
Datum: Geodetic

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

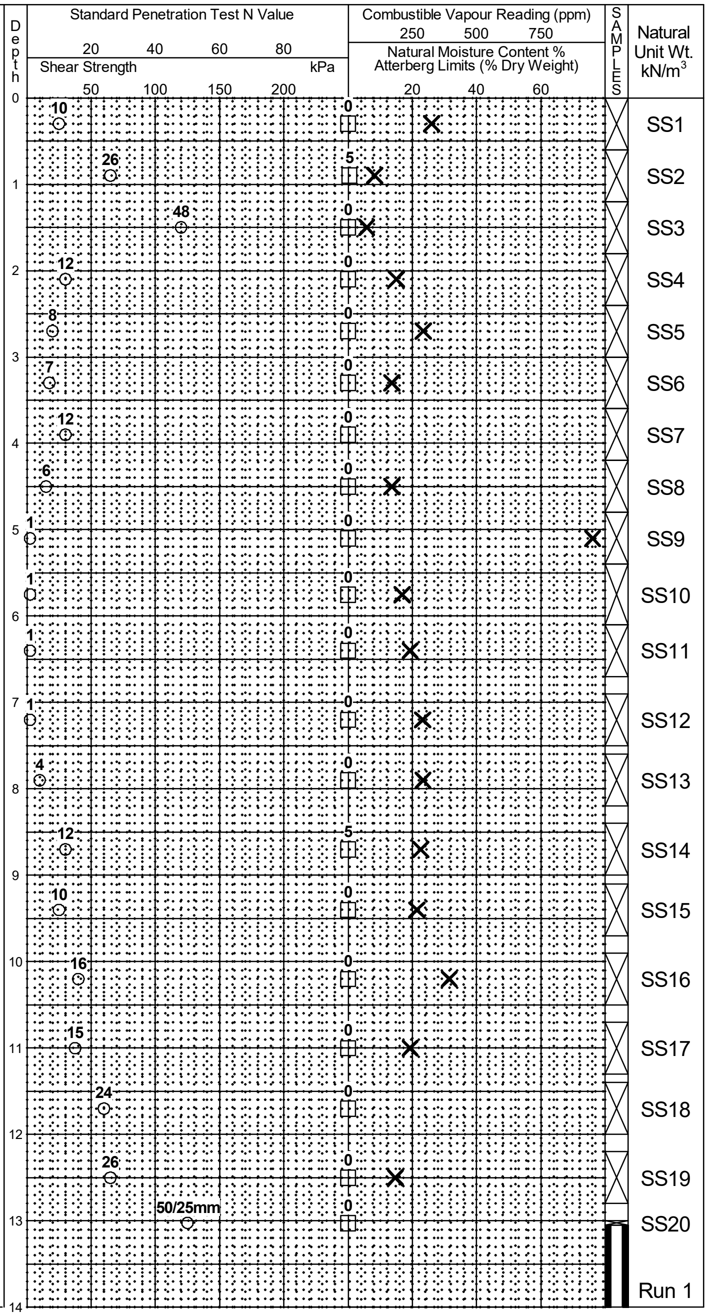
Figure No. 9

Page. 1 of 2

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



SOIL DESCRIPTION	Geodetic m
	53.69
FILL Silty sand with gravel, brown and grey, damp to moist, no odours, no stains (very loose to dense) reddish brown stains from 0.6 m to 1.2 m depths brick debris and black stains form 1.8 m to 3.6 m depths black stains from 3.6 m to 4.2 m depths wood debris from 4.2 m to 4.8 m depths 75 mm thick peat seam at 4.8 m depth	
	48.1
FILL Silty sand and gravel with silty clay, dark brown and dark grey, moist to wet, no odours, no stains, (very loose to compact) organics from 6.1 m to 10.5 m depths brick debris from 7.6 m to 8.2 m depths	
	45.19
	42.7
GLACIAL TILL Silty sand with gravel, cobbles, boulders, dark brown, wet, no odour, black organic-like stains, (compact)	
	40.7



Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 51 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
2 days	10.6	-
14 days	10.6	-
82 days	7.8	
91 days	8.5	
99 days	8.7	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	13 - 14.5	100	63
2	14.5 - 16.3	82	42
3	16.3 - 17.8	100	97

Log of Borehole BH 7

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Figure No. 9

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value 20 40 60 80 Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m^3
		LIMESTONE BEDROCK Some shaley partings and seams, mud and fractured rock seams, grey (poor to excellent quality) *(continued)*	39.69	14 15 16 17				Run 2 Run 3
			35.9					
		Borehole Terminated at 17.8 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 51 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
2 days	10.6	-
14 days	10.6	-
82 days	7.8	
91 days	8.5	
99 days	8.7	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	13 - 14.5	100	63
2	14.5 - 16.3	82	42
3	16.3 - 17.8	100	97

Log of Borehole BH 8

Project No: OTT-00250193-C0

Project: Geotechnical Investigation and Phase Two Environmental Site Assessment

Location: Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.

Figure No. 10

Page. 1 of 1

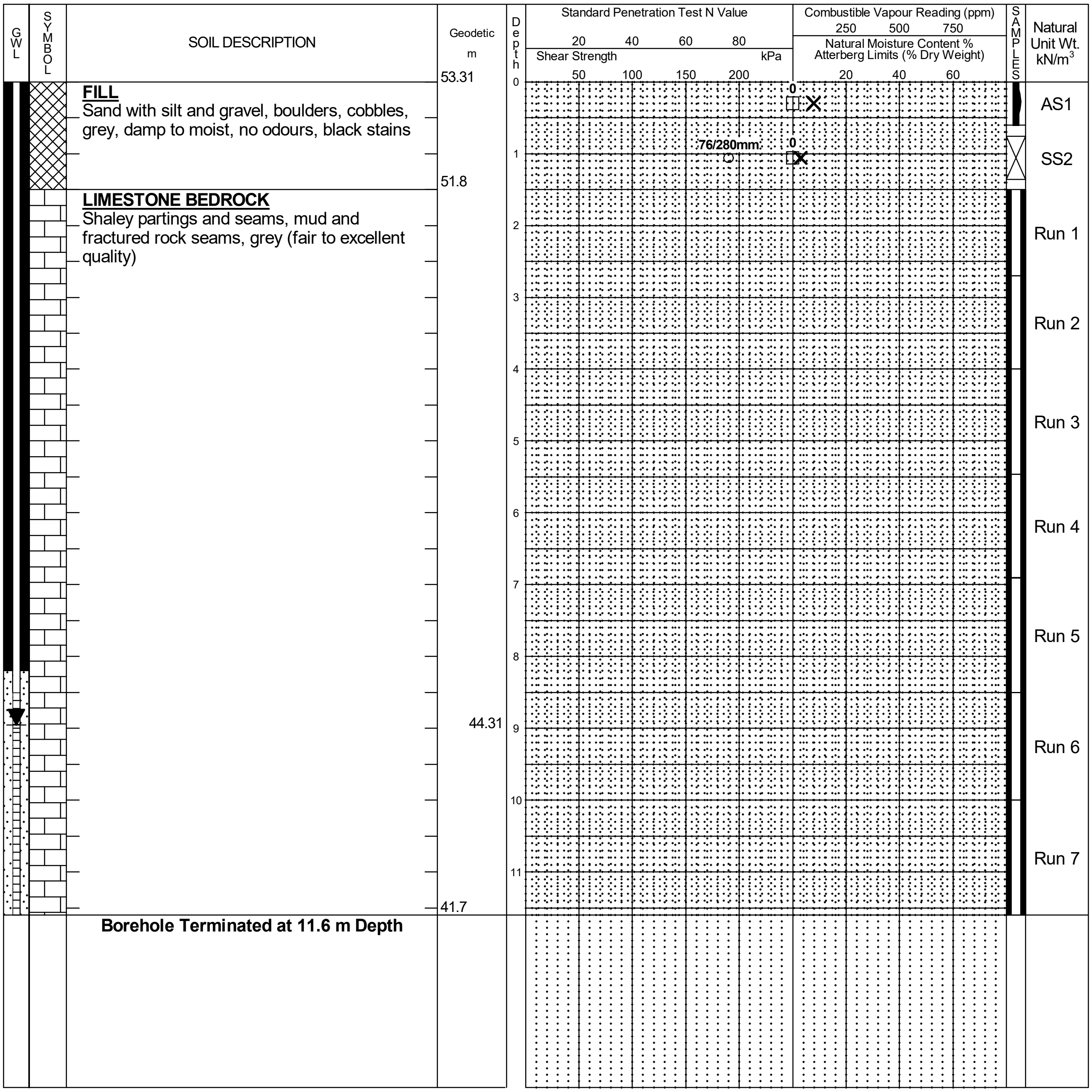
Date Drilled: 'February 25, 2019

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic

Logged by: M.D   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	▲



FILL
Sand with silt and gravel, boulders, cobbles, grey, damp to moist, no odours, black stains

LIMESTONE BEDROCK
Shaley partings and seams, mud and fractured rock seams, grey (fair to excellent quality)

Borehole Terminated at 11.6 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. 32 mm diameter monitoring well installed in borehole 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-00250193-C0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
9 days	4.5	-
77 days	9.3	-
86 days	9.2	
94 days	9.0	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	1.5 - 2.7	84	62
2	2.7 - 4	100	81
3	4 - 5.5	100	91
4	5.5 - 6.9	100	84
5	6.9 - 8.5	100	95
6	8.5 - 10	100	100
7	10 - 11.6	100	97

EXP Services Inc.
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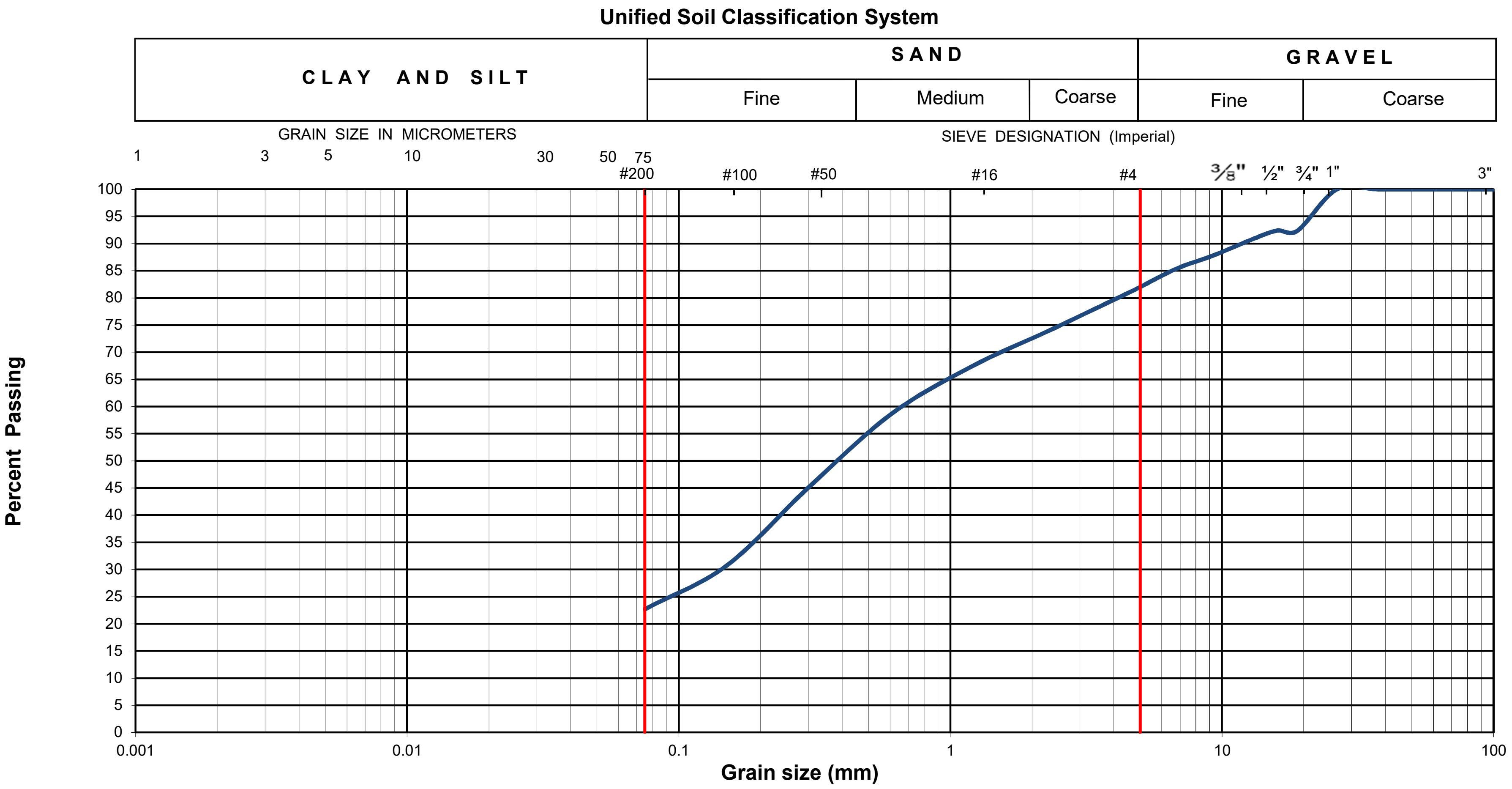
Ottawa, ON K2B 8H6

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



EXP Project No.:	**OTT-00250193-C0**	**Project Name :**	**Geotechnical Investigation and Phase Two Environmental Site Assessment**				
Client :	**Zibi Canada**	**Project Location :**	**Chaudière Island, Ottawa, Ontario**				
Date Sampled :	**February 21, 2019**	**Borehole No:**	**BH3**	**Sample:**	**SS3**	**Depth (m) :**	**1.5-2.1**
Sample Composition :		**Gravel (%)** 18	**Sand (%)** 59	**Silt & Clay (%)**	**23**	**Figure :**	**11**
Sample Description :	**Silty Sand with Gravel (SM)**						

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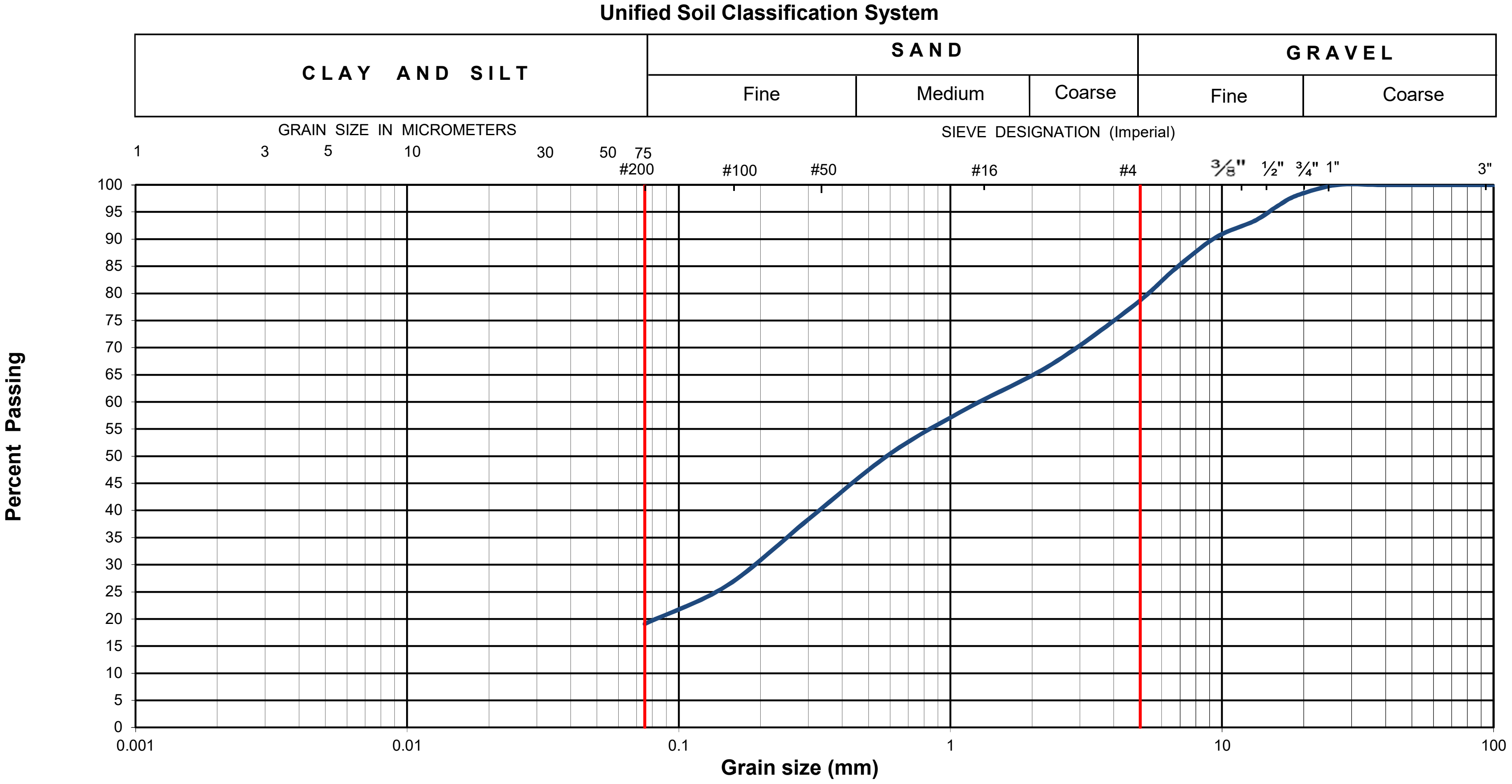
Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136



EXP Project No.:	**OTT-00250193-C0**	**Project Name :**	**Geotechnical Investigation and Phase Two Environmental Site Assessment**				
Client :	**Zibi Canada**	**Project Location :**	**Chaudière Island, Ottawa, Ontario**				
Date Sampled :	**February 21, 2019**	**Borehole No:**	**BH5**	**Sample:**	**AS1**	**Depth (m) :**	**0-0.6**
Sample Composition :		**Gravel (%)**	**22**	**Sand (%)**	**59**	**Silt & Clay (%)**	**19**
Sample Description :	**Silty Sand with Gravel (SM)**					**Figure :**	**12**

EXP Services Inc.
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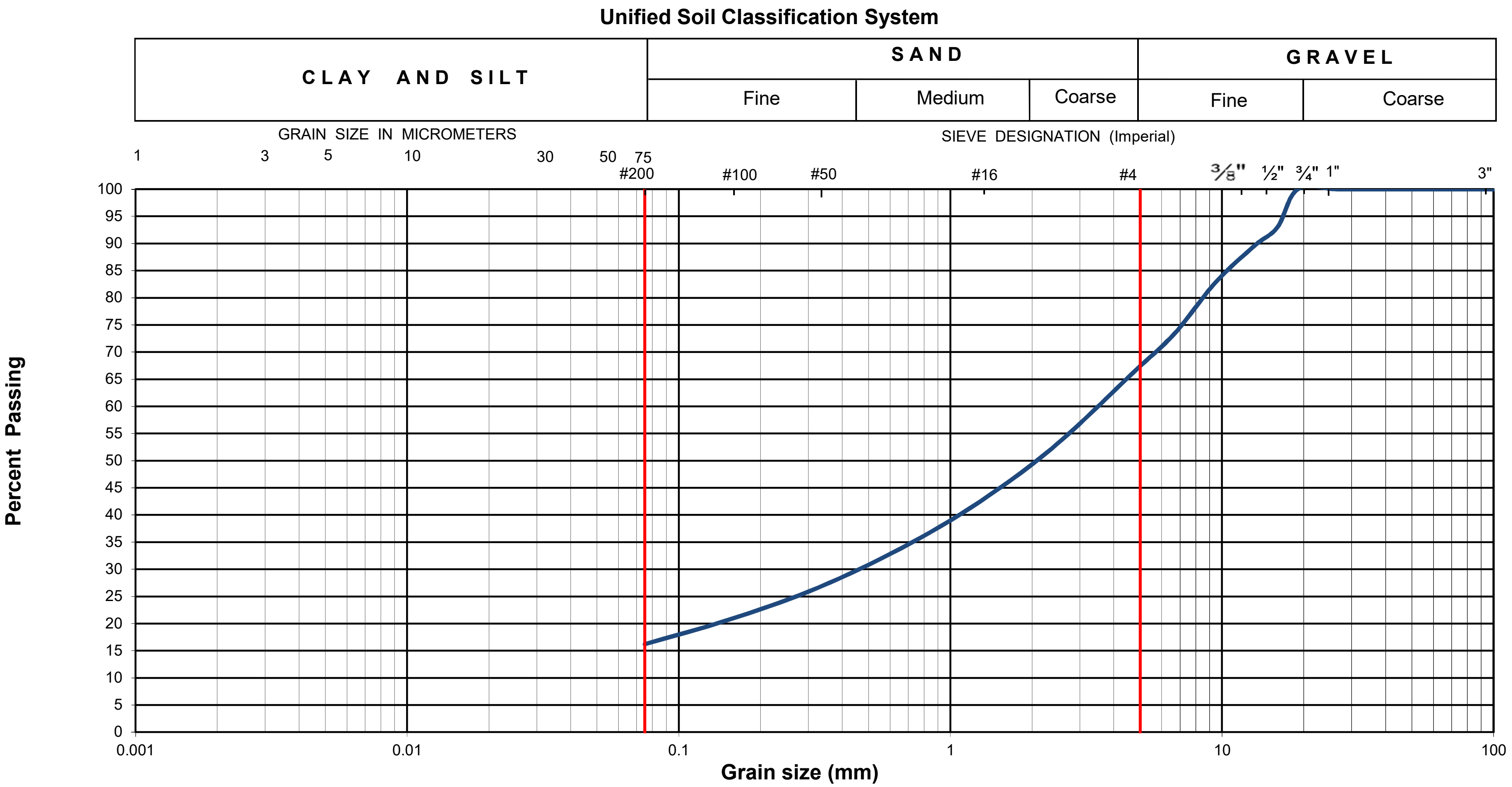
Ottawa, ON K2B 8H6

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



EXP Project No.:	OTT-00250193-C0	**Project Name :**	Geotechnical Investigation and Phase Two Environmental Site Assessment					
Client :	Zibi Canada	**Project Location :**	Chaudière Island, Ottawa, Ontario					
Date Sampled :	February 21, 2019	**Borehole No:**	BH7	**Sample:**	SS3		**Depth (m) :**	1.2-1.8
Sample Composition :		**Gravel (%)**	34	**Sand (%)**	50	**Silt & Clay (%)** 16	**Figure :**	13
Sample Description :	Silty Sand with Gravel (SM)							

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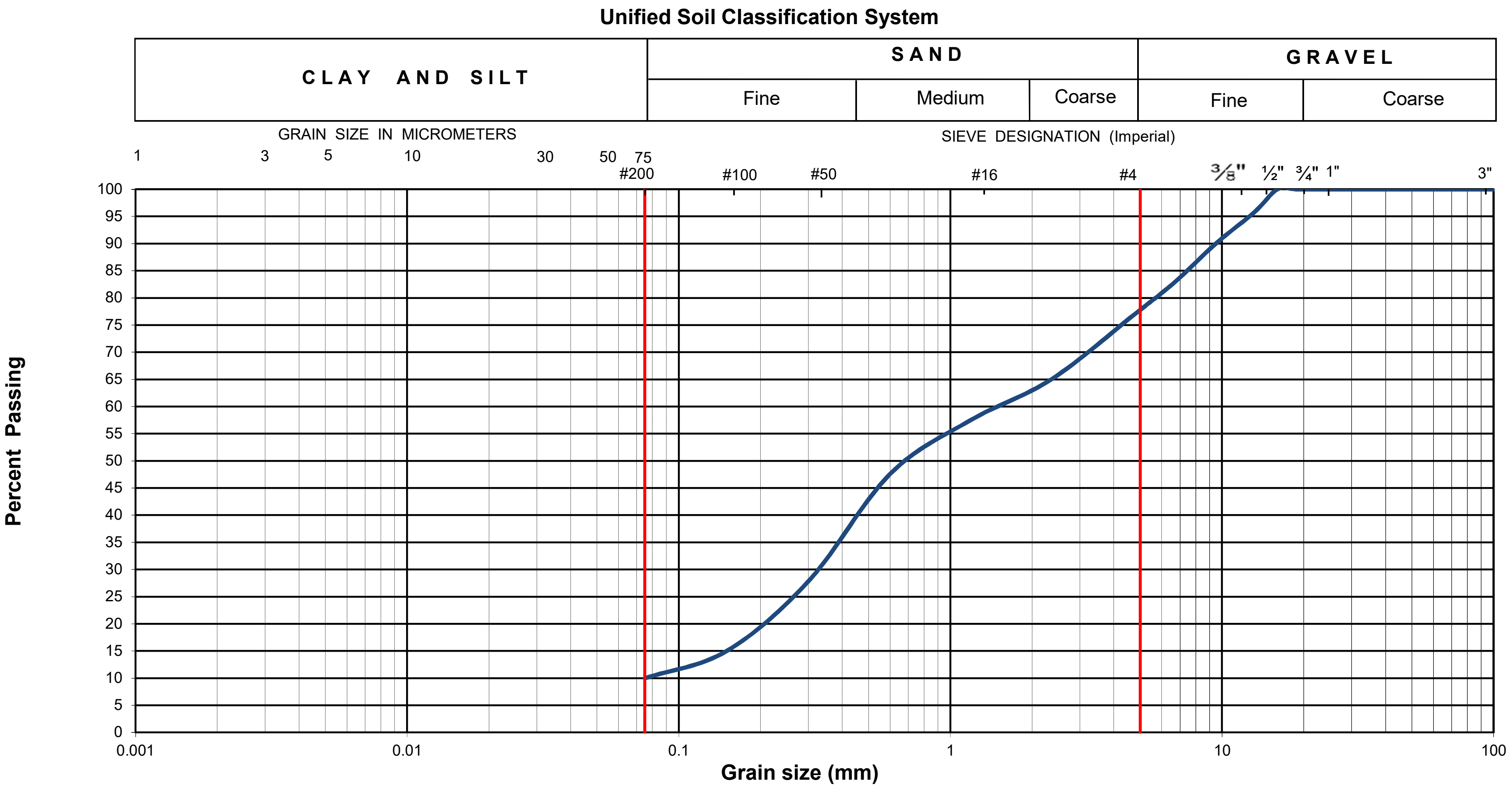
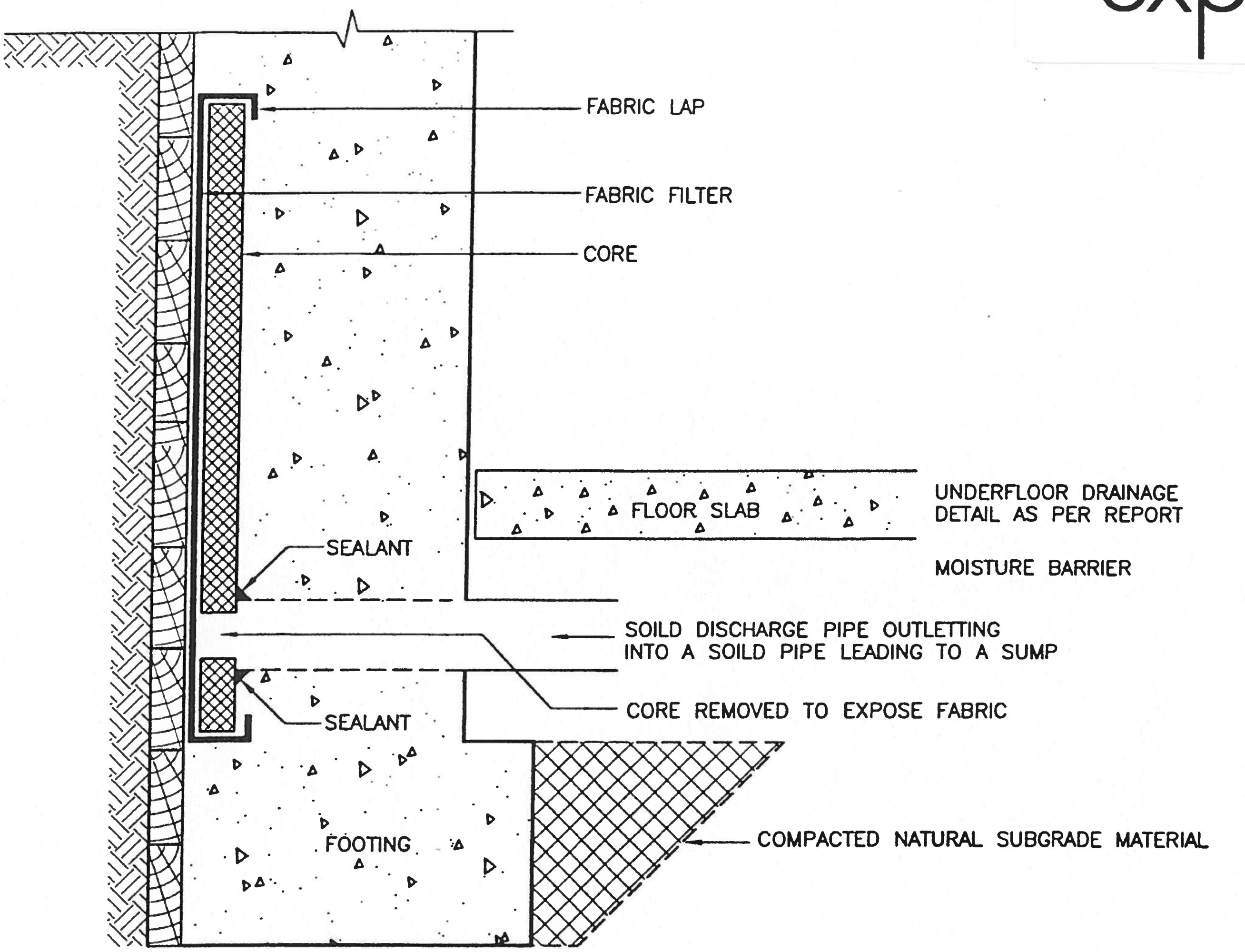
Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136



EXP Project No.:	**OTT-00250193-C0**	**Project Name :**	**Geotechnical Investigation and Phase Two Environmental Site Assessment**						
Client :	**Zibi Canada**	**Project Location :**	**Chaudière Island, Ottawa, Ontario**						
Date Sampled :	**February 21, 2019**	**Borehole No:**	**BH8**	**Sample:**	**AS1**			**Depth (m) :**	**0-0.6**
Sample Composition :		**Gravel (%)**	**23**	**Sand (%)**	**67**	**Silt & Clay (%)**	**10**	**Figure :**	**14**
Sample Description :	**Poorly Graded Sand with Silt and Gravel (SP-SM)**								

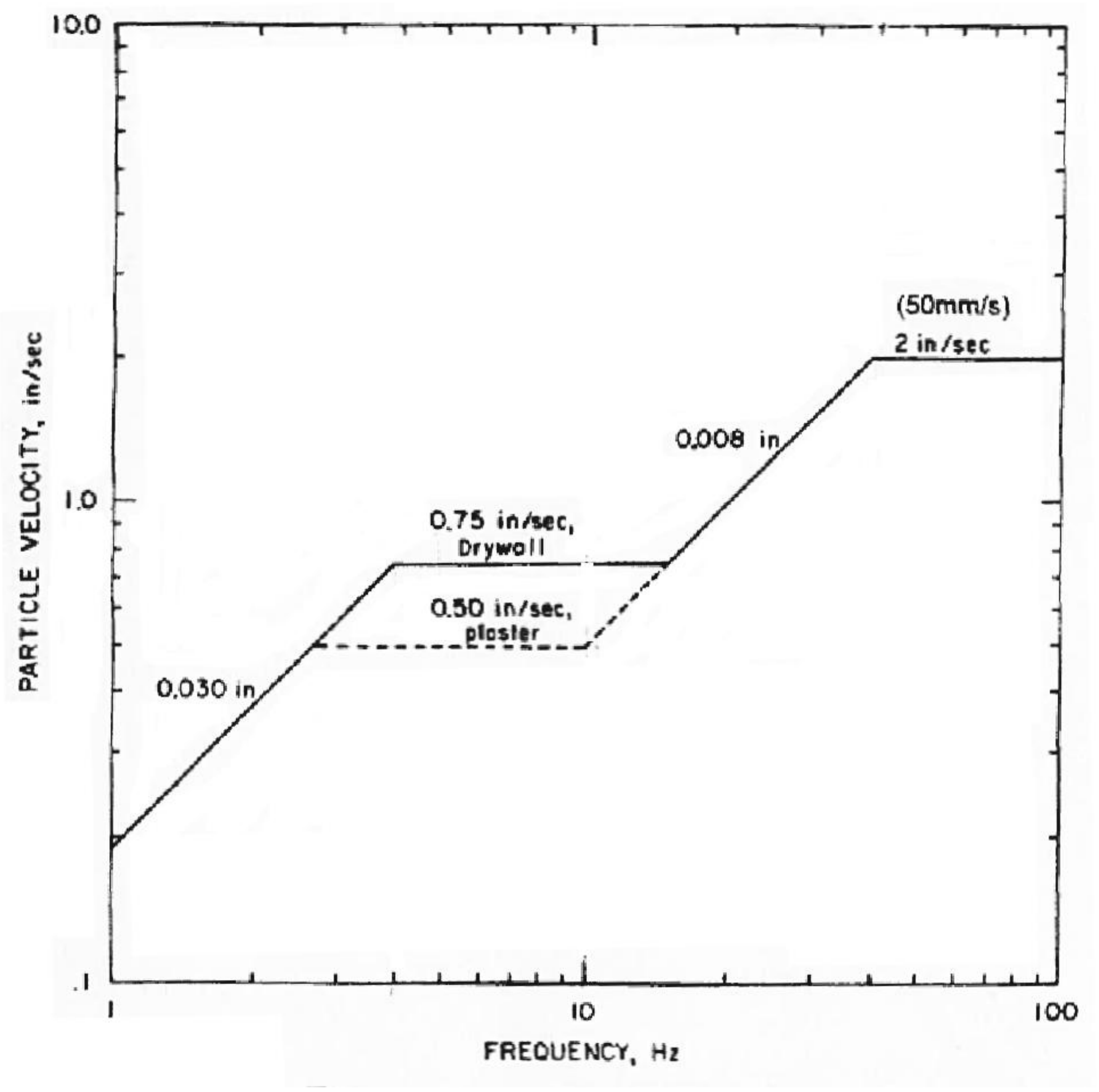


SECTION AT DISCHARGE PIPE

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



UNITED STATES BUREAU OF MINES VIBRATION CRITERIA

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scale NTS

date April 2019

drawn by M.N.

checked by S.K.A.

CLIENT: ZIBI Canada

TITLE: UNITED STATES BUREAU OF MINES VIBRATION CRITERIA
Block 211, Chaudière Island, Ottawa, Ontario

project no. OTT-00250193-C0

FIG 16

Filename: r:\210000\212000\212365-a0 pw\working drawings\2014-08 final report sketches 2.2 gdm\212365-a0 fig 23.dwg
Last Saved: 8/8/2014 2:11:30 PM
Last Plotted: 8/8/2014 2:11:50 PM Plotted by: nugentm Pen Table:: trow standard, july 01, 2004.ctb

Appendix A: 2006 Borehole Logs – DST Consulting Engineers Ltd. – 2014 and 2015 Paterson Group Inc. Report

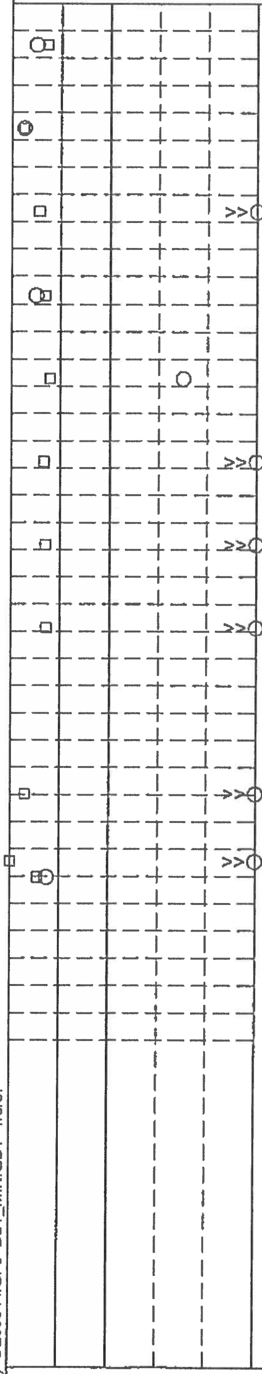
LOG OF BOREHOLE / MONITORING WELL BHMW16

DST REF. No.: **OE06544**
CLIENT: **National Capital Commission**
PROJECT: **Phase I & II Environmental Site Assessment**
LOCATION: **Chaudiere and Albert Islands, Ottawa, Ontario**
SURFACE ELEV.: **51.81 m (Geodetic)**

Drilling Data
METHOD: **CME 55 Drill Rig**
DIAMETER: **200 mm**

DATE: **July 26 2006**

CCGD/PID *
○ RKI EAGLE (PPM) 20 40 60 80
□ *MINIRAE (PPM)* 5 10 15 20



PPM	No.	Type	SPT N-Value
10 *3.6*	SS1	▨	23
5 *1.2*	SS2	▨	11
160 *2.8*	SS3	▨	8
10 *3.4*	SS4	▨	31
70 *3.9*	SS5	▨	20
580 *3.3*	SS6	▨	12
430 *3.5*	SS7	▨	11
110 *3.6*	SS8	▨	15
	SS9	▨	19
450 *1.5*	SS10	▨	22
300 *0*	SS11	▨	100
15 *2.8*	SGS12		

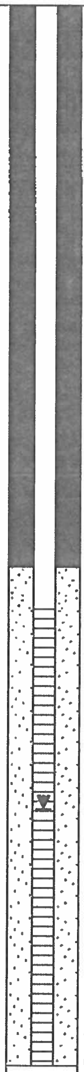
SUBSURFACE PROFILE

SURFACE

MATERIAL DESCRIPTION
SAND AND GRAVEL - silty, some brick, possible boulders, dark brown (fill)
CLAY - silty, trace coal, dark grey (fill)
SAND - silty, some coal and brick, trace gravel, dark brown (fill)
SAND AND GRAVEL - trace clay, possible boulders, grey (fill)
GRAVEL - some sand, trace silt, grey (fill)
SAND AND GRAVEL - trace silt (fill)
BEDROCK - limestone
End of borehole at 7.8 m depth.

DPTH m	ELEV m
0.5	51
1.0	
1.5	50
2.0	
2.5	49
3.0	
3.5	48
4.0	
4.5	47
5.0	
5.5	46
6.0	
6.5	45
7.0	
7.5	

WATER DATA



REMARKS

Flushmount installed.

Groundwater recorded at 5.9 m depth on August 1, 2006.

Spoon refusal at 6.5 m depth.

Black soil with stong odour noted from auger cuttings at 6.7 m depth.

Down hole hammer from 7.0 m to 7.8 m depth.

DST CONSULTING ENGINEERS

DST Consulting Engineers Inc.
203 - 2150 THURSTON DRIVE
OTTAWA, ONTARIO, K1G 5T9
PH: (613)748-1415
FX: (613)748-1356
Email: ottawa@dstgroup.com
Web: www.dstgroup.com

* - Catalytic Combustible Gas Detector

SAMPLE TYPE LEGEND

☐ Auger Sample	▩ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
▯ Thin Wall Tube	⊽ Grab Sample	

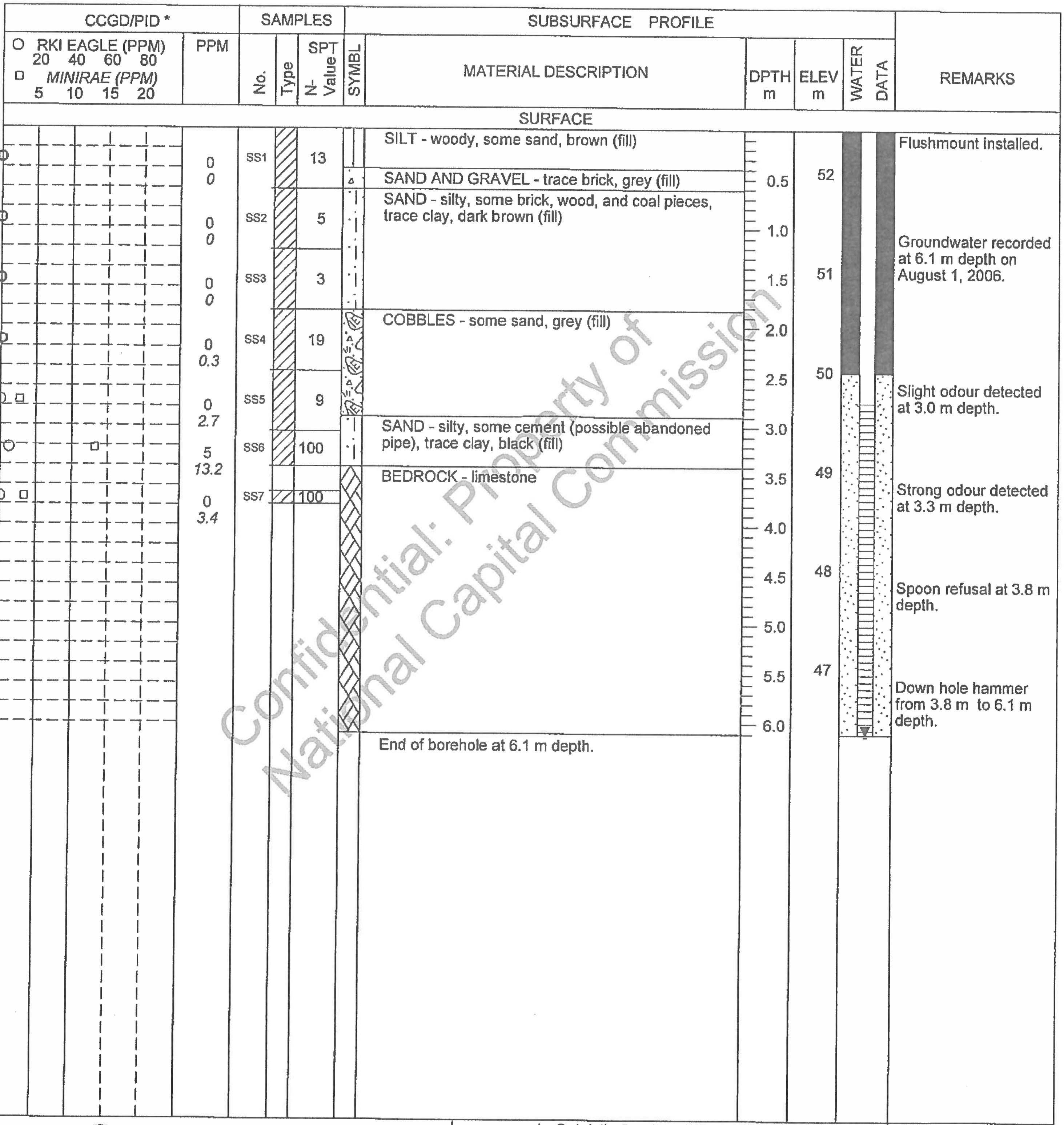
APPENDIX C

LOG OF BOREHOLE / MONITORING WELL BHMW17

DST REF. No.: **OE06544**
CLIENT: **National Capital Commission**
PROJECT: **Phase I & II Environmental Site Assessment**
LOCATION: **Chaudiere and Albert Islands, Ottawa, Ontario**
SURFACE ELEV.: **52.43 m (Geodetic)**

Drilling Data
METHOD: **CME 55 Drill Rig**
DIAMETER: **200 mm**

DATE: **July 27 2006**



CCGD/PID * PPM (RKI EAGLE MINIRAE)	SAMPLES No.	Type	SPT N-Value
0 0	SS1	Split Spoon Sample	13
0 0	SS2	Split Spoon Sample	5
0 0	SS3	Split Spoon Sample	3
0 0.3	SS4	Split Spoon Sample	19
0 2.7	SS5	Split Spoon Sample	9
5 13.2	SS6	Split Spoon Sample	100
0 3.4	SS7	Split Spoon Sample	100

MATERIAL DESCRIPTION	DPTH m	ELEV m
SURFACE		
SILT - woody, some sand, brown (fill)	0.5	52
SAND AND GRAVEL - trace brick, grey (fill)	1.0	
SAND - silty, some brick, wood, and coal pieces, trace clay, dark brown (fill)	1.5	51
COBBLES - some sand, grey (fill)	2.0	
	2.5	50
SAND - silty, some cement (possible abandoned pipe), trace clay, black (fill)	3.0	
BEDROCK - limestone	3.5	49
	4.0	
	4.5	48
	5.0	
	5.5	47
	6.0	
End of borehole at 6.1 m depth.		

REMARKS

Flushmount installed.

Groundwater recorded at 6.1 m depth on August 1, 2006.

Slight odour detected at 3.0 m depth.

Strong odour detected at 3.3 m depth.

Spoon refusal at 3.8 m depth.

Down hole hammer from 3.8 m to 6.1 m depth.

GASTECBH (OTTAWA) OE06544.GPJ DST_MIN.GDT 1/6/07

DST CONSULTING ENGINEERS

DST Consulting Engineers Inc.
203 - 2150 THURSTON DRIVE
OTTAWA, ONTARIO, K1G 5T9
PH: (613)748-1415
FX: (613)748-1356
Email: ottawa@dstgroup.com
Web: www.dstgroup.com

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SAMPLE TYPE LEGEND

▤ Auger Sample	▩ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
☐ Thin Wall Tube	▽ Grab Sample	

APPENDIX C

LOG OF BOREHOLE / MONITORING WELL BHMW18

DST REF. No.: **OE06544**
CLIENT: **National Capital Commission**
PROJECT: **Phase I & II Environmental Site Assessment**
LOCATION: **Chaudiere and Albert Islands, Ottawa, Ontario**
SURFACE ELEV.: **53.26 m (Geodetic)**

Drilling Data
METHOD: **CME 55 Drill Rig**
DIAMETER: **200 mm**

DATE: **July 26 2006**

CCGD/PID *: ○ RKI EAGLE (PPM) 20 40 60 80; □ MINIRAE (PPM) 5 10 15 20

SAMPLES

PPM	No.	Type	SPT N-Value
110 0.4	SS1	▨	10
0 0	SS2	▨	18
0 1.8	SS3	▨	6
0 1.0	SS4	▨	38
0 2.7	SS5	▨	1
0 0	SS6	▨	17
0 0	SS7	▨	23
5 3.1	SS8	▨	12
0 0	SS9	▨	5
0 3.8	SS10	▨	7
5 3.8	SS11		100

SUBSURFACE PROFILE

SURFACE

MATERIAL DESCRIPTION
GRASS COVER - silty, sand and organics (topsoil)
SAND AND GRAVEL - boulders, grey (fill) - brick
SAND - some silt, brown (fill)
SAND - silty, organics, black (fill) - cement
SAND - some silt and gravel, brown to light brown (fill) - black layer and rust
SAND AND GRAVEL - some silt, grey (fill)
SAND AND GRAVEL - possible boulders, grey (fill)
BEDROCK - limestone

DPTH m	ELEV m
	53
0.5	
1.0	
	52
1.5	
2.0	
	51
2.5	
3.0	
	50
3.5	
4.0	
	49
4.5	
5.0	
	48
5.5	
6.0	
	47
6.5	
7.0	
	46
7.5	
8.0	
	45
8.5	
9.0	
	44
9.5	

WATER DATA

REMARKS

Flushmount installed.

Drop in auger from 2.4 m to 3.1 m depth.

Down hole hammer from 6.7 m to 18.9 m depth.

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SAMPLE TYPE LEGEND

☐ Auger Sample	▩ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
▯ Thin Wall Tube	☐ Grab Sample	

APPENDIX C

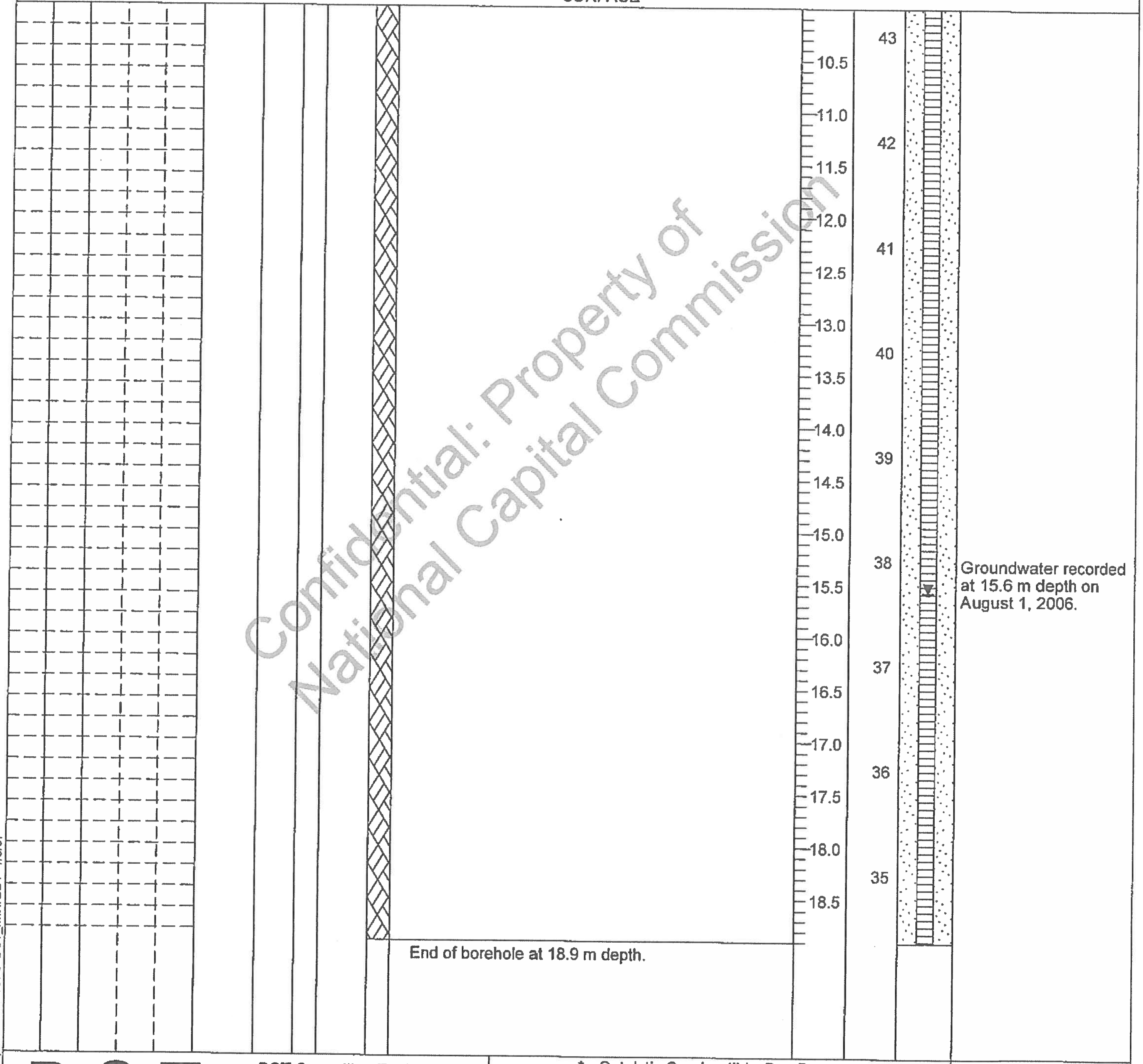
LOG OF BOREHOLE / MONITORING WELL BHMW18

DST REF. No.: **OE06544**
CLIENT: **National Capital Commission**
PROJECT: **Phase I & II Environmental Site Assessment**
LOCATION: **Chaudiere and Albert Islands, Ottawa, Ontario**
SURFACE ELEV.:**53.26 m (Geodetic)**

Drilling Data
METHOD: **CME 55 Drill Rig**
DIAMETER: **200 mm**

DATE: **July 26 2006**

CCGD/PID *		SAMPLES			SUBSURFACE PROFILE					REMARKS
○ RKI EAGLE (PPM) 20 40 60 80 □ MINIRAE (PPM) 5 10 15 20	PPM	No.	Type	SPT N-Value	SYMBL	MATERIAL DESCRIPTION	DPTH m	ELEV m	WATER DATA	
						SURFACE				
						End of borehole at 18.9 m depth.				Groundwater recorded at 15.6 m depth on August 1, 2006.



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Email: ottawa@dstgroup.com
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SAMPLE TYPE LEGEND

☰ Auger Sample	▩ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
☐ Thin Wall Tube	☰ Grab Sample	

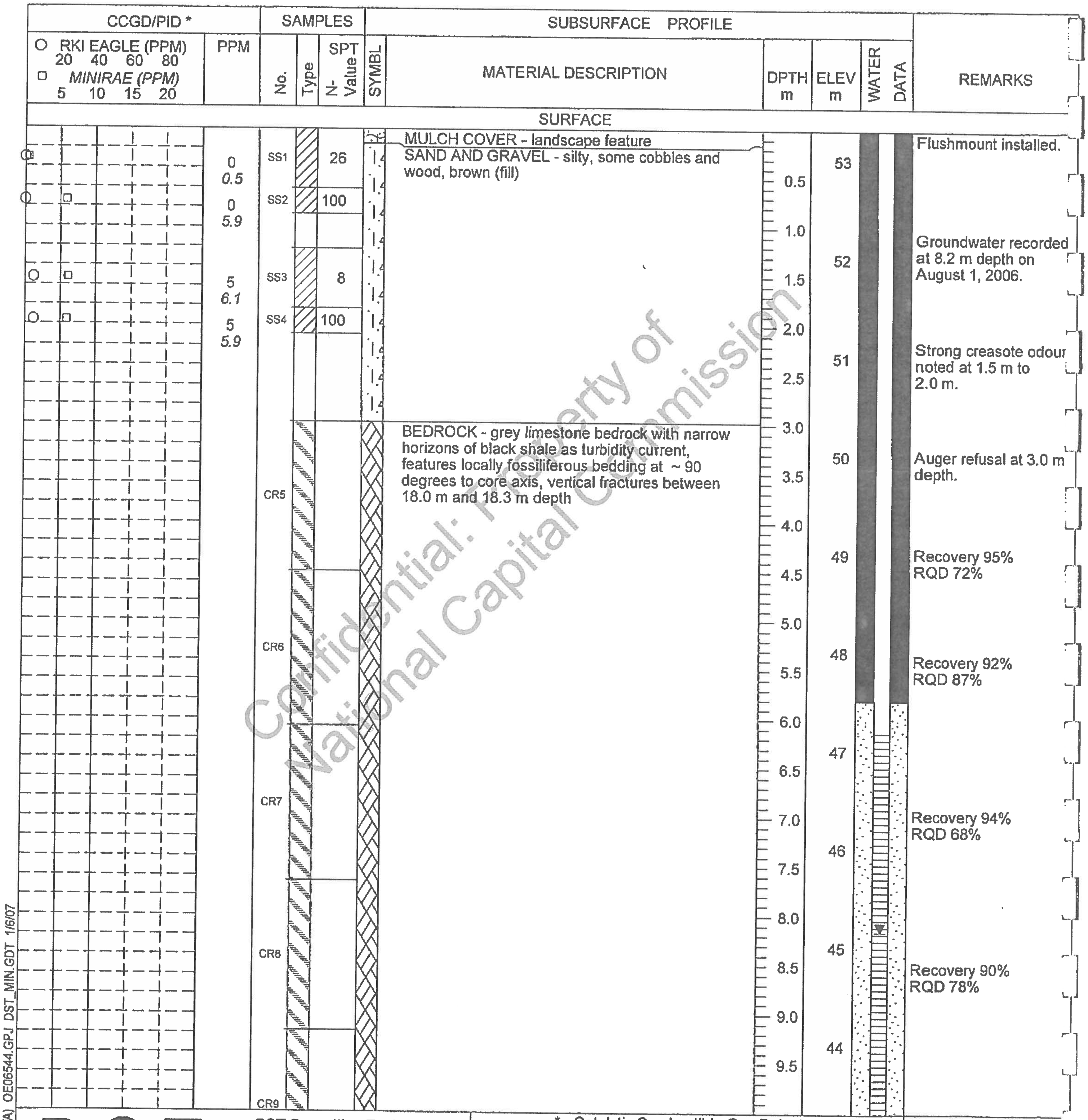
APPENDIX C

LOG OF BOREHOLE / MONITORING WELL BHMW19

DST REF. No.: **OE06544**
CLIENT: **National Capital Commission**
PROJECT: **Phase I & II Environmental Site Assessment**
LOCATION: **Chaudiere and Albert Islands, Ottawa, Ontario**
SURFACE ELEV.: **53.31 m (Geodetic)**

Drilling Data
METHOD: **CME 75 Drill Rig**
DIAMETER: **200 mm**

DATE: **July 26 2006**



PPM	No.	Type	SPT N-Value
0 0.5	SS1	▨	26
0 5.9	SS2	▨	100
5 6.1	SS3	▨	8
5 5.9	SS4	▨	100
	CR5	▧	
	CR6	▧	
	CR7	▧	
	CR8	▧	
	CR9	▧	

MATERIAL DESCRIPTION

MULCH COVER - landscape feature

SAND AND GRAVEL - silty, some cobbles and wood, brown (fill)

BEDROCK - grey limestone bedrock with narrow horizons of black shale as turbidity current, features locally fossiliferous bedding at ~ 90 degrees to core axis, vertical fractures between 18.0 m and 18.3 m depth

REMARKS

Flushmount installed.

Groundwater recorded at 8.2 m depth on August 1, 2006.

Strong creasote odour noted at 1.5 m to 2.0 m.

Auger refusal at 3.0 m depth.

Recovery 95%
RQD 72%

Recovery 92%
RQD 87%

Recovery 94%
RQD 68%

Recovery 90%
RQD 78%

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SAMPLE TYPE LEGEND

▭ Auger Sample	▩ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
▯ Thin Wall Tube	▽ Grab Sample	

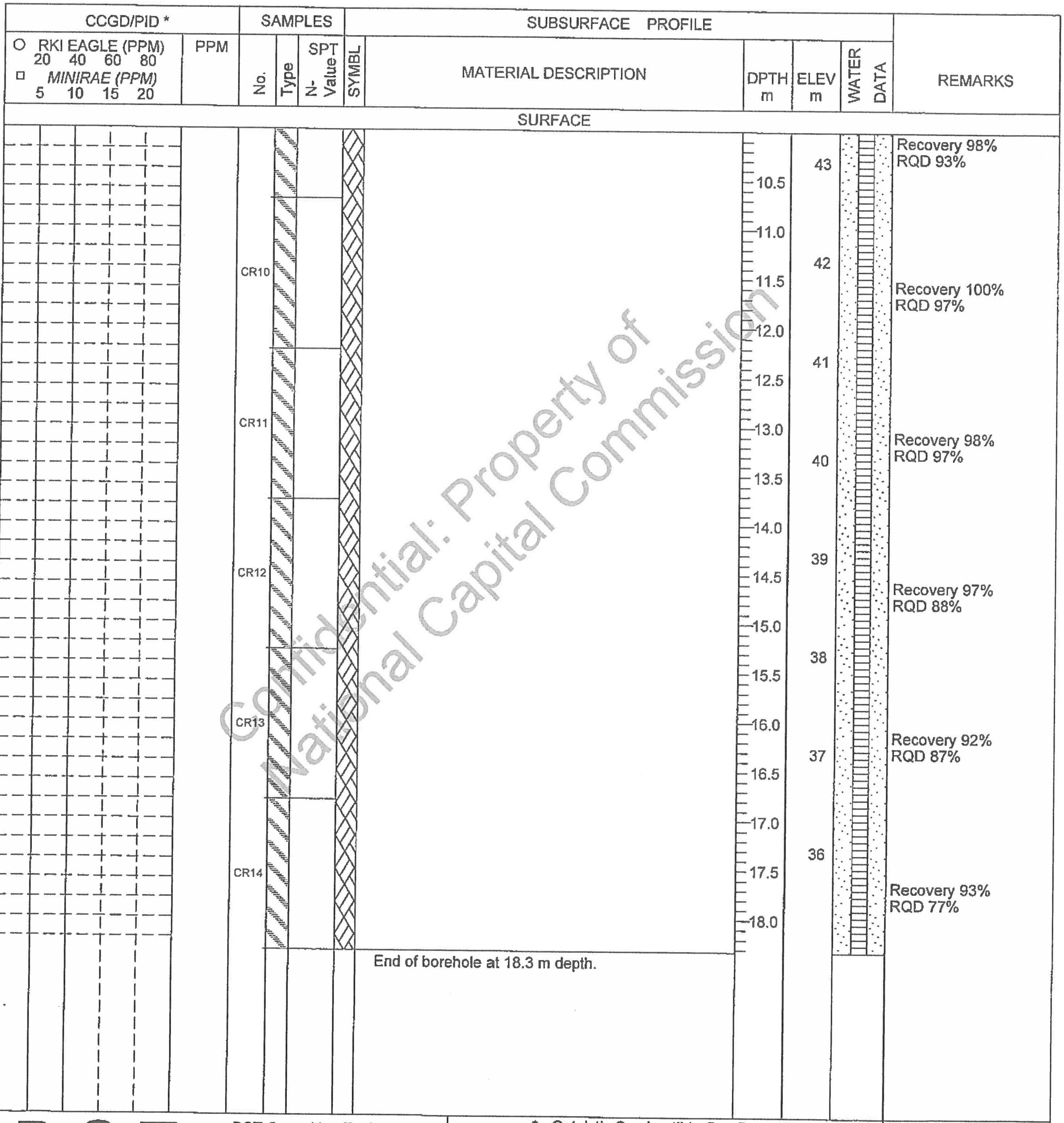
APPENDIX C

LOG OF BOREHOLE / MONITORING WELL BHMW19

DST REF. No.: **OE06544**
CLIENT: **National Capital Commission**
PROJECT: **Phase I & II Environmental Site Assessment**
LOCATION: **Chaudiere and Albert Islands, Ottawa, Ontario**
SURFACE ELEV.: **53.31 m (Geodetic)**

Drilling Data
METHOD: **CME 75 Drill Rig**
DIAMETER: **200 mm**

DATE: **July 26 2006**



No.	REMARKS
	Recovery 98% RQD 93%
CR10	Recovery 100% RQD 97%
CR11	Recovery 98% RQD 97%
CR12	Recovery 97% RQD 88%
CR13	Recovery 92% RQD 87%
CR14	Recovery 93% RQD 77%

End of borehole at 18.3 m depth.

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SAMPLE TYPE LEGEND

☰ Auger Sample	▩ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
☐ Thin Wall Tube	▽ Grab Sample	

APPENDIX C

LOG OF BOREHOLE / MONITORING WELL BHMW26

DST REF. No.: **OE06544**
CLIENT: **National Capital Commission**
PROJECT: **Phase I & II Environmental Site Assessment**
LOCATION: **Chaudiere and Albert Islands, Ottawa, Ontario**
SURFACE ELEV.: **52.94 m (Geodetic)**

Drilling Data
METHOD: **CME 55 Drill Rig**
DIAMETER: **200 mm**

DATE: **July 27 2006**

CCGD/PID *: ○ RKI EAGLE (PPM) 20 40 60 80; □ *MINIRAE (PPM)* 5 10 15 20

PPM	No.	Type	SPT N-Value	MATERIAL DESCRIPTION	REMARKS
				SURFACE	
				ASPHALT - 100 mm	Flushmount installed.
5 *1.0*	SS1	▨	7	SAND - silty, black (fill)	
5 *2.8*	SS2	▨	100	SAND AND GRAVEL - some silt, grey (fill)	
				BEDROCK - limestone	Spoon refusal at 0.7 m depth. Down hole hammer from 0.7 m to 10.7 m depth. Groundwater recorded at 9.3 m depth on August 1, 2006.

DPTH m	ELEV m
0.5	
1.0	52
1.5	
2.0	51
2.5	
3.0	50
3.5	
4.0	49
4.5	
5.0	48
5.5	
6.0	47
6.5	
7.0	46
7.5	
8.0	45
8.5	
9.0	44
9.5	
	43

WATER DATA

GASTECBH (OTTAWA) OE06544.GPJ DST_MIN.GDT 1/6/07

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SAMPLE TYPE LEGEND

☐ Auger Sample	▩ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
▯ Thin Wall Tube	⊟ Grab Sample	

APPENDIX C

LOG OF BOREHOLE / MONITORING WELL BHMW26

DST REF. No.: **OE06544**
CLIENT: **National Capital Commission**
PROJECT: **Phase I & II Environmental Site Assessment**
LOCATION: **Chaudiere and Albert Islands, Ottawa, Ontario**
SURFACE ELEV.:**52.94 m (Geodetic)**

Drilling Data
METHOD: **CME 55 Drill Rig**
DIAMETER: **200 mm**

DATE: **July 27 2006**

CCGD/PID * ○ RKI EAGLE (PPM) 20 40 60 80 □ MINIRAE (PPM) 5 10 15 20	PPM	SAMPLES No.	SAMPLES Type	SAMPLES SPT N-Value	SUBSURFACE PROFILE SYMBL	MATERIAL DESCRIPTION	DPTH m	ELEV m	WATER DATA	REMARKS
						SURFACE				
						End of borehole at 10.7 m depth.	10.5	42		

GASTECBH (OTTAWA) OE06544.GPJ DST_MIN.GDT 1/6/07

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SAMPLE TYPE LEGEND

☰ Auger Sample	▩ Rock Core	▥ Ponar Sample
▨ Split Spoon Sample	▧ Side Sampler	
☐ Thin Wall Tube	⊽ Grab Sample	

APPENDIX C

Appendix B: Bedrock Core Photographs

DRY BEDROCK CORES

WET BEDROCK CORES

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borehole no.	core runs	PROJECT	project no.
BH2	Run 1: 3.2m - 4.2m Run 2: 4.2m - 5.6m Run 3: 5.6m - 7.0m Run 4: 7.0m - 8.5m Run 5: 8.5m - 10.0m Run 6: 10.0m - 11.5m	Preliminary Geotechnical Investigation - Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.	OTT-00250193-C0
date cored Feb 21, 2019		ROCK CORE PHOTOGRAPHS	**Figure: B-1**

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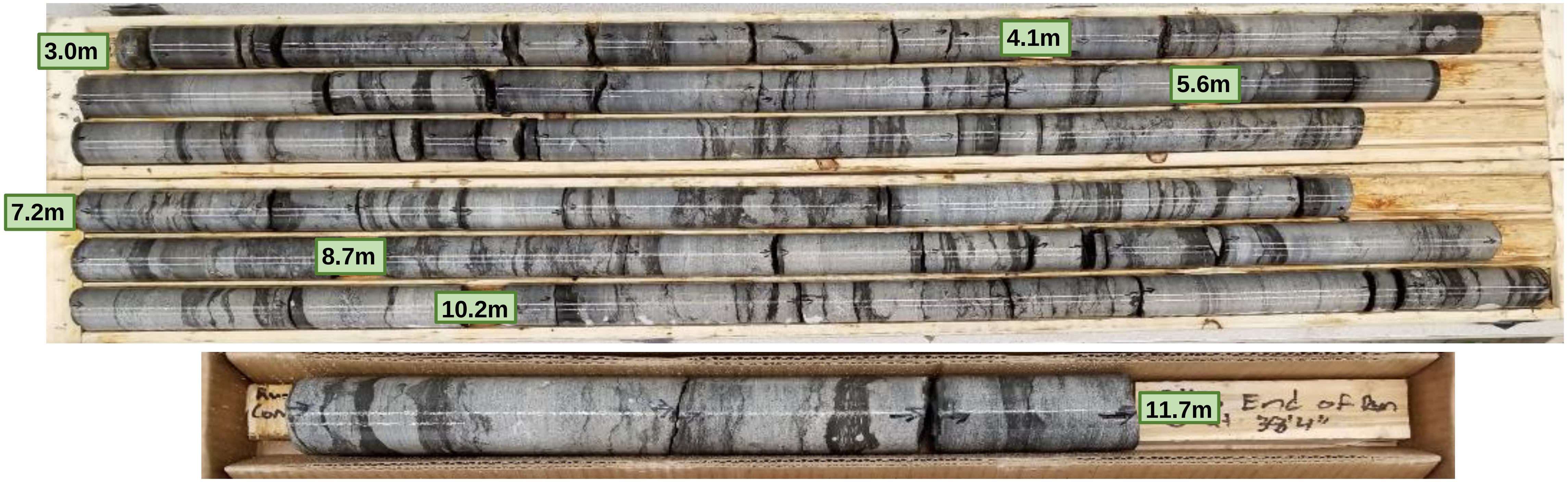

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borehole no. **BH3**	core runs Run 1: 3.0m - 4.1m Run 2: 4.1m - 5.6m Run 3: 5.6m - 7.2m Run 4: 7.2m - 8.7m Run 5: 8.7m - 10.2m Run 6: 10.2m - 11.7m	PROJECT Preliminary Geotechnical Investigation - Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.	project no. OTT-00250193-C0
date cored Feb 21, 2019		ROCK CORE PHOTOGRAPHS	**Figure: B-2**

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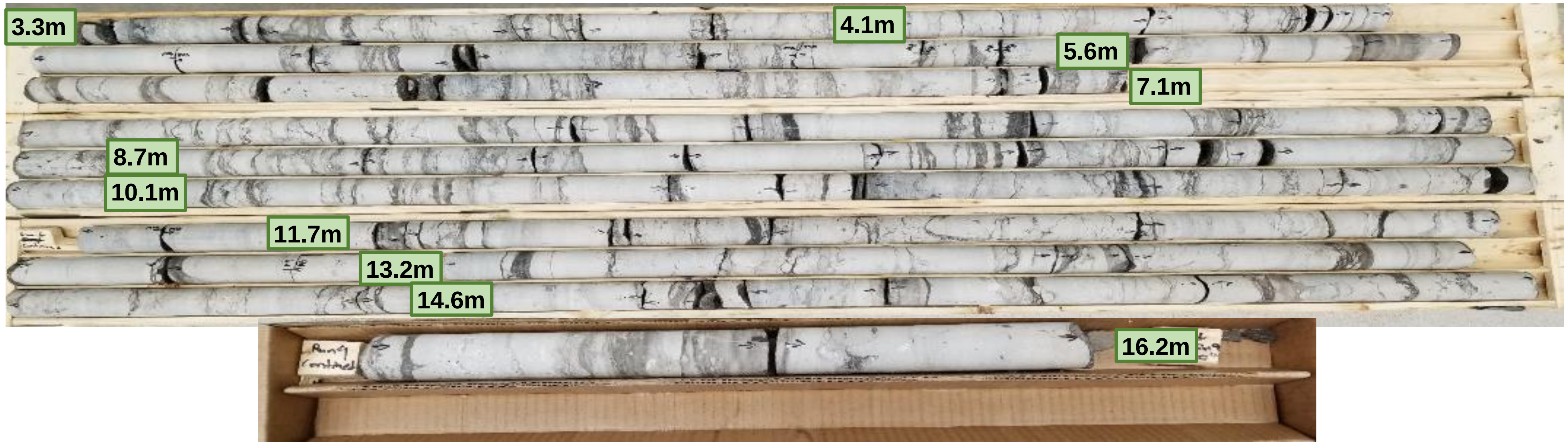
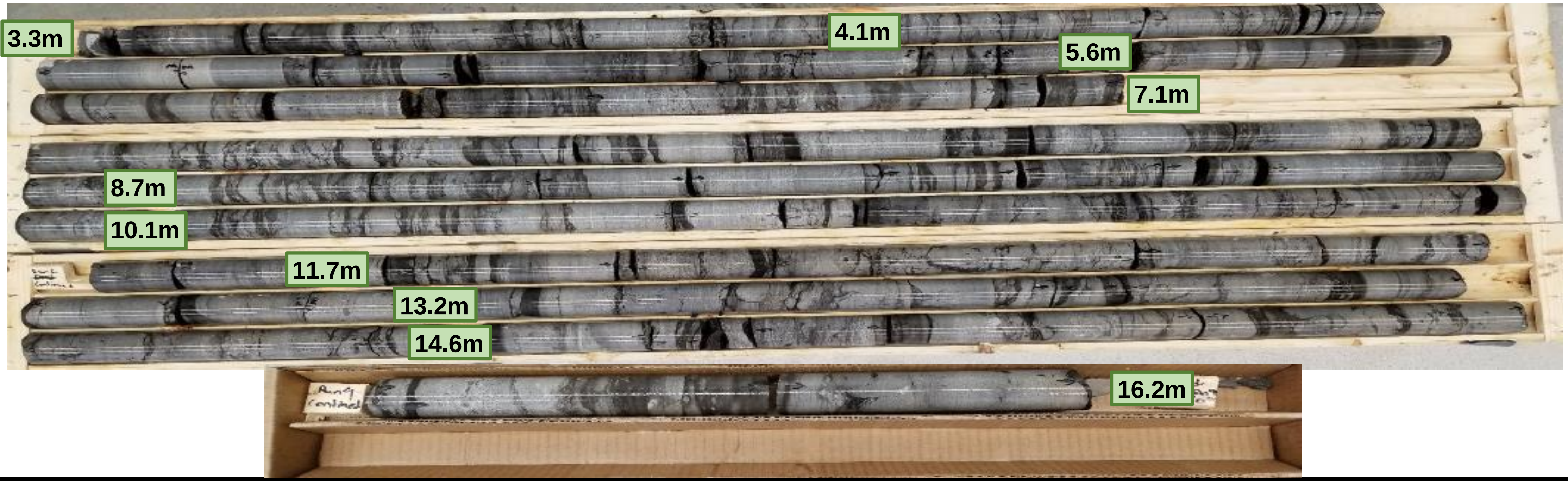
borehole no. **BH4**	core runs Run 1: 4.0m - 5.5m	PROJECT Preliminary Geotechnical Investigation -Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.	project no. OTT-00250193-C0
date cored Feb 21, 2019		ROCK CORE PHOTOGRAPHS	**Figure: B-3**

DRY BEDROCK CORES



WET BEDROCK CORES



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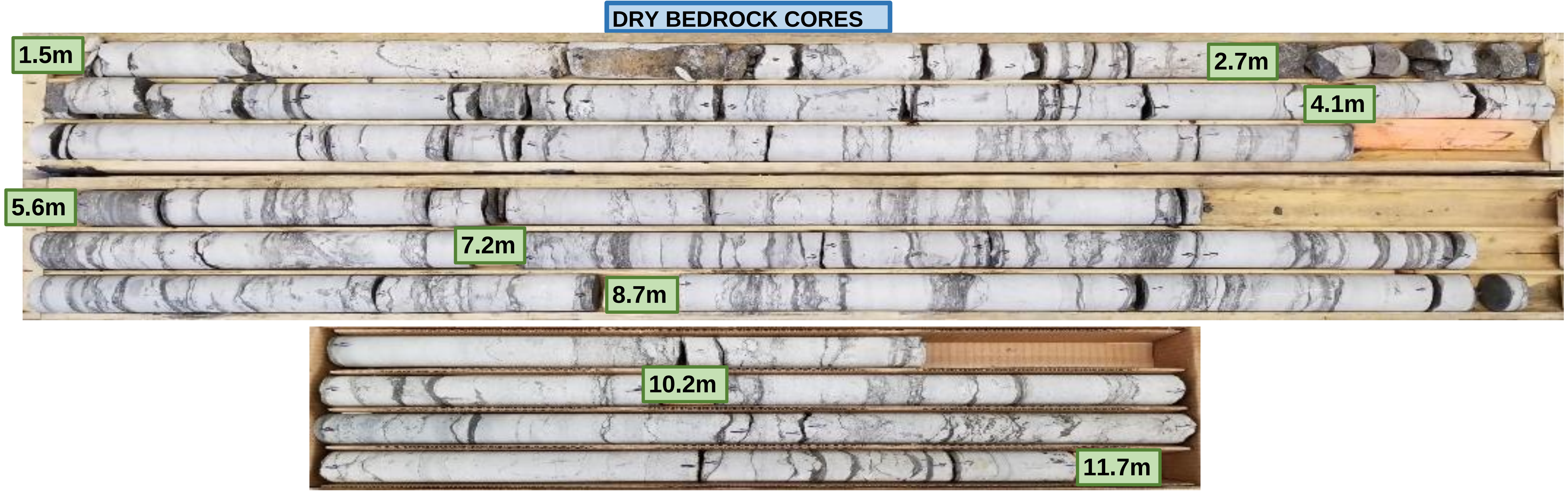
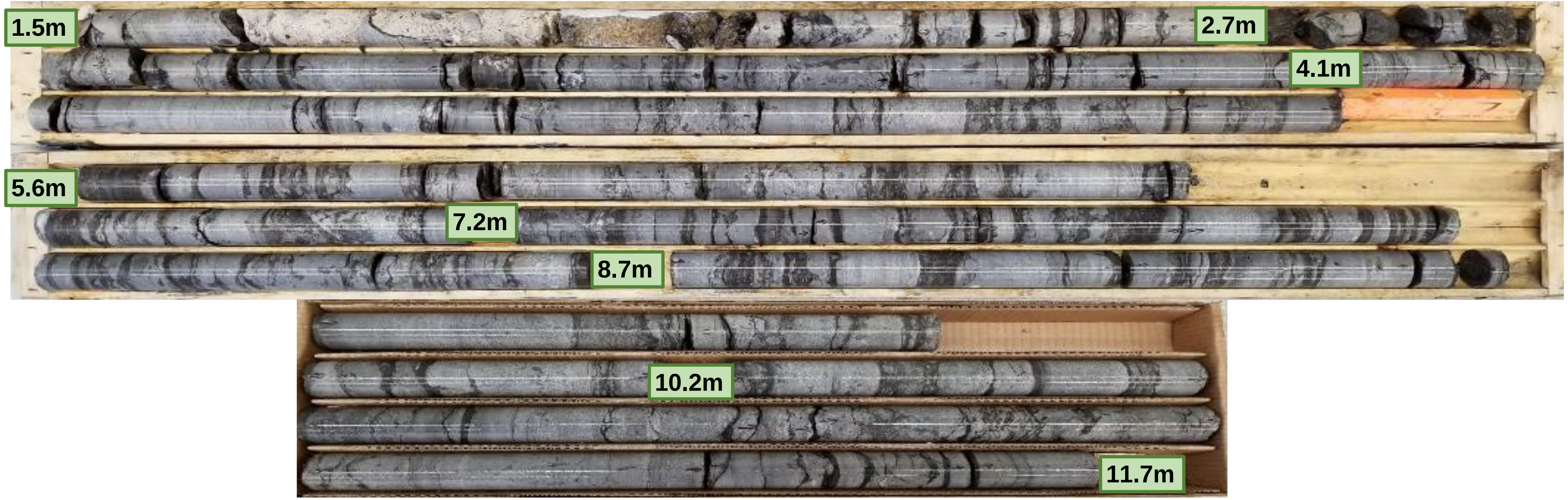
borehole no. **BH5**	core runs Run 1: 3.3m - 4.1m Run 2: 4.1m -5.6m Run 3: 5.6m - 7.1m Run 4: 7.1m - 8.7m Run 5: 8.7m - 10.1m Run 6: 10.1m - 11.7m Run 7: 11.7m - 13.2m Run 8: 13.2m - 14.6m Run 9: 14.6m - 16.2m	PROJECT Preliminary Geotechncial Investigation - Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.	project no. OTT-00250193-C0
date cored Feb 22, 2019		ROCK CORE PHOTOGRAPHS	**Figure: B-4**

DRY BEDROCK CORES



WET BEDROCK CORES



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borehole no.	core runs	PROJECT	project no.
BH6	Run 1: 1.5m - 2.7m Run 2: 2.7m - 4.1m Run 3: 4.1m - 5.6m Run 4: 5.6m - 7.2m Run 5: 7.2m - 8.7m Run 6: 8.7m - 10.2m Run 7: 10.2m - 111.7m	Preliminary Geotechnical Investigation - Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.	OTT-00250193-C0
date cored Feb 25, 2019		ROCK CORE PHOTOGRAPHS	**Figure: B-5**

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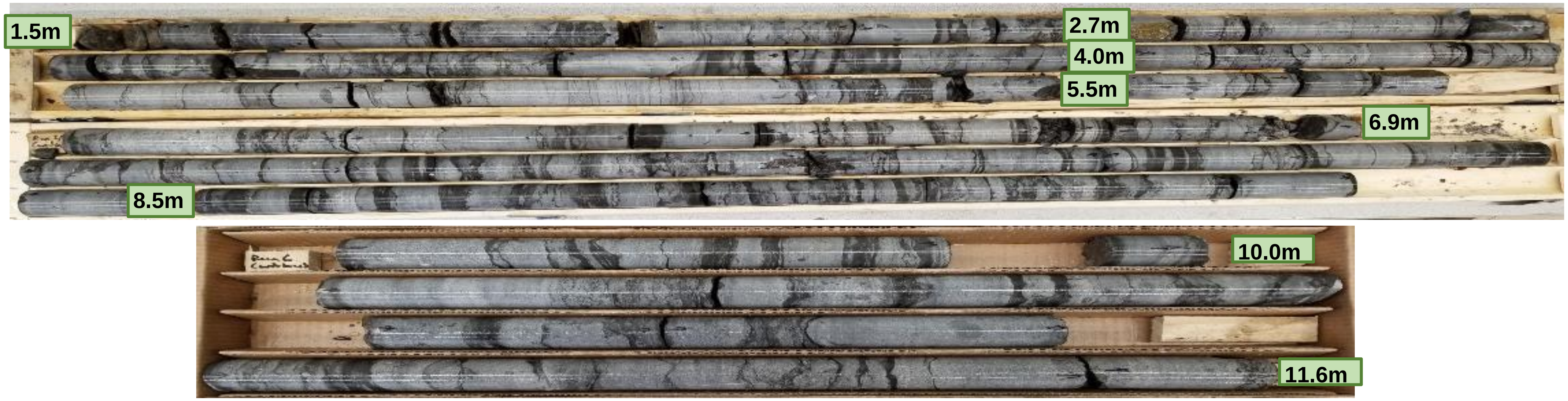
borehole no. **BH7**	core runs Run 1: 13.0m - 14.5m Run 2: 14.5m - 16.3m Run 3: 16.3m - 17.8m	PROJECT Preliminary Geotechnical Investigation - Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.	project no. OTT-00250193-C0
date cored Feb 20, 2019		ROCK CORE PHOTOGRAPHS	**Figure: B-6**

DRY BEDROCK CORES

WET BEDROCK CORES



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borehole no. **BH8**	core runs Run 1: 1.5m - 2.7m Run 2: 2.7m - 4.0m Run 3: 4.0m - 5.5m Run 4: 5.5m - 6.9m Run 5: 6.9m - 8.5m Run 6: 8.5m - 10.0m Run 7: 10.0m - 11.6m	PROJECT Preliminary Geotechnical Investigation - Block 211 + Land North of Block 211, Chaudiere Island, Ottawa, ON.	project no. OTT-00250193-C0
date cored Feb 25, 2019		ROCK CORE PHOTOGRAPHS	**Figure: B-7**

Appendix C: Multi-channel Analysis of Surface Waves (MASW) Survey Report

GEOPHYSICS GPR INTERNATIONAL INC.

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

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Fax : (514) 521-4128
info@geophysicsgpr.com
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May 21st, 2019

Transmitted by email: ismail.taki@exp.com
Our Ref.: GPR-19-01369

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

Subject: **Shear Wave Velocity Soundings for Site Classes Determination**
Chaudière Island, Ottawa (ON)
[Project: OTT-00250193]

Dear Sir,

Geophysics GPR International Inc. has been requested by **exp** Services Inc. to carry out seismic shear wave surveys on the Chaudière Island, in Ottawa (ON). The geophysical investigations used the Multi-channel Analysis of Surface Waves (MASW), the Extended SPatial AutoCorrelation (ESPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocities values were calculated.

The surveys were carried out, on May 3rd, by Mr. Alexis Marchand and Mr. Dominic Déraps, tech. The 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.

METHODS PRINCIPLES

MASW Survey

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 usually 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.

Seismic Refraction Survey

The method consists in measuring the propagation delays of the direct and refracted seismic waves (P and/or S) produced by an artificial source in the axis of a seismic linear spread. The seismic velocities of the materials can be directly calculated, then the refractors depths.

INTERPRETATION METHODS

MASW Surveys

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.

Seismic Refraction surveys

The General Reciprocal Method was used, with signal sources at both ends of the seismic spreads, to consider seismic wave propagation for two opposite directions. The seismic wave's arrival times were identified for each geophone. The measurements were realised to calculate the rock depth (using P waves).

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

SURVEY DESIGN

The seismic acquisition spreads were located on a possible filled crevasse (L1-East side), and over a shallow rock covered with fill material (L2-West side).

The East Side Seismic Section (L1), was 69 m long, using 3 metres geophones spacing for the principal spread, and 1 metre geophones spacing for the spread dedicated to shallow materials. Some possible side effects could have limited the effective depth of investigation.

The West Side Seismic Section (L2), was 57.5 m long, using 2.5 metres geophones spacing for the principal spread, and 0.5 metre geophones spacing for the spread dedicated to shallow materials.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and 4096 data, sampled at 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.

Unlike the refraction method, which allows producing a result point beneath each geophone, 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 made with a seismograph Terraloc MK6 (from ABEM Instrument), and the geophones were 4.5 Hz. A 10 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic spreads.

RESULTS

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. This value represents an equivalent homogeneous single layer response.

L1-East:

From seismic refraction, the rock depth calculation was not possible due to the site surrounding noise, and possibly due to the crevasse sides effects. The exp geotechnical boreholes results presented a rock between 11 and 13 metres deep. The seismic shear wave velocity of the sound shallow part of the rock was calculated between 1805 and 1820 m/s (cf. Figure 5). These results were used as initial parameters for the basic geophysical model, prior to the MASW dispersion curves inversions.

The MASW calculated velocities of the seismic shear wave (V_S) results are illustrated at Figure 6 and the numerical results are also presented at Table 1. Possibly due to sides effects limitations, the maximal resolvable V_S depth was approximately 15 to 18 metres deep. The seismic refraction results (V_S) were extrapolated until 30 metres deep to allow the $\overline{V}_{S30}$ value calculation.

The calculated $\overline{V}_{S30}$ value of the actual site is 756.0 m/s, corresponding to the Site Class "C" (cf. Table 1).

L2-West:

From seismic refraction, the rock depth was calculated between 2.3 and 6 metres deep (±1 metre). The exp geotechnical boreholes results presented the rock between 1.5 and 4 metres deep. The seismic shear wave velocity of the sound shallow portion of the rock was calculated between 1755 and 1985 m/s (cf. Figure 7), with and average of 1870 m/s. These results were used as initial parameters for the basic geophysical model, prior to the MASW dispersion curves inversions.

The MASW calculated velocities of the seismic shear wave (V_S) results are illustrated at Figure 8 and the numerical results are also presented at Table 2.

The calculated $\overline{V}_{S30}$ value of the actual site is 975.3 m/s, corresponding to the Site Class “B” (cf. Table 2). Less than 3 metres of unconsolidated material should take place between the rock and the lower part of the foundation to allow considering this Site Class. In the case than less than 1.15 metre could take place between the rock and the lower portion of the foundations, the Site Class “A” could be used.

CONCLUSION

Geophysical surveys were carried out on the Chaudière Island, in Ottawa (ON). The site was located north-east of Booth Street. As previous geotechnical boreholes revealed two different rock depths areas, two seismic surveys were produced to characterize each of them. The first was located over a possible filled crevasse (L1-East), while the second one was over a shallow rock covered with fill materials (L2-West). The seismic surveys used the MASW, ESPAC analysis methods, as well as the complementary seismic refraction method, to calculate the $\overline{V}_{S30}$ values for the Site Classes determination. The $\overline{V}_{S30}$ calculations are presented in Table 1 and Table 2.

The calculated $\overline{V}_{S30}$ value of the actual East Site area (possible crevasse) is 756 m/s corresponding to the Site Class "C" ($360 < \overline{V}_{S30} \leq 760$ m/s), as determined through the MASW, ESPAC and seismic refraction methods, Table 4.1.8.4.A of the NBC, and the Building Code, O. Reg. 332/12.

The calculated $\overline{V}_{S30}$ value of the actual West Site area is 975 m/s, corresponding to the Site Class "B" ($760 < \overline{V}_{S30} \leq 1500$ m/s). Nevertheless, the Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated materials between the rock and the bottom of the spread footing or mat foundations. Considering the case than less than 1.15 metre of unconsolidated materials could take place between the rock and the lower portion of the foundations, 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, soft 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.

P.Eng.

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

LICENSED PROFESSIONAL ENGINEER
J.-L. ARSENAULT
100148122
PROVINCE OF ONTARIO
2019-05-21

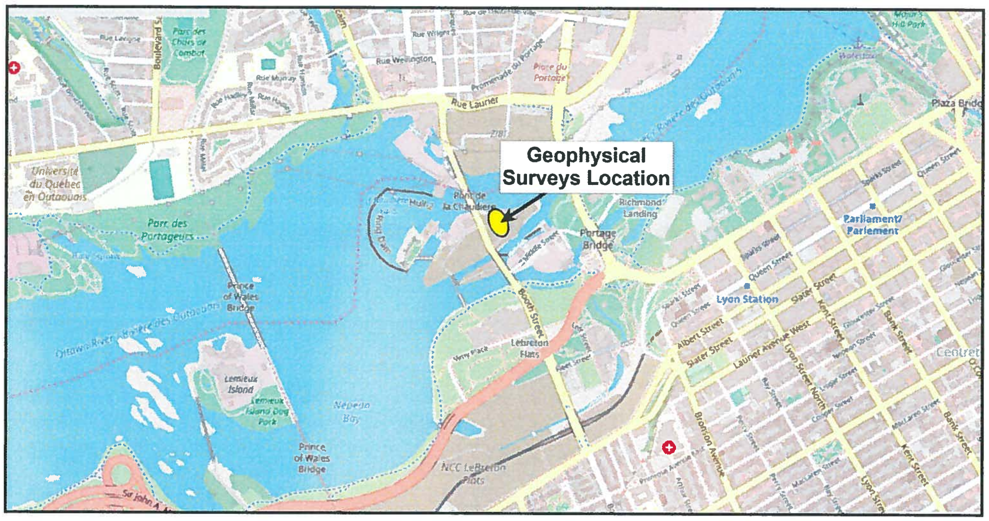

Geophysical Surveys Location

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

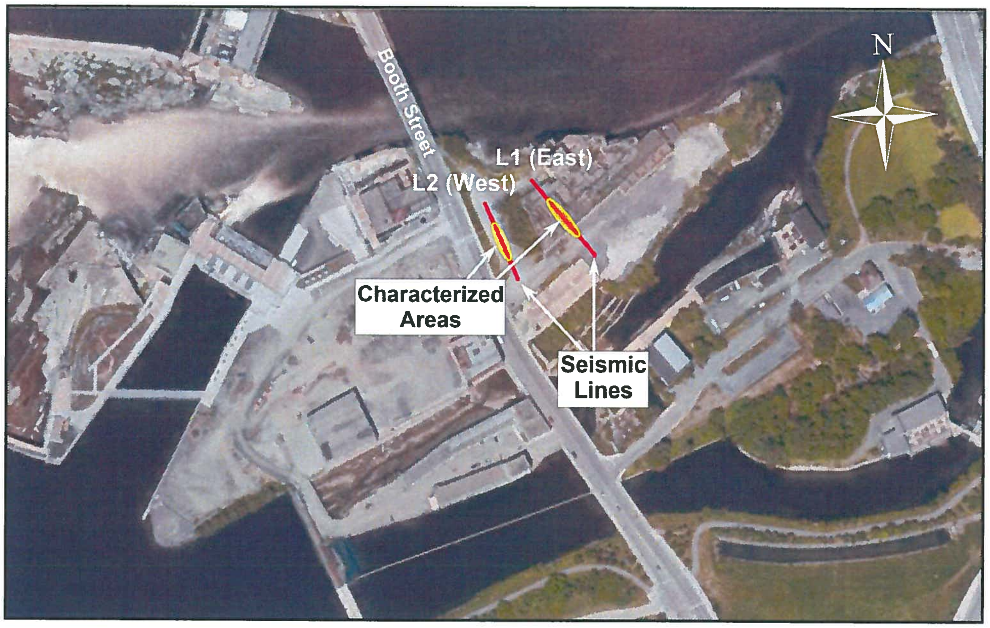

Characterized Areas
Seismic Lines

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

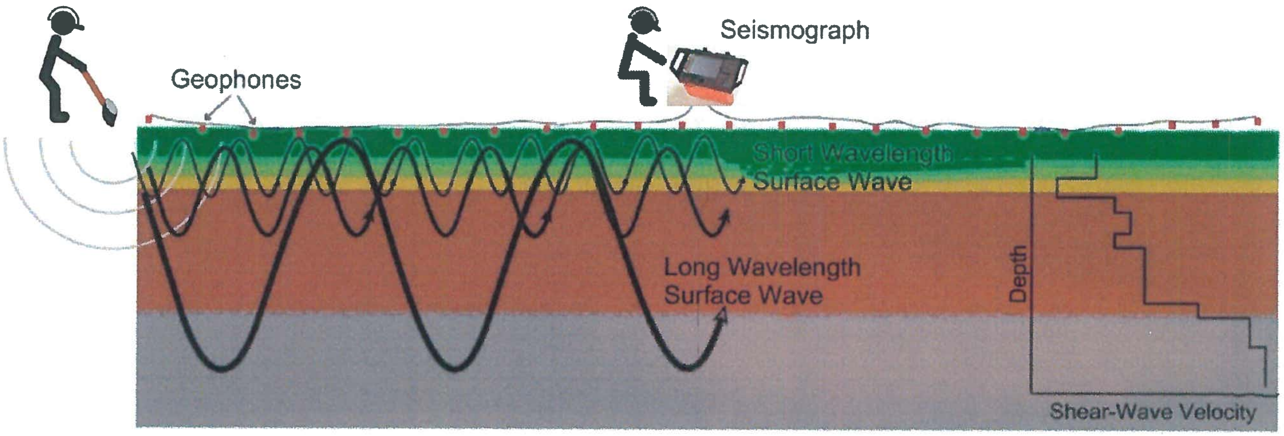
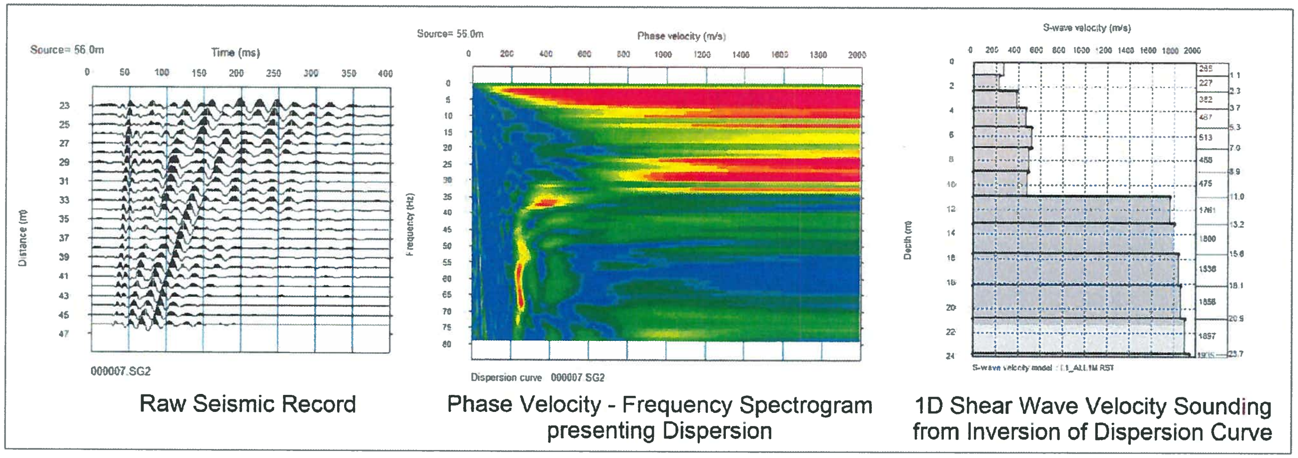


Figure 3: MASW Operating Principle



Raw Seismic Record

Phase Velocity - Frequency Spectrogram presenting Dispersion

1D Shear Wave Velocity Sounding from Inversion of Dispersion Curve

Figure 4: Example of a MASW/ESPAC record, Phase Velocity - Frequency curve and resulting 1D Shear Wave Velocity Model (L1-East ; dX = 1m)

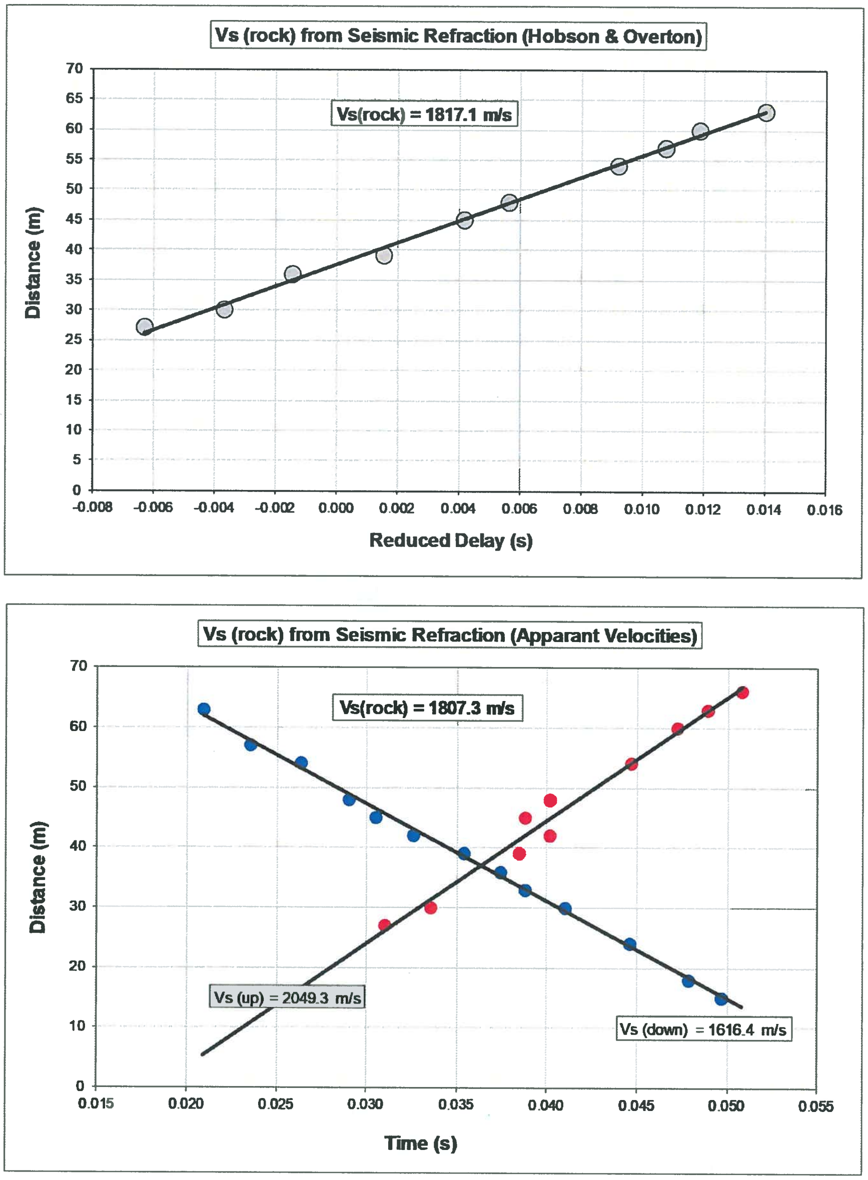


Figure 5: L1 (East) Rock V_S from Seismic Refraction

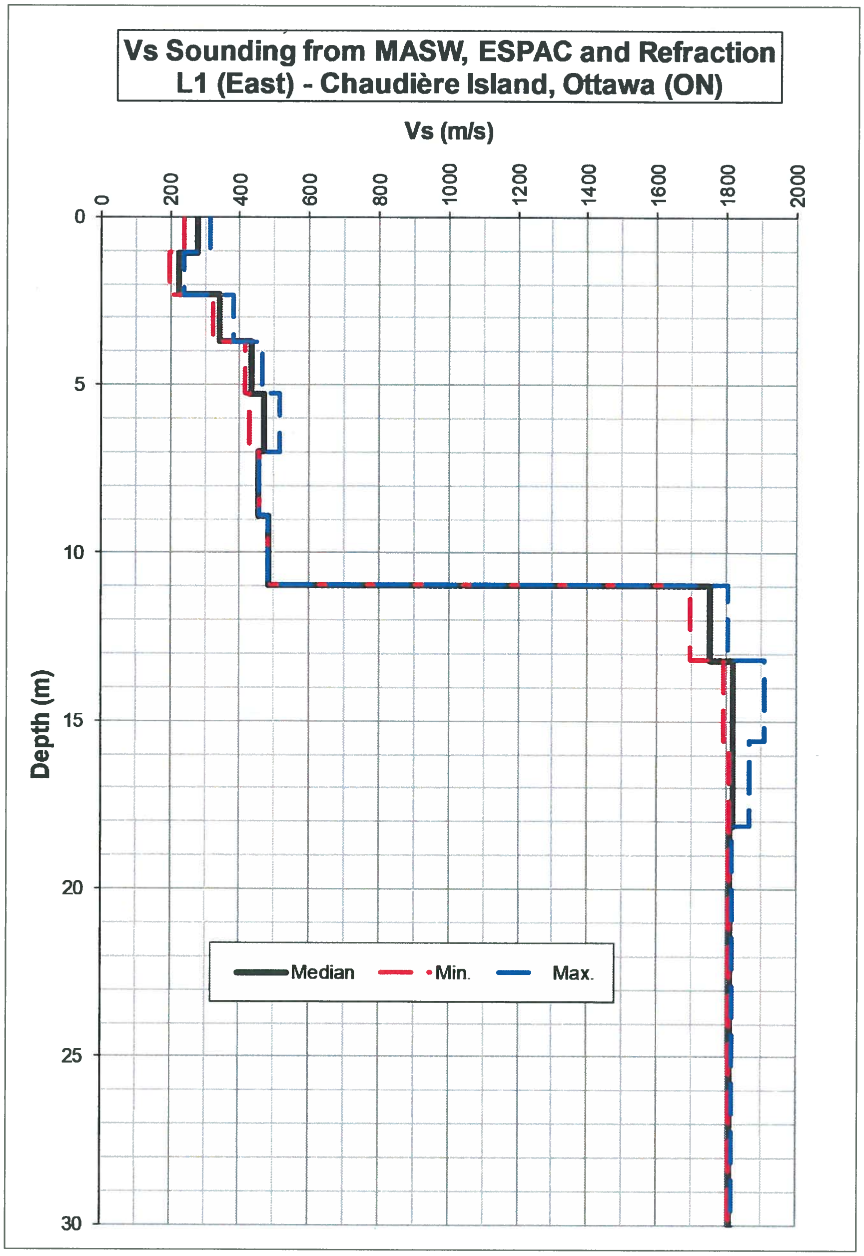


Figure 6: L1 (East) MASW Shear-Wave Velocities Sounding

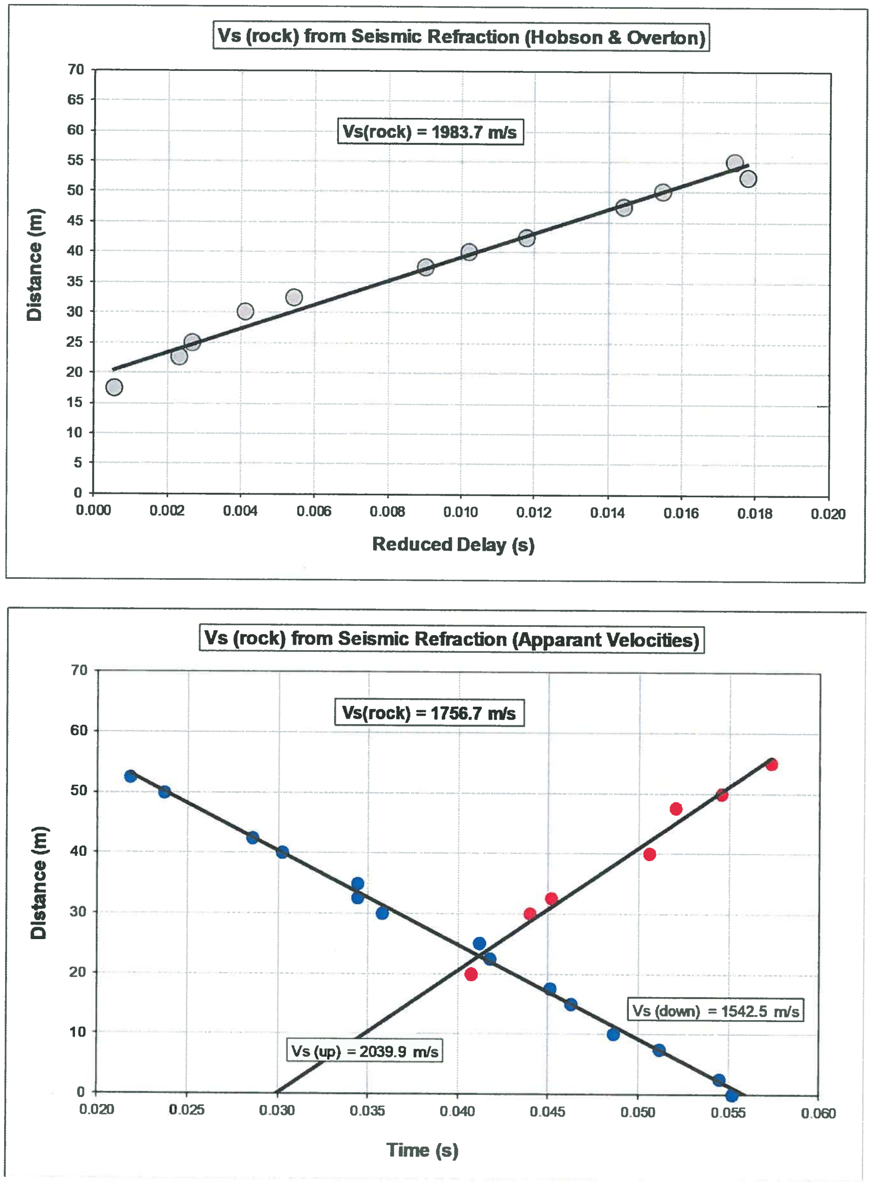


Figure 7: L2 (West) Rock V_S from Seismic Refraction

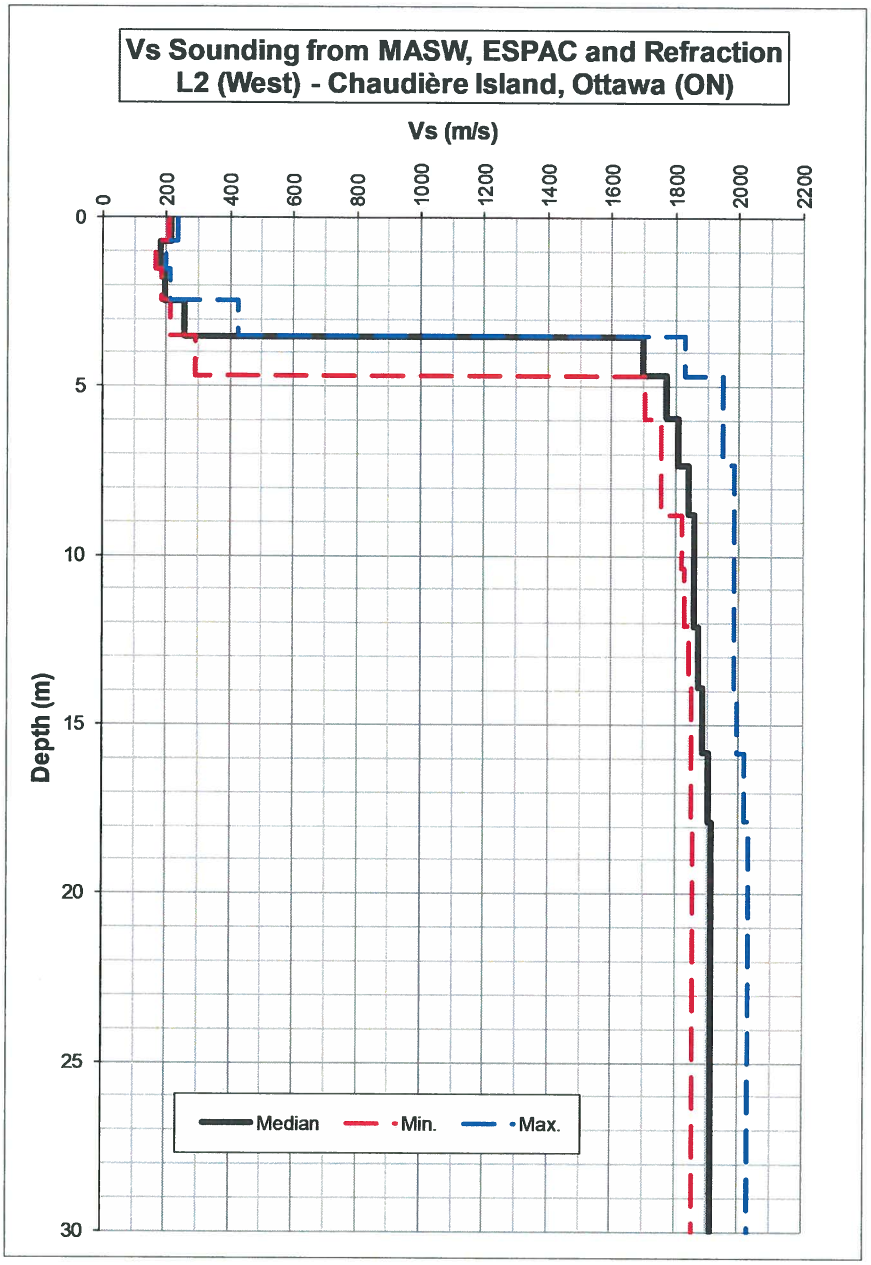


Figure 8: L2 (West) MASW Shear-Wave Velocities Sounding

TABLE 1

L1 (East) - V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Med. Vs	Cumulative Delay	Vs at Given Deth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	240.3	277.4	314.5					
1.07	197.6	225.8	240.7	1.07	1.07	0.003863	0.003863	277.4
2.31	322.8	341.7	382.8	1.24	2.31	0.005476	0.009338	247.1
3.71	413.4	434.3	467.0	1.40	3.71	0.004100	0.013439	276.0
5.27	426.7	470.9	515.2	1.57	5.27	0.003605	0.017044	309.5
7.01	456.5	456.5	456.5	1.73	7.01	0.003675	0.020720	338.1
8.90	482.7	482.7	482.7	1.90	8.90	0.004153	0.024872	357.9
10.96	1697.5	1751.2	1804.8	2.06	10.96	0.004269	0.029141	376.2
13.19	1792.8	1817.1	1910.4	2.23	13.19	0.001271	0.030412	433.6
15.58	1807.3	1817.1	1869.0	2.39	15.58	0.001315	0.031727	491.0
18.13	1807.3	1812.2	1817.1	2.55	18.13	0.001406	0.033133	547.2
20.85	1807.3	1812.2	1817.1	2.72	20.85	0.001501	0.034634	602.1
23.74	1807.3	1812.2	1817.1	2.88	23.74	0.001592	0.036225	655.2
26.79	1807.3	1812.2	1817.1	3.05	26.79	0.001683	0.037908	706.6
30				3.21	30.00	0.001774	0.039682	756.0

V_{S30} (m/s)	**756.0**
Site Class	**C**

TABLE 2

L2 (West) - V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Med. Vs	Cumulative Delay	Vs at Given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	206.6	215.8	236.2					
0.71	167.9	183.2	196.9	0.71	0.71	0.003310	0.003310	215.8
1.54	186.8	196.6	211.7	0.82	1.54	0.004498	0.007808	197.0
2.47	211.2	255.2	425.7	0.93	2.47	0.004750	0.012558	196.9
3.52	290.9	1698.4	1827.8	1.04	3.52	0.004090	0.016648	211.2
4.67	1704.9	1771.8	1949.8	1.15	4.67	0.000679	0.017328	269.5
5.93	1756.7	1806.8	1950.1	1.26	5.93	0.000713	0.018041	328.9
7.31	1756.7	1840.7	1983.7	1.37	7.31	0.000760	0.018801	388.7
8.79	1821.5	1858.4	1983.7	1.48	8.79	0.000806	0.019607	448.4
10.38	1826.8	1857.2	1983.7	1.59	10.38	0.000857	0.020465	507.4
12.09	1842.6	1872.7	1983.7	1.70	12.09	0.000917	0.021382	565.3
13.90	1849.7	1887.5	1992.6	1.81	13.90	0.000968	0.022350	622.0
15.82	1853.6	1904.5	2015.3	1.92	15.82	0.001019	0.023369	677.2
17.86	1855.8	1915.9	2032.7	2.03	17.86	0.001067	0.024436	730.8
24.29	1867.7	1925.0	2040.3	6.43	24.29	0.003355	0.027791	873.9
30				5.71	30.00	0.002968	0.030760	975.3

V_{S30} (m/s)	**975.3**
Site 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 and the underside of footing or mat foundations.

Appendix D: 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-250193-CO

AGAT WORK ORDER: 19Z449568

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Supervisor

DATE REPORTED: Mar 29, 2019

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

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

Certificate of Analysis

AGAT WORK ORDER: 19Z449568

PROJECT: OTT-250193-CO

CLIENT NAME: EXP SERVICES INC

ATTENTION TO: Susan Potyondy

SAMPLING SITE:Zibi - Botth St.

SAMPLED BY:exp

Inorganic Chemistry

DATE RECEIVED: 2019-03-25

DATE REPORTED: 2019-03-29

			SAMPLE DESCRIPTION:	BH5 Run 8 45'5"- 45'9"	BH5 Run 9 51'7"- 52'4"	BH7 Run 1 45'0"- 45'5"	BH7 Run 2 52'0"- 52'4"	BH7 Run 3 59'1"- 59'6"
			SAMPLE TYPE:	Rock	Rock	Rock	Rock	Rock
			DATE SAMPLED:	2019-02-22	2019-02-22	2019-02-20	2019-02-20	2019-02-20
Parameter	**Unit**	**G / S**	**RDL**	9989214	9989217	9989218	9989219	9989220
pH (2:1)	pH Units		N/A	8.63	8.72	8.38	8.51	8.53
Resistivity (2:1)	ohm.cm		1	5260	5430	3730	4550	5920
Chloride (2:1)	µg/g		2	38	29	66	45	39
Sulphate (2:1)	µg/g		2	10	10	67	44	35

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

9989214-9989220 As this analysis was performed on a sample matrix which is outside of the scope of our test method it is deemed non-routine and therefore, no information is available for Accuracy, Precision or Measurement Uncertainty.
Samples were received and analyzed beyond recommended hold times.
EC, pH, Chloride and Sulphate were determined on the DI water extract obtained from the 2:1 leaching procedure (2 parts DI water:1 part solid sample).
Resistivity is a calculated parameter.

Certified By: Amanjot Bhela

AMANJOT BHELA

Quality Assurance

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-250193-CO
SAMPLING SITE:Zibi - Botth St.

AGAT WORK ORDER: 19Z449568
ATTENTION TO: Susan Potyondy
SAMPLED BY:exp

Soil Analysis

RPT Date: Mar 29, 2019			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Inorganic Chemistry															
pH (2:1)	9989214	9989214	8.63	8.70	0.8%	N/A	101%	90%	110%	NA			NA		
Chloride (2:1)	9989214	9989214	38	38	0.3%	< 2	90%	70%	130%	111%	70%	130%	93%	70%	130%
Sulphate (2:1)	9989214	9989214	10	10	NA	< 2	110%	70%	130%	114%	70%	130%	92%	70%	130%

Comments: NA signifies Not Applicable.
Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL

Amanjot Bhela

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
AMANJOT BHELA
CHEMIST

Certified By: ______________________

Method Summary

CLIENT NAME: EXP SERVICES INC

PROJECT: OTT-250193-CO

SAMPLING SITE:Zibi - Botth St.

AGAT WORK ORDER: 19Z449568

ATTENTION TO: Susan Potyondy

SAMPLED BY:exp

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

AGAT Laboratories

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

Laboratory Use Only

Work Order #: 19T449568

Cooler Quantity: One

Arrival Temperatures: 19.6 | 19.8 | 19.1
9.6 | 9.8 | 18.4

Custody Seal Intact: ☐Yes ☐No ☐N/A

Notes: no ice (a ice)

Chain of Custody Record

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

Report Information:

Company: Exp Services
Contact: Susan Potyondy
Address: 2650 Queensview drive Suite 100
Ottawa ON K2B 8H6
Phone: 613-688-1899 Fax:
Reports to be sent to:
1. Email: Susan.Potyondy@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-250193-CO
Site Location: Zibi - Booth st
Sampled By: exp
AGAT Quote #: PO:
Please note: If quotation number is not provided, client will be billed full price for analysis.

Invoice Information:

Bill To Same: Yes ☒ No ☐

Company:
Contact:
Address:
Email:

Sample Matrix Legend

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

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI Y / N	O. Reg 153 Metals and Inorganics	O. Reg 153 ☐ All Metals ☐ 153 Metals (excl. Hydrides) ☐ Hydride Metals ☐ 153 Metals (Incl. Hydrides)	O. Reg 153 ORPs: ☐ B-HWS ☐ Cl⁻ ☐ CN⁻ ☐ Cr⁶⁺ ☐ EC ☐ FOC ☐ Hg ☐ pH ☐ SAR	Full Metals Scan	Regulation/Custom Metals	Nutrients: ☐ TP ☐ NH₃ ☐ TKN ☐ NO₃ ☐ NO₂ ☐ NO₃+NO₂	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	Sulphates	Chlorides	Electro Resistivity	
BH 5 Run 8 45'5"-45'9"	Feb 22/19																				✓	✓	✓	✓	
BH 5 Run 9 51'7"-52'4"	"																				✓	✓	✓	✓	
BH 7 Run 1 45'0-45'5"	Feb 20/19																				✓	✓	✓	✓	
BH 7 Run 2 52'0-52'4"	↓																				✓	✓	✓	✓	
BH 7 Run 3 59'1"-59'6"																					✓	✓	✓	✓	

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time	
Ryan DiGiuseppe	Mar 22/19	6:00pm	C Berthelet	19-03-25	9h11	
UPS to FedEx	19-03-25	16h00	Sima	19/3/26	955	Page 1 of 1
						Nº: T 078138

List of Distribution

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Zibi Canada, Brownsfield Coordinator, Taryn Glancy - tglancy@zibi.ca