Best Western Plus

Geotechnical Investigation

Type of Document
Final (supersedes June 20, 2018 report)

Project Name
Proposed Vertical Extension of Best Western Plus Hotel
1274 Carling Avenue, Ottawa, Ontario

Project Number
OTT-00245894-A0

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Date Submitted
August 7, 2018

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Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation recently completed for the proposed vertical extension of the Best Western Plus hotel located at 1274 Carling Avenue, Ottawa, ON. Terms of reference for the geotechnical investigation is defined under the March 22, 2018 engineering services assignment and July 4, 2018 proposal for additional geotechnical investigation. In addition to the geotechnical investigation, EXP's scope of work for this project also includes structural design, mechanical and electrical design, building envelope review and infrastructure design. Written authorization for EXP to proceed with this work was provided by Best Western Plus. The other engineering services to be completed by EXP will be presented under separate covers.

It is our understanding that plans call for the design and construction of two (2) additional floors above the existing north wing of the hotel to match the existing four storey central building and south wing of the hotel. The geotechnical investigation forms part of the engineering services required to support the proposed vertical extension.

The fieldwork for the geotechnical investigation was undertaken on May 3 and 4 and July 25, 2018 and consisted of drilling four (4) boreholes (Borehole Nos.1 to 4). The boreholes (Borehole Nos.1 to 3) located outside the north wing of the hotel were drilled to depths ranging from 9.4 m to 19.2 m below existing grade. The borehole undertaken in the basement of the north wing of the hotel (Borehole No. 4) was drilled to a 4.1 m depth below the basement floor slab and through the existing footing into the underlying founding soils. The purpose of the interior borehole is to confirm the 'as built' details of the existing basement slab and one (1) footing and to confirm the footing founding depth, founding soil type and soil strength profile. The fieldwork was supervised on a full-time basis by a representative from EXP.

A Multi-channel Analysis of Surface Waves (MASW) survey was conducted on May 23, 2018 by Geophysics GPR International Inc. The purpose of the MASW survey was to determine the site classification for seismic response and assess the liquefaction potential of subsurface soils.

The exterior borehole information indicates the subsurface conditions consist of a surficial pavement structure underlain by fill to depths ranging from 0.7 m to 2.4 m (Elevation 74.2 m to 72.6 m) followed by native sensitive marine clay to 3.3 m and 3.8 m depths (Elevation 71.5 m to 71.71 m), glacial till to 15.0 m depth (Elevation 59.9 m) and limestone bedrock. The groundwater level is at 2.5 m to 2.7 m depths (Elevation 72.3 m).

The interior borehole, Borehole No.4, indicates the basement floor slab is 110 mm thick and is reinforced with wire mesh. Granular and sand fill was encountered between the underside of the slab and the top of the footing and is approximately 400 mm thick. The footing extends to a 1.0 m depth (Elevation 71.6 m) and is founded on the native grey clay. The footing is reinforced with 15 M reinforcing bars located at 130 mm and 305 mm from the top of the footing.

The native grey clay extends to 2.1 m depth below the basement slab (Elevation 70.5 m) and is underlain by glacial till. The groundwater level was measured at 2.1 m depth below the basement floor slab (Elevation 70.5 m).

Based on a review of the 1981 Architectural Drawing Nos. A2 and A6, the basement slab elevation of the north wing is at Elevation 72.55 m rising to Elevation 73.01 m at the central building of the hotel. Based on the interior borehole located in the basement of the north wing of the hotel, the founding elevation of the footing is at 71.6 m.

The site has been classified as **Class C** for seismic site classification in according with the 2012 Ontario Building Code (2012 OBC). The on-site soils are not susceptible to liquefaction during a seismic event.

The following foundation alternatives are considered feasible to accommodate the increased loading from the additional two (2) storeys proposed for the north wing of the hotel:

- Utilize the existing footings to support the increased loading, provided the soils supporting the existing footings have the capacity to carry the increased loading. Depending on the magnitude of the increased loading and capacity of the underlying supporting soils, the size of the existing footings may need to be increased to reduce the load stress imposed on the supporting soils. Based on the borehole information, strip footings having a maximum width of 3 m and square footings having a maximum size of 5 m by 5 m and founded on the native stiff to very stiff brown and grey clay or loose to very dense glacial till may be designed for a bearing pressure at serviceability limit state (SLS) of 96 kPa and a factored geotechnical resistance at ultimate limit state (ULS) of 150 kPa. The factored ULS value includes a resistance factor of 0.5 in accordance with the 2006 Canadian Foundation Engineering Manual (CFEM).

- If insufficient capacity is available in the existing footings and founding soil to support the additional building storeys, additional capacity may be achieved by underpinning the existing footings using either helical piers or micro-piles extended to the glacial till or bedrock.

The subsurface basement walls should be designed to resist lateral static and dynamic (seismic) earth forces.

Excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA). Seepage of the surface and subsurface water into the excavations is anticipated. However, it should be possible to collect any water entering the excavations in perimeter ditches and to remove it by pumping from sumps.

The above and other related considerations are discussed in greater detail in the report.

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1 Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation recently completed for the proposed vertical extension of the Best Western Plus hotel located at 1274 Carling Avenue, Ottawa, ON. Terms of reference for the geotechnical investigation is defined under the March 22, 2018 engineering services assignment and July 4, 2018 proposal for additional geotechnical investigation. In addition to the geotechnical investigation, EXP's scope of work for this project also includes structural design, mechanical and electrical design, building envelope review and infrastructure design. Written authorization for EXP to proceed with this work was provided by Best Western Plus. The other engineering services to be completed by EXP will be presented under separate covers.

It is our understanding that plans call for the design and construction of two (2) additional floors above the existing north wing of the hotel to match the existing four storey central building and south wing of the hotel.

The geotechnical investigation forms part of the engineering services required to support the proposed vertical extension of the Best Western Plus hotel.

The geotechnical investigation was undertaken to:

a) Establish the subsurface soil, bedrock and groundwater conditions at three (3) boreholes located outside the north wing of the hotel and one (1) borehole located inside the hotel building. The purpose of the interior borehole is to confirm the 'as built' details of the existing basement slab and one (1) footing and to confirm the footing founding depth, founding soil type and soil strength profile.

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) Make recommendations regarding feasible foundation alternatives to support the additional building loads imposed by the proposed two (2) storey addition to the north wing of the hotel. Foundation alternatives include the most suitable type of foundations, founding depth and bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance at ultimate limit state (ULS) of the founding strata and comment on the anticipated total and differential settlements of the recommended foundation type.

d) Discuss excavation conditions and de-watering requirements during construction.

e) Provide backfill requirements and suitability of on-site soils for backfilling purposes.

f) Discuss recommendations regarding the reinstatement of the existing pavement structure.

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

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

2 Background Information

The relevant sections of the following available drawings and letter type report regarding the existing original hotel building and the north wing were used as reference in the preparation of this geotechnical report:

- 2015 Report – *"Proposed 2 Story Expansion to Existing Best Western Hotel, 1274 Carling Avenue, Ottawa, ON.,"* dated April 28, 2015 and prepared by MTE Consultants Inc. (MTE File: C40189-100).
- 1981 Architectural Drawing Nos. A2, A3, A5 to A7, A9, A14 to A16, A18 and A19 prepared by Craig, Kohler and Dickey Architects.
- 1981 Structural Drawing Nos. S1 to S6 prepared by Oliver Mangione and McCalla (now EXP Services Inc.).
- 1972 Structural Drawing Nos. S1 to S5 prepared by J.S. Hall and Associates Consulting Civil Engineers.

Based on a review of the above drawings, relevant information used in the preparation of this geotechnical engineering report include the following:

- The north wing is a two (2) storey building with basement. The elevation of the ground floor is at Elevation 75.65 m. The elevation of the basement floor is at Elevation 72.55 m rising to Elevation 73.01 m at the existing central building of the hotel.
- Plan drawings indicate strip footings along the exterior east and west foundation walls are 2.0 m in width and strip footings along the interior column lines (parallel to the east and west walls) are 2.75 m in width. The strip footing along the north wall is 0.8 m in width.
- The 1981 structural drawings indicate the footings have been designed for an allowable bearing pressure of 95 kPa when bearing on the dense glacial till and 70 kPa when bearing on the firm grey silty clay.
- The 1972 structural drawings for the central building indicate that the footings have been designed for an allowable bearing pressure of 96 kPa (2,000 lbs/ft^2).
- The MTE report indicates the following options for the proposed two (2) storey vertical extension:
    - Have the existing bearing pressure value of the founding soil re-checked by a qualified geotechnical engineer to determine if the bearing pressure value provided in the 1981 drawings are conservative and if a higher bearing pressure may be available in the founding soils.
    - Underpin the existing footings with either micropiles or helical piers to support the additional loads. Removal of the existing slab would be required for this option.
    - Extend the existing footing to increase the bearing area and reduce bearing pressure. Removal of the existing slab would be required for this option.

3 Geology of the Site

3.1 Surficial Geology

Review of published geology maps indicates the site is covered by off-shore marine deposits consisting of silt and clay underlain by glacial till.

3.2 Bedrock Geology

Review of published geology maps indicates the site is underlain by limestone bedrock of the Ottawa Formation. The limestone bedrock may have shaley partings.

4 Site Description

The subject site is located on the south side of Carling Avenue between Merivale Road and Archibald Street in Ottawa, ON. The site is currently occupied by the two (2) to four (4) storey hotel building surrounded by paved parking lots and access roads. The location of the site is shown in Figure 1.

The topography of the site is relatively flat with ground surface elevations ranging from Elevation 75.02 m to 74.83 m at the three (3) boreholes located around the north wing of the hotel.

5 Procedure

The fieldwork for the geotechnical investigation was undertaken on May 3 and 4 and July 25, 2018 and consisted of drilling four (4) boreholes (Borehole Nos.1 to 4). The boreholes (Borehole Nos.1 to 3) located outside the north wing of the hotel were drilled to depths ranging from 9.4 m to 19.2 m below existing grade. The borehole undertaken in the basement of the north wing of the hotel (Borehole No. 4) was drilled to a 4.1 m depth below the basement floor slab. The scope of work included the drilling of two (2) boreholes inside the hotel building. However, due to conflict with underground services, only one (1) borehole could be advanced inside the hotel. The fieldwork was supervised on a full-time basis by a representative from EXP.

The locations and geodetic elevations of the boreholes were established on site by EXP. The elevation of the basement floor slab was taken from the available drawings. The borehole locations are shown on Figure Nos. 2 and 3. Prior to commencement of the fieldwork, the borehole locations were cleared of any underground services by a local cable locating company and EXP.

The exterior boreholes were drilled using a CME-75 truck-mounted drill rig owned and operated by a local drilling contractor. The boreholes were advanced to auger refusal depth of 15.0 m in Borehole No. 1 and to termination depths of 9.3 m and 9.7 m in Borehole Nos. 3 and 2, respectively. Borehole No. 1 was advanced into the bedrock to a 19.2 m termination depth by core drilling technique using an NQ-size core barrel. Auger samples were retrieved in each borehole below the asphaltic concrete layer to a 0.6 m depth. Standard penetration tests (SPT; ASTM 1586) were undertaken within the soil from 0.75 m to 2.6 m depth intervals and the soil samples retrieved by the split-barrel sampler. Relatively undisturbed tube samples (Shelby tube) were retrieved at selected depths within the clay. The undrained shear strength of the clay was measured by conducting penetrometer and in-situ vane tests at selected depth intervals. During rock coring operations, careful records were kept of the wash water return, the colour of wash water, and any sudden drops of the drill rods. Dynamic cone penetration tests (DCPT) were conducted from 9.9 m to cone refusal at 12.3 m depth in Borehole No. 2 and from 9.3 m to cone refusal at 9.4 m depth in Borehole No. 3.

Standpipe piezometers consisting of 19 mm diameter slotted PVC pipe were installed in Borehole Nos. 2 and 3 for long-term monitoring of groundwater levels. The standpipe piezometers were installed in accordance with EXP standard practice and their installation configuration is documented on the respective borehole log.

The interior borehole (Borehole No. 4) was located 1.4 m south of Column Line Nos. D-2 in the basement of the north wing of the hotel. The borehole was advanced using manual drilling equipment. The borehole cored through the basement floor slab and footing and was advanced into the underlying soil by conducting the SPT at 0.6 m depth interval to the 4.1 m termination depth below the basement floor slab. SPT was also conducted from the underside of the basement slab to the top of the footing. For the SPT, a hammer having a third weight of the standard hammer weight was used and the N-values shown on the attached borehole log have been corrected to the standard hammer weight.

Upon completion of the fieldwork, all the soil samples and rock cores were transported to the EXP laboratory located in the City of Ottawa. All soil samples and rock cores were visually examined in the laboratory by a senior geotechnical engineer for textural classification. The engineer also assigned the laboratory testing, which consisted of performing the following tests on soil samples and rock cores. All tests were conducted in accordance with the American Society for Testing and Materials (ASTM).

Soil Samples:

Natural Moisture Content .... 35 tests

Natural Unit Weight .... 6 tests

Grain-Size Analysis .... 5 tests

Atterberg Limits .... 5 tests

pH, Sulphate, Chlorides and Resistivity Analyses .... 2 tests

Rock Cores:

Unit Weight and Unconfined Compressive Strength Tests on Rock Cores .... 3 tests

5.1 Multi-channel Analysis of Surface Waves (MASW) Seismic Survey

A Multi-channel Analysis of Surface Waves (MASW) survey was conducted on May 23, 2018 by Geophysics GPR International Inc. The purpose of the MASW survey was to determine the site classification for seismic response and assess the liquefaction potential of subsurface soils.

6 Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface soil, bedrock and groundwater conditions determined from the boreholes is given on the attached Borehole Logs, Figure Nos. 4 to 7. 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 soil and bedrock conditions with depth and groundwater level measurements.

6.1 Exterior Boreholes (Borehole Nos. 1 to 3)

6.1.1 Asphaltic Concrete

A 75 mm and 90 mm thick surficial asphaltic concrete layer was encountered in all three (3) boreholes.

6.1.2 Fill

Fill was contacted beneath the topsoil layer in all three (3) boreholes and extends to depths ranging from 0.7 m to 2.4 m (Elevation 74.2 m to 72.6 m). The fill consists of a silty sand and gravel. Based on the standard penetration test N-values of 2 and 3, the fill is in a very loose state. The natural moisture content ranges from 2 percent to 7 percent.

6.1.3 Clay

Sensitive marine clay was contacted beneath the fill at 0.7 m to 2.4 m depths (Elevation 74.2 m to 72.6 m) in the three (3) boreholes. The clay consists of an upper desiccated brown crust underlain by grey clay.

Brown Clay (Desiccated Crust)

The upper brown desiccated clay crust extends to depths ranging from 2.3 m to 3.2 m (Elevation 72.5 m and 71.8 m). The clay crust is approximately 0.8 m to 1.7 m thick. The undrained shear strength of the crust is 106 kPa to greater than 215 kPa indicating a very stiff to hard consistency. The sensitivity values of the clay are 2.6 and 6.3, indicating the sensitivity of the clay may be described as medium sensitive to sensitive. The natural moisture content and unit weight of the crust are 33 percent to 59 percent and 17.4 kN/m^3 to 18.6 kN/m^3 respectively.

The results from grain-size analysis and Atterberg limit determination of one (1) sample of the brown clay crust are summarized in Tables I and II. The grain-size distribution curve is shown in Figure 8.

Table I: Summary of Results from Grain-size Analysis – Brown Clay Sample

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain Size Analysis (%)		
		Gravel	Sand	Fines (Silt and Clay)
BH3 – SS3	1.5 – 2.0 (73.3 – 72.8)	0	5	95

Table II: Summary of Atterberg Limit Results – Brown Clay Sample

Borehole No. - Sample No.	Depth (Elevation) (m)	Atterberg Limit Results (%)			
		w_n	LL	PL	PI
BH3 – SS3	1.5 – 2.0 (73.3 – 72.8)	55	68	26	42

W_n: Natural Moisture Content; **LL**: Liquid Limit; **PL:** Plastic Limit; **PI**: Plasticity Index
(1): Refer to Casagrande Plasticity Chart (1932).

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

Grey Clay

The upper brown desiccated clay crust is underlain by the grey clay at 2.3 m to 3.2 m depths (Elevation 72.5 m and 71.8 m). The grey clay extends to depths of 3.3 m and 3.8 m (Elevation 71.5 m to 71.1 m) and is 0.6 m to 1.4 m thick. The undrained shear strength of the clay ranges from 53 to 67 kPa, indicating the grey clay has a stiff consistency. The sensitivity values of 1.0 to 5.6 indicate the clay may be described as having a sensitive to low to medium sensitivity. The clay has a moisture content of 28 percent to 65 percent.

The results from grain-size analysis and Atterberg limit determination of one (1) sample of the grey clay are summarized in Tables III and IV. The grain-size distribution curve is shown in Figure 9.

Table III: Summary of Results from Grain-size Analysis – Grey Clay Sample

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain Size Analysis (%)		
		Gravel	Sand	Fines (Silt and Clay)
BH1 – SS5	3.0 – 3.5 (71.9 – 71.4)	1	2	97

Table IV: Summary of Atterberg Limit Results – Grey Clay Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Atterberg Limit Results (%)			
		w_n	LL	PL	PI
BH1 – SS5	3.0 – 3.5 (71.9 – 71.4)	65	55	21	34

W_n: Natural Moisture Content; **LL**: Liquid Limit; **PL:** Plastic Limit; **PI**: Plasticity Index

(1): Refer to Casagrande Plasticity Chart (1932).

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

6.1.4 Glacial Till

The grey clay in the three (3) boreholes is underlain by glacial till at 3.3 m and 3.8 m depths (Elevation 71.5 m to 71.1 m). The glacial till extends to a 15.0 m depth (Elevation 59.9 m) in Borehole No. 1. The glacial till extends to 9.8 m depth (Elevation 65.2 m) in Borehole No. 2 and is inferred from 9.8 m depth (Elevation 65.2 m) to cone refusal depth of 12.3 m (Elevation 62.7 m). In Borehole No. 3, the glacial till extends from 3.3 m to 9.3 m depths (Elevation 71.5 m to 65.5 m) and is inferred in the lower 100 mm from 9.3 m to cone refusal at 9. 4 m depths (Elevation 65.5 m to 65.4 m). The N values from the SPT of 6 to 79 indicate the glacial till is in a loose to very dense state. The glacial till contains cobbles and boulders likely causing the difficult augering and high N-values observed within the glacial till. The natural moisture content of the glacial till is 7 percent to 21 percent.

The results from the grain-size analysis conducted on selected samples of the glacial till are summarized in Table V. The grain-size distribution curves are shown in Figures 10 to 12.

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

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain Size Analysis (%)		
		Gravel	Sand	Fines (Silt and Clay)
BH1 – SS7	4.6 – 5.2 (70.3 – 69.7)	21	47	32
BH1 – SS10	9.1 – 9.7 (65.8 – 65.2)	28	60	12
BH2 – SS9	7.6 – 8.2 (67.4 – 66.8)	16	47	37

Based on a review of the results of the grain-size analysis, the glacial till may be classified in accordance with the USCS as a silty sand with gravel (SM) to sand with silt and gravel (SW-SM).

6.1.5 Refusal Depths

Auger refusal was met at 15.0 m depth (Elevation 59.9 m) in Borehole No. 1 and cone refusal (from the dynamic cone penetration test) was met at 12.3 m depth (Elevation 62.7 m) in Borehole No. 2 and at 9.4 m depth (Elevation 65.4 m) in Borehole No. 3. It is not known whether refusal was met on inferred boulders or on bedrock.

6.1.6 Limestone Bedrock

The presence of the bedrock was confirmed in Borehole No. 1 by coring the bedrock from 15.0 m to 19.2 m depths (Elevation 59.9 m to 55.7 m).

The Total Core Recovery (TCR) and Rock Quality Designation (RQD) of the cored bedrock is 100 percent and 11 percent to 68 percent respectively, indicating the rock has a very poor to good quality. Rock core photographs are shown in Appendix A.

Unit weight determination and unconfined compressive strength tests were performed on three (3) selected rock cores and the results are presented in Table VI.

Table VI: Unit Weight and Unconfined Compressive Strength Test Results - Bedrock Cores

Borehole No.	Rock Core Depth (Elevation) (m)	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)
1	16.0 – 16.1 (58.9 – 58.8)	26.3	140.4
1	16.4 – 16.5 (58.5 – 58.4)	26.0	152.3
1	17.9 – 18.1 (57.0 – 56.8)	26.0	165.3

Based on a review of the unconfined compressive strength test results, the rock may be classified as having an intact strength of very strong.

6.1.7 Groundwater Levels

Groundwater level observations were made in the standpipe piezometers installed in Borehole Nos. 2 and 3 subsequent to the completion of drilling operations and are shown on the borehole logs. The recent groundwater level observations are summarized in Table VII.

Table VII: Summary of Groundwater Levels in Boreholes

Borehole No.	Ground Surface Elevation (m)	Drill Date	Date of Groundwater Level Measurement (Number of Days After Drilling)	Depth of Groundwater Level (m)	Elevation of Groundwater Level (m)
BH 2	75.02	May 4, 2018	May 16, 2018 (12 days)	2.7	72.3
BH 3	63.49	May 4, 2018	May 16, 2018 (13 days)	2.5	72.3

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

6.2 Interior Borehole (Borehole No. 4)

The interior borehole, Borehole No.4, indicates the basement floor slab is 110 mm thick and reinforced with wire mesh. Granular and sand fill was encountered between the underside of the slab and the top of the footing with a total thickness of approximately 400 mm. The footing extends to a 1.0 m depth below the basement floor (Elevation 71.6 m) and is founded on the native grey clay. The footing is reinforced with 15 M reinforcing bars located 130 mm and 305 mm from the top of the footing.

The native grey clay extends to 2.1 m depth below the basement slab (Elevation 70.5 m). The clay contains trace to some sand and gravel below the 1.9 m depth (Elevation 70.7 m). Based on undrained shear strength measurements of 55 kPa and 70 kPa, the clay has a stiff consistency. The sensitivity values of 4.9 and 5.6 indicate the clay may be described as being sensitive. The clay has a moisture content of 29 percent and 59 percent.

The clay is underlain by glacial till contacted at 2.1 m depth below the basement slab (Elevation 70.5 m). Based on N-values of 7 and 8, the glacial till is in a loose state. The glacial till may contain cobbles and boulders. The natural moisture content of the glacial till is 12 percent to 21 percent. The natural unit weight of the glacial till is 23.2 kN/m^3.

Upon completion of the borehole, the groundwater level was measured at 2.1 m depth below the basement floor slab (Elevation 70.5 m).

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

The geotechnical investigation revealed that the subsurface conditions at the site consist of a surficial pavement structure underlain by fill overlying native clay, glacial till and bedrock.

7.1 Liquefaction Potential

As indicated in Section 6 of this report, the liquid and plastic limits of the brown and grey clay are 55 percent and 68 percent and 21 percent and 26 percent, respectively. The natural moisture content of the brown and grey clay ranges from 33 percent to 65 percent. Based on these results, the clay deposit is not considered to be susceptible to liquefaction during a seismic event, as per Bray et al. (2004) criteria for fine-grained soils as shown in Figure 13.

The loose zone of the glacial till (N-values from SPT of 6 to 8) in Borehole Nos. 1, 2 and 4 at various depths within the till may have a potential to liquefy during a seismic event. However, the MASW survey report shown in Appendix B did not indicate the presence of any deep seated loose soils (shear wave velocity less than 200 m/s) within the glacial till. Therefore, the glacial till is not considered to be liquefiable during a seismic event. The loose N-values may be attributed to possible disturbance of the soil during drilling and sampling operations.

7.2 Seismic Site Classification

The MASW survey indicates the average shear wave velocity to a 30 m depth is 599.2 m/s, which is within the range of average soil shear wave velocity for a seismic classification of Class C in accordance with Table 4.1.8.4A of the 2012 Ontario Building Code (OBC).

The site class beneath the existing footing at Elevation 71.6 m was also calculated and determined to be Class B. However, the OBC does not permit the use of Class B if there is more than 3 m of soil between the bedrock and underside of the footing, which is the case for this project. Therefore, **Site Class C** governs for seismic site response.

For reference, the 2015 National Building Code Seismic Hazard Calculation for the site is shown in Appendix C.

8 Foundation Considerations

8.1 Discussion

The subsurface conditions encountered in the exterior and interior boreholes are similar; fill underlain by stiff clay followed by loose glacial till.

Based on a review of the 1981 Architectural Drawing Nos. A2 and A6, the basement slab elevation of the north wing is at Elevation 72.55 m rising to Elevation 73.01 m at the central building of the hotel. Based on the interior borehole located in the basement of the north wing of the hotel, the founding elevation of the footing is at 71.6 m.

The following foundation alternatives are considered feasible to accommodate the increased loading from the additional two (2) storeys proposed for the north wing of the hotel:

- Utilize the existing footings to support the increased loading, provided the soils supporting the existing footings have the capacity to carry the increased loading. The size of the existing footings may need to be increased to reduce the load stress imposed on the supporting soils.
- Underpin the existing footings using either helical piers or micro-piles.

Each foundation alternative is discussed in the following sections of this report.

8.2 Available Capacity of Founding Soil of Existing Footings

Strip footings having a maximum width of 3 m and square footings having a maximum size of 5 m by 5 m and founded on the native stiff to very stiff brown and grey clay or loose to very dense glacial till may be designed for a bearing pressure at serviceability limit state (SLS) of 96 kPa and a factored geotechnical resistance at ultimate limit state (ULS) of 150 kPa. The factored ULS value includes a resistance factor of 0.5 in accordance with the 2006 Canadian Foundation Engineering Manual (CFEM).

Settlements of the footings designed for the SLS value above and properly constructed are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential movements.

The existing footings may be increased to the maximum sizes noted above to accommodate the increased loading of the additional two (2) storeys while maintaining the SLS bearing pressure at 96 kPa.

The above SLS value is valid for the given footing sizes and founding soil type and assumes there will be no grade raise at the site. If the assumption of zero grade raise is not valid, EXP should be contacted to provide updated SLS and ULS values for footings.

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

8.3 Underpinning Existing Foundations using Micro Piles

If insufficient capacity is available in the existing footings and founding soil to support the additional building storeys, additional foundation capacity may be achieved by underpinning the existing footings using micro piles that extend into the bedrock. A micro pile is a small diameter (typically less than 300 mm) drilled and grouted non-displacement pile that is typically reinforced. The load is transferred by bond from the thread bar to the grout and from the grout to the surrounding ground. Two types of micro piles being considered are:

- uncased micro piles.
- cased micro piles.

8.3.1 Uncased Micro Piles

The following expression may be used to compute the ultimate load per pile for uncased micro piles. From a geotechnical perspective, these piles will carry the load in bond between the overburden (soil) and the grout and between the bedrock and the grout. The following expression assumes that the piles will be pressure grouted through the casing during casing withdrawal. Since the piles will be socketed into bedrock, the ULS will govern the design.

$$P_{ult} = \pi\,(\alpha_1\, l_1\, d_1 + \alpha_2\, l_2\, d_2 + \alpha_3\, l_3\, d_2)$$

Where

Where	P_{ult}	=	Ultimate load carrying capacity of pile kN based on the sum of bond between the grout and the overburden (soil) and between the grout and the bedrock.
	α_1	=	130 kPa ultimate bond between overburden (soil) and grout from underside of footing founded at assumed Elevation 72.2 m to Elevation 71.1 m
	l_1	=	Length of pile from Elevation 72.2 m to Elevation 71.1 m
	d_1	=	Diameter of drilled hole in overburden (m)
	α_2	=	200 kPa ultimate bond between loose to very dense glacial till and the grout
	l_2	=	Length of pile in glacial till from Elevation 71.1 m to top of bedrock and inferred bedrock at Elevation 65.4 m to 59.9 m
	d_2	=	Diameter of drilled hole in bedrock
	α_3	=	1500 kPa ultimate bond between bedrock and grout
	l_3	=	Length of pile socketed in bedrock, m

The computed ultimate capacity of the pile should be multiplied by a geotechnical resistance factor of 0.4 when computing the factored ULS axial load carrying capacity of the pile and by a geotechnical resistance factor of 0.3 when computing the factored ULS uplift capacity of the pile.

8.3.2 Cased Micro Piles

It is considered that an alternative to uncased micro piles is to case a micro pile into the overburden (soil) and upper level of the bedrock and construct the remainder of the micro piles by drilling uncased holes in the bedrock. Such a pile will carry the load in bond between the grout and the bedrock. The load carrying capacity of a cased micro pile with the casing embedded in the upper level of the bedrock may be computed from the following expression:

$$P_{ult} = \pi \, \alpha_1 \, l_1 \, d_1$$

Where P_{ult} = Ultimate load carrying capacity of pile, kN.

α_1 = 1500 kPa ultimate bond between bedrock and grout

l_1 = Length of pile socketed in bedrock, m

d_1 = Diameter of drilled hole in bedrock, m

In this case, the bond between the casing and the soil should be neglected.

The computed ultimate capacity of the piles should be multiplied by a geotechnical resistance factor of 0.4 when computing the factored ULS axial capacity of the piles, and by a geotechnical resistance factor of 0.3 when computing the factored ULS uplift capacity of the piles.

8.3.3 Recommendations Applicable to Uncased and Cased Micro Piles

It is noted that the pile borings should be cased in the overburden to prevent cave-in of the granular soils and to reduce the groundwater seepage into the pile holes. It is imperative that the holes for installation of the piles are cleaned properly so that the grout is in contact with overburden and clean bedrock free of any soil smearing, etc. All water should be pumped out from the pile borings prior to the placement of the grout. It is noted from the subsurface investigation that the glacial till contains numerous boulders and cobbles which the installation contractor should take into consideration when selecting the method of drilling of the micro-piles. Also, water inflow into the drilled micro piles holes should be expected.

It is recommended that the pile capacity should be proven by performing pile load tests in accordance with the requirements of ATSM D1143/D1143M-07. If the piles will also be subjected to tension and lateral loads, the capability of the piles to support these loads should also be proven by performing appropriate pile load tests in accordance with the procedures specified in ASTM.

8.4 Underpinning Existing Foundations Using Helical Piers

Alternatively, the existing footings may be underpinned using helical piers that extend into the glacial till or bedrock. Helical piers are a proprietary product and should be designed and installed by a recognized specialist contractor in the province of Ontario.

As noted above, the contractor should be made aware that the glacial till contains cobbles and boulders and should consider the most appropriate method for the installation of the helical piers.

8.5 Additional Comment

Portions of the existing basement floor slab may require removal to accommodate the above foundation alternatives. It is assumed the basement floor has been designed as a slab-on-grade. The floor slab should be reinstated to match existing conditions.

9 Subsurface Walls

The subsurface basement walls should be backfilled with free draining material, such as OPSS 1010 Granular B Type II and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the walls. The walls will be subjected to lateral static and dynamic (seismic) earth forces. The expressions below assume the subsurface basement walls are non-yielding, will have free draining backfill material, a perimeter drainage system, level backfill surface behind the wall and vertical face on the back side of the wall.

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

$$P = K_0 h (½ \gamma h + q)$$

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

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

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

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

q = surcharge load, kPa

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

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

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

H = height of wall, m

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

$\frac{a_h}{g}$ = seismic coefficient = 0.278 for 2 percent probability of exceedance in 50 years (refer to 2015 National Building Code Seismic Hazard Calculation shown in Appendix C)

F_b = thrust factor = 1.0

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

All subsurface basement walls should be properly damp-proofed.

10 Excavations and De-Watering Requirements

10.1 Excavations

The soils at the site may be excavated with conventional mechanical equipment capable of removing cobbles and boulders for excavations extending into the glacial till.

It should be noted that the surface of the clay is susceptible to disturbance due to movement of workers and construction equipment especially if the excavations are undertaken during wet weather periods. It is therefore considered that depending on the weather conditions prevailing at the time of construction, footing beds may have to be covered with a mud slab to prevent disturbance to the clay subgrade.

The excavations at the site may be undertaken as open-cut provided they meet the requirements of the Ontario Occupational Health and Safety Act. The soils are classified as Type 3 and must be cut back at 1H:1V from the bottom of the excavation. For excavations below the groundwater level, the side slopes should be flattened to a gradient ranging from 2H:1V to 3H:1V from the bottom of the excavation. If space restrictions prevent open-cut excavation, the excavations will have to be undertaken within the confines of an engineered support system designed and constructed in accordance with the above regulation.

Caution should be exercised during excavation operations so as not to extend excavations below the base of existing footings resulting in the undermining of the footings. If excavations are required to extend below the base of the existing footings, underpinning of the existing footings will be required.

It is recommended a preconstruction survey of the existing north wing of the hotel be undertaken prior to start of the construction. A settlement monitoring program of the north wing, consisting of periodic monitoring of strategically placed settlement monitoring points should be undertaken prior to, during and following construction to assess the behavior of the north wing of the hotel.

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.

10.2 De-watering Requirements

Seepage of surface and subsurface water into the excavations is anticipated. However, it should be possible to collect any water entering the excavations in perimeter ditches and to remove it by pumping from sumps.

It is noteworthy to mention that new legislation came into force in Ontario on March 29, 2016 to regulate groundwater takings for construction de-watering 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 de-watering, 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 de-watering 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 de-watering 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 field work and commented on de-watering and general construction problems, conditions may be present that are difficult to establish from standard boring and test pit excavating techniques and which may affect the type and nature of de-watering 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 de-watering systems.

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

The suitability of the existing backfill soils along the exterior side of the basement walls for reuse as backfill soil will have to be assessed during construction of the proposed vertical extension.

The subsurface soils encountered in the boreholes consist of silty sand and gravel fill and clay. From a geotechnical perspective, these soils are not considered suitable for re-use as backfill material against the existing basement walls of the hotel building.

Typically, the backfill material against basement walls should consist of free draining granular material preferably conforming to the Ontario Provincial Standard Specification (OPSS) for Granular B Type II placed in 300 mm thick layers which each layer compacted to 95 percent standard Proctor maximum dry density (SPMDD). Caution should be exercised when compacting close to the existing basement walls, so as not to damage the walls. Also, basement walls are typically equipped with a perimeter drainage system that is suitably outletted.

Portions of the existing weeping tile, assumed to be present around the exterior perimeter of the north wing of the hotel, may require removal to facilitate the new work and should be reinstated to match existing conditions. Inspection of the existing weeping tile may also be required to ensure it is operating properly. Depending on the inspection findings, sections of the weeping tile may require removal and replacement.

12 Corrosion Potential of Subsurface Soils

Chemical tests limited to pH, sulphate, chloride and resistivity tests were performed on two (2) selected soil samples. The results are shown on Table No. VIII. The Certificate of Analysis is included in Appendix D.

Table VIII: Results of pH, Sulphate, Chloride and Resistivity Tests on Soil Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Soil	pH	Sulphate (%)	Chloride Content (%)	Resistivity (ohm.cm)
			<5	>0.1 %	>0.04 %	
BH 1 – SS5	3.0 – 3.5 (71.9 – 71.4)	Grey Clay	7.58	0.0161	0.1540	457
BH 1 – SS9	7.6 – 8.2 (67.3 – 66.7)	Glacial Till	7.98	0.0093	0.0278	1,830

The results indicate a soil with a sulphate content of less than 0.1 percent. This concentration of sulphate in the soil would have a negligible potential of sulphate attack on subsurface concrete. The concrete should be designed in accordance with CSA A.23.1-14. However, the concrete should be dense, well compacted and cured. The elevated chloride content of 0.1540 percent in Borehole No.1 SS5 (3.0 m to 3.5 m depths) should be taken into consideration for the design of the concrete mix.

The results of the resistivity tests indicate that the soil is severely corrosive to buried steel elements. Appropriate measures should be undertaken to protect buried steel elements from corrosion.

13 Pavement Structure Reinstatement

The reinstated pavement structure should consist of asphaltic concrete comprising of 40 mm SP 12.5 (PG 58-34) Level B/HL3 and 50 mm SP19.0 (PG 58-34) Level B/HL8 underlain by 150 mm OPSS Granular A base layer which is further underlain by 450 mm OPSS Granular B Type II sub-base layer. The asphaltic concrete should be compacted from 92 to 97 percent of the maximum relative density. The Granular A and B (Type II) should be compacted to 100 percent SPMDD.

Proper drainage of the pavement structure must be provided to ensure satisfactory performance. If subdrains are encountered, they should be reinstated.

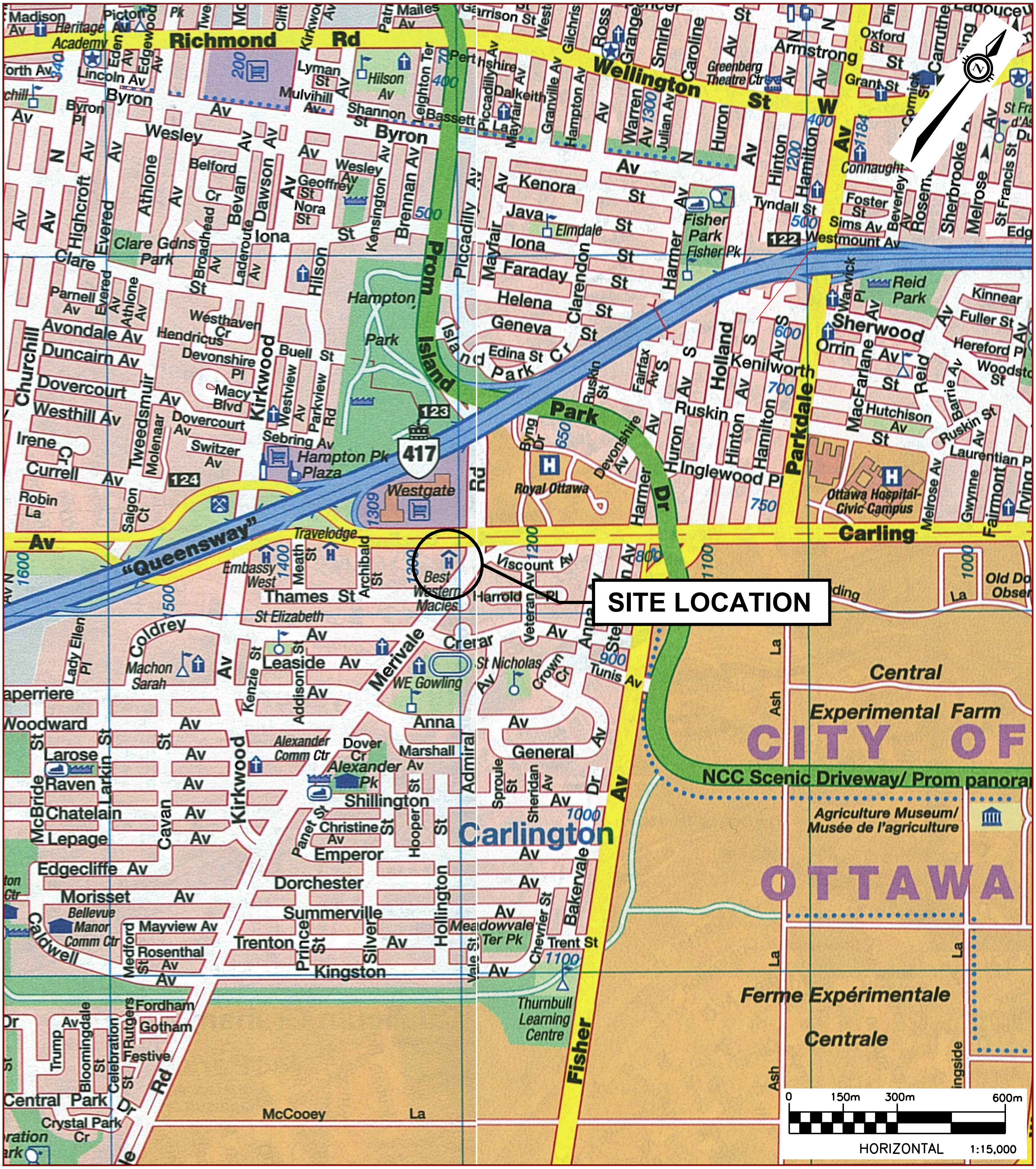
14 General Comments

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. Should specific information be required, including for example, the presence of pollutants, contaminants or other hazards in the soil, additional testing may be required.

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



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scale AS SHOWN	PROJECT: BEST WESTERN PLUS PROPOSED VERTICAL EXTENSON OF BEST WESTERN PLUS 1274 CARLING AVENUE, OTTAWA, ON	project no. OTT-00245894-A0	
date MAY 2018		design	checked S.P.
drawn by J.R.	TITLE: SITE LOCATION PLAN	FIG 1	

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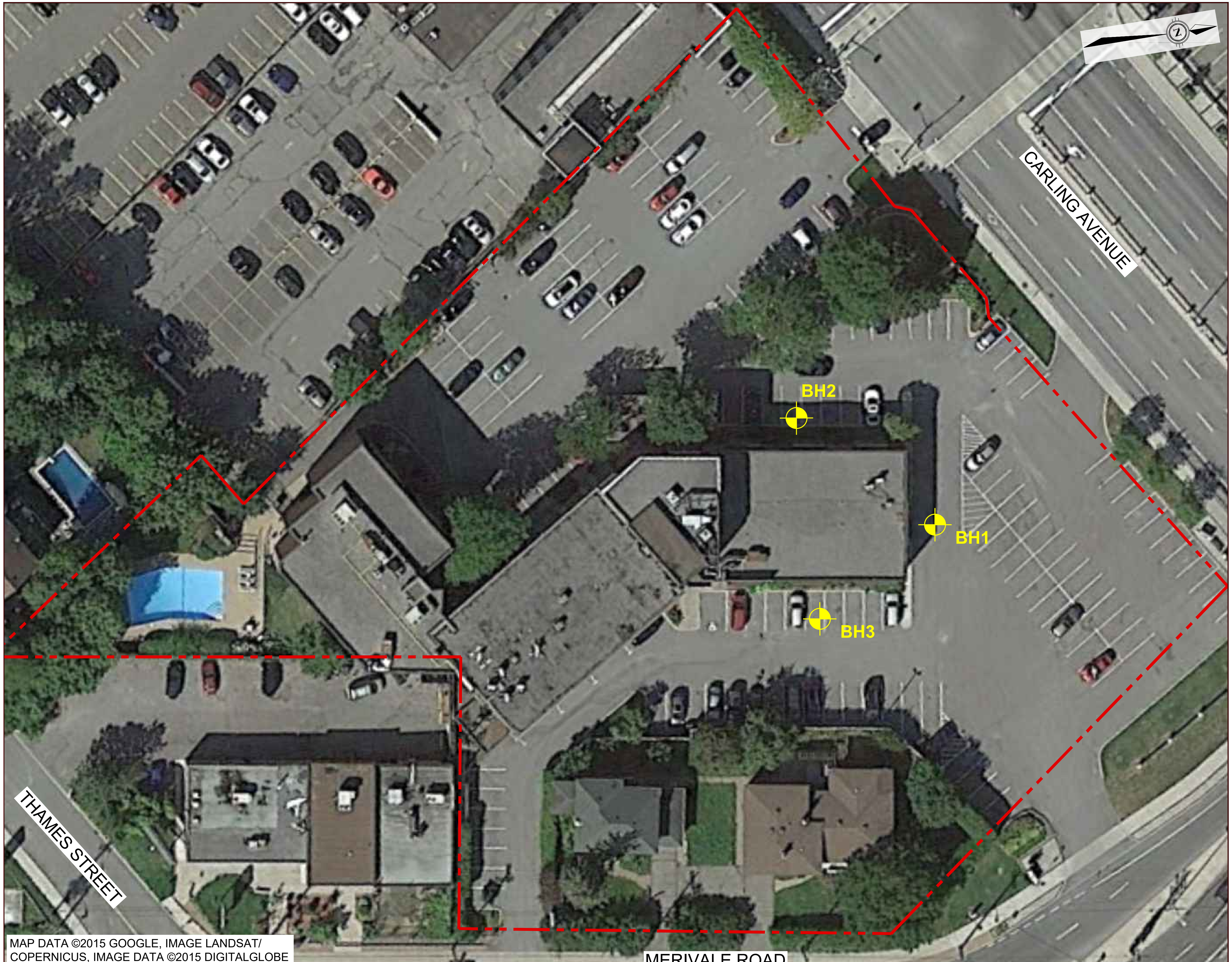

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NOTES :

1. THE BOUNDARIES AND SOIL/ ROCK TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND ROCK WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. TOPSOIL QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. MAP DATA ©2015 GOOGLE, IMAGE DATA ©2015 DIGITALGLOBE

LEGEND:

BH1 — BOREHOLE LOCATION AND NUMBER

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

CLIENT
BEST WESTERN PLUS

PROJECT
PROPOSED VERTICAL EXPANSION OF BEST WESTERN PLUS
1274 CARLING AVENUE, OTTAWA, ON

TITLE
LOCATION PLAN
OF EXTERIOR BOREHOLES

date	AUGUST 2018	project no.	OTT-00245894-A0
design by	S.P.	drawing no.	FIG 2
prepared by	J.R.		
reviewed by	I.T.		

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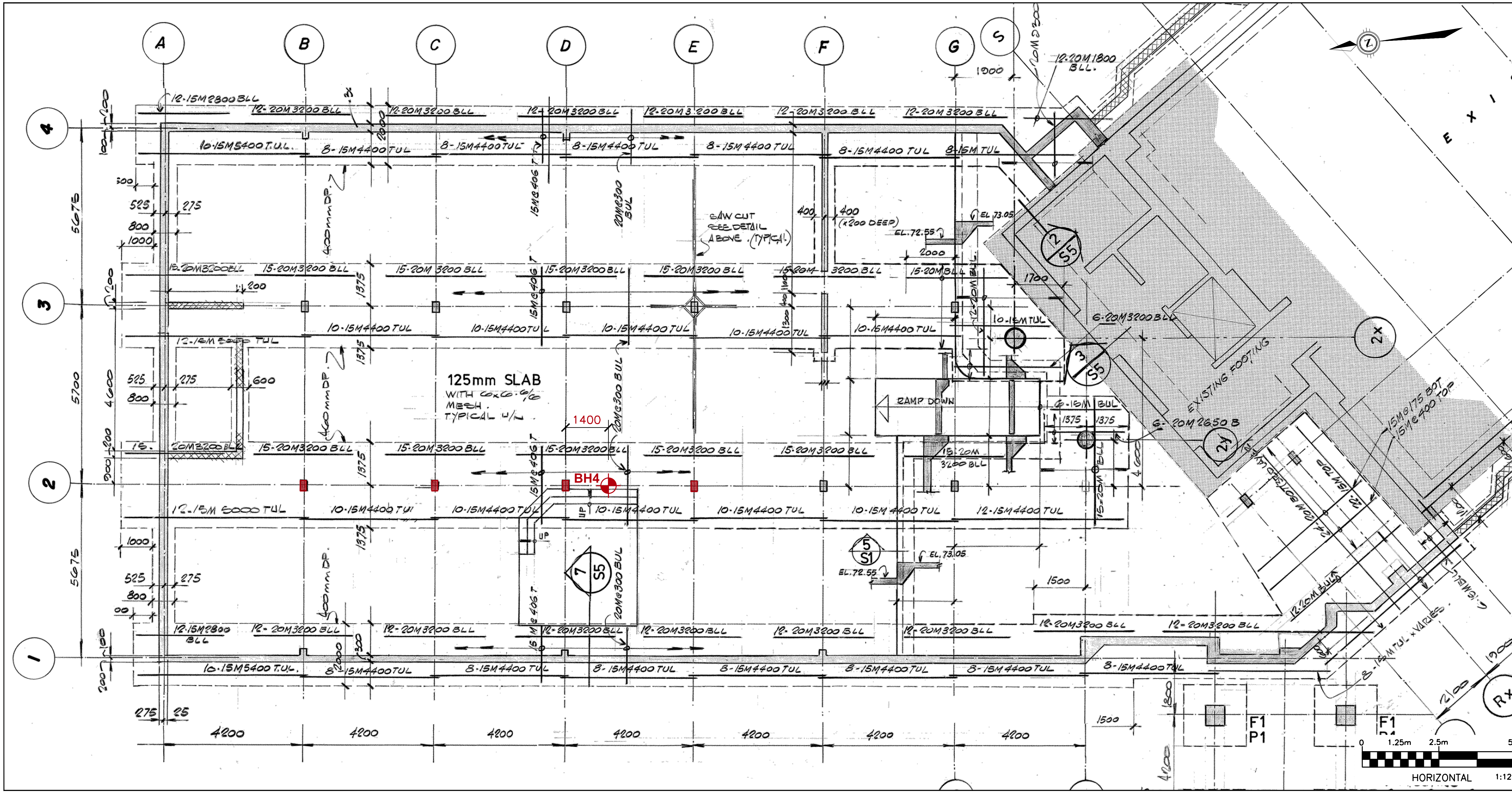

NOTES :
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5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. BASE DRAWING FROM OLIVER, MANGIONE, McCALLA AND ASSOCIATES. DRAWING S1, REV 2, FOUNDATION PLAN, FOR PROJECT 81-2777 MACIES OTTAWAN MOTEL, DATED JAN, 1982.

LEGEND:

DIMENSIONS IN MILLIMETERS

BH1 BOREHOLE LOCATION AND NUMBER

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scale 1:125	PROJECT: PROPOSED VERTICAL EXPANSION OF BEST WESTERN PLUS 1274 CARLING AVENUE, OTTAWA, ON		project no. OTT-00245889-A0
date AUGUST 2018			
drawn by J.R.	TITLE: LOCATION PLAN OF INTERIOR BOREHOLE		FIG 3

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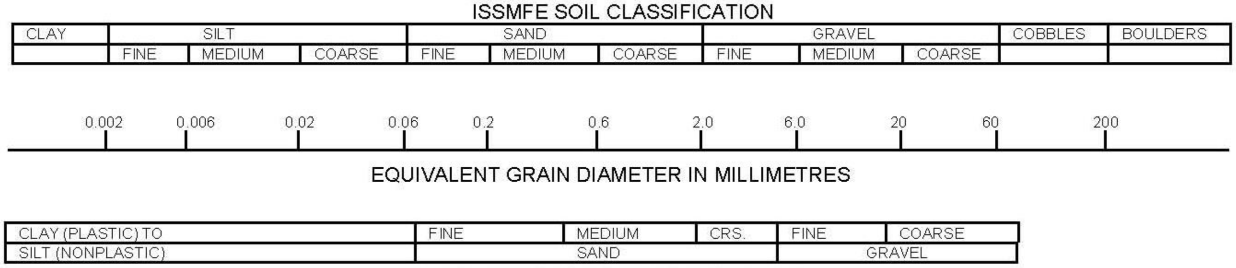
Notes On Sample Descriptions

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

ISSMFE SOIL CLASSIFICATION



UNIFIED SOIL CLASSIFICATION

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

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

Log of Borehole BH 1

Project No: OTT-00245894-A0

Project: Geotechnical Investigation - Proposed Vertical Extension of Best Western Plus

Location: 1274 Carling Avenue, Otawa, Ontario

Figure No. 4

Page. 1 of 2

Date Drilled: 'May 3, 2018

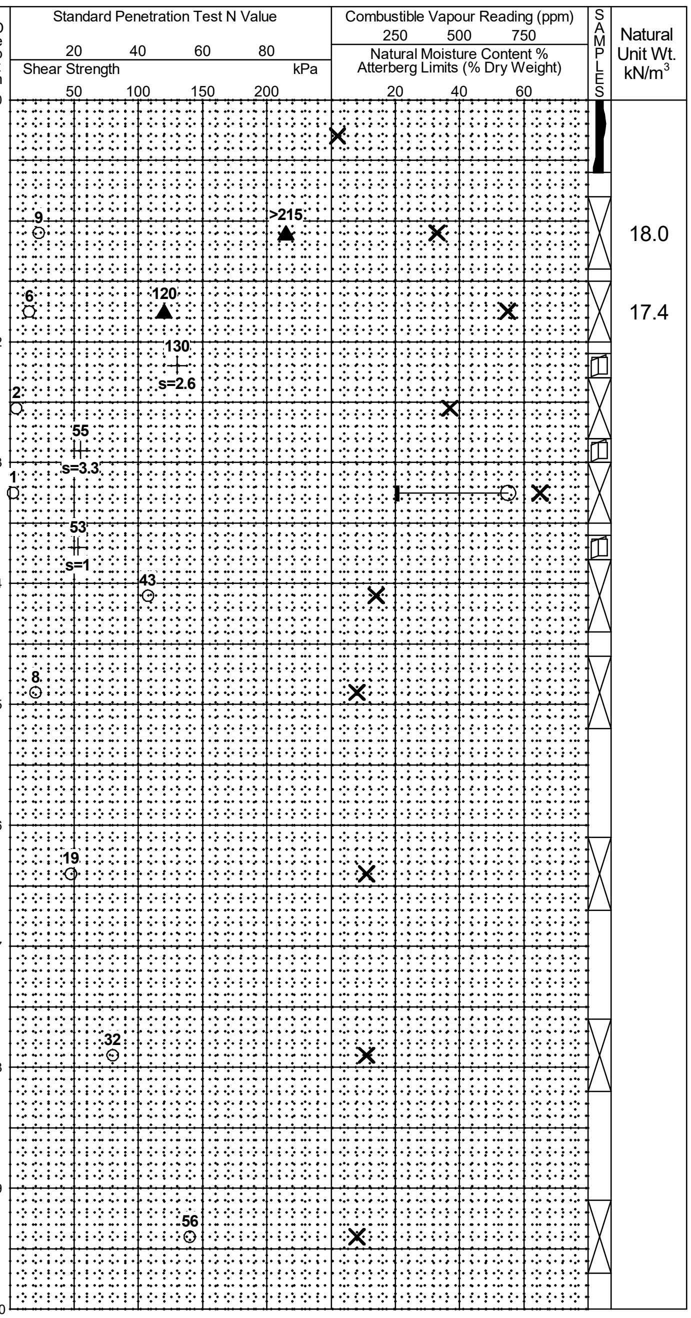
Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: A.N. Checked by: S.P.

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

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			74.93	0
		ASPHALTIC CONCRETE ~90 mm	74.8	
		FILL Silty sand and gravel, brown, damp	74.2	
		CLAY Medium sensitivity, brown, damp, (very stiff to hard)	72.5	1–2
▼		**CLAY** Shell fragments, low to medium sensitivity, high plasticity, grey, wet, (stiff)	72.33	
			71.1	3
		GLACIAL TILL Silty sand with gravel to sand with silt and gravel, cobbles, boulders, grey, wet, (loose to very dense) difficult augering from 3.8 m to 6.1 m depths rootlets at 6.1 m depth sand seam at 6.7 m depth		4–10



Continued Next Page

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-00245894-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	2.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	15 - 16.2	100	11
2	16.2 - 17.7	100	68
3	17.7 - 19.2	100	65

Log of Borehole BH 1

Project No: OTT-00245894-A0

Project: Geotechnical Investigation - Proposed Vertical Extension of Best Western Plus

Figure No. 4

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	Standard Penetration Test N Value (20 40 60 80) / Shear Strength kPa (50 100 150 200)	Combustible Vapour Reading (ppm) (250 500 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20 40 60)	SAMPLES	Natural Unit Wt. kN/m³
		GLACIAL TILL Silty sand with gravel to sand with silt and gravel, cobbles, boulders, grey, wet, (loose to very dense) difficult augering from 3.8 m to 6.1 m depths rootlets at 6.1 m depth sand seam at 6.7 m depth *(continued)*	64.93	10				
				12	79			
		LIMESTONE BEDROCK Horizontal fractures, grey, (very poor to good quality) mud seams from 16.0 m to 16.2 m depths	59.9	15				RunNo.1
				17				RunNo.2
				18				RunNo.3
		Borehole Terminated at 19.2 m Depth	55.7	19				

NOTES:

1. Borehole data requires interpretation by EXP before use by others
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5. Log to be read with EXP Report OTT-00245894-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	2.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	15 - 16.2	100	11
2	16.2 - 17.7	100	68
3	17.7 - 19.2	100	65

Log of Borehole BH 2

Project No: OTT-00245894-A0

Project: Geotechnical Investigation - Proposed Vertical Extension of Best Western Plus

Location: 1274 Carling Avenue, Otawa, Ontario

Figure No. 5

Date Drilled: 'May 4, 2018

Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: A.N. Checked by: S.P.

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

GWL	Symbol	Soil Description	Geodetic Elevation (m)	Depth	Standard Penetration Test N Value (20 40 60 80) / Shear Strength kPa (50 100 150 200)	Combustible Vapour Reading (ppm) (250 500 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20 40 60)	Samples	Natural Unit Wt. kN/m³
			75.02	0				
		ASPHALTIC CONCRETE ~75 mm	74.9			×		
		FILL Silty sand and gravel, brown, damp, (very loose)		1	2	×		
				2	3	×		
		CLAY Sensitive, brown, moist, (very stiff)	72.6		2; 106; s=6.3	×		18.5
▼ 72.32				3				
		CLAY Sensitive, grey, wet	71.8					
		GLACIAL TILL Silty sand with gravel, cobbles, boulders, grey, wet, (loose to very dense) difficult augering from 4.3 m to 6.6 m depths	71.2	4	56	×		
				5	6	×		
				6	17	×		
				8	7	×		
				9	8	×		
			65.2	10				

Continued Next Page

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 19 m diameter standpipe peizometer installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245894-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	4.0	
12 days	2.7	

CORE DRILLING RECORD

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

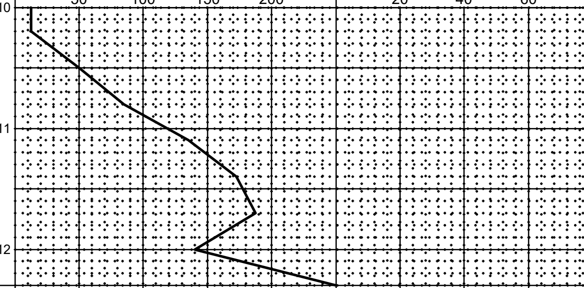
Log of Borehole BH 2

Project No: OTT-00245894-A0

Project: Geotechnical Investigation - Proposed Vertical Extension of Best Western Plus

Figure No. 5

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	Standard Penetration Test N Value 20 40 60 80 / Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 / Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m³
		Dynamic Cone Penetration Test (DCPT) from 9.8 m to Cone Refusal at 12.3 m depths *(continued)*	65.02	10				
				11				
			62.7	12				
		Cone Refusal at 12.3 m Depth						



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 m diameter standpipe peizometer installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245894-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	4.0	
12 days	2.7	

CORE DRILLING RECORD

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

Log of Borehole BH 3

Project No: OTT-00245894-A0

Project: Geotechnical Investigation - Proposed Vertical Extension of Best Western Plus

Location: 1274 Carling Avenue, Otawa, Ontario

Figure No. 6

Page. 1 of 1

Date Drilled: 'May 3, 2018

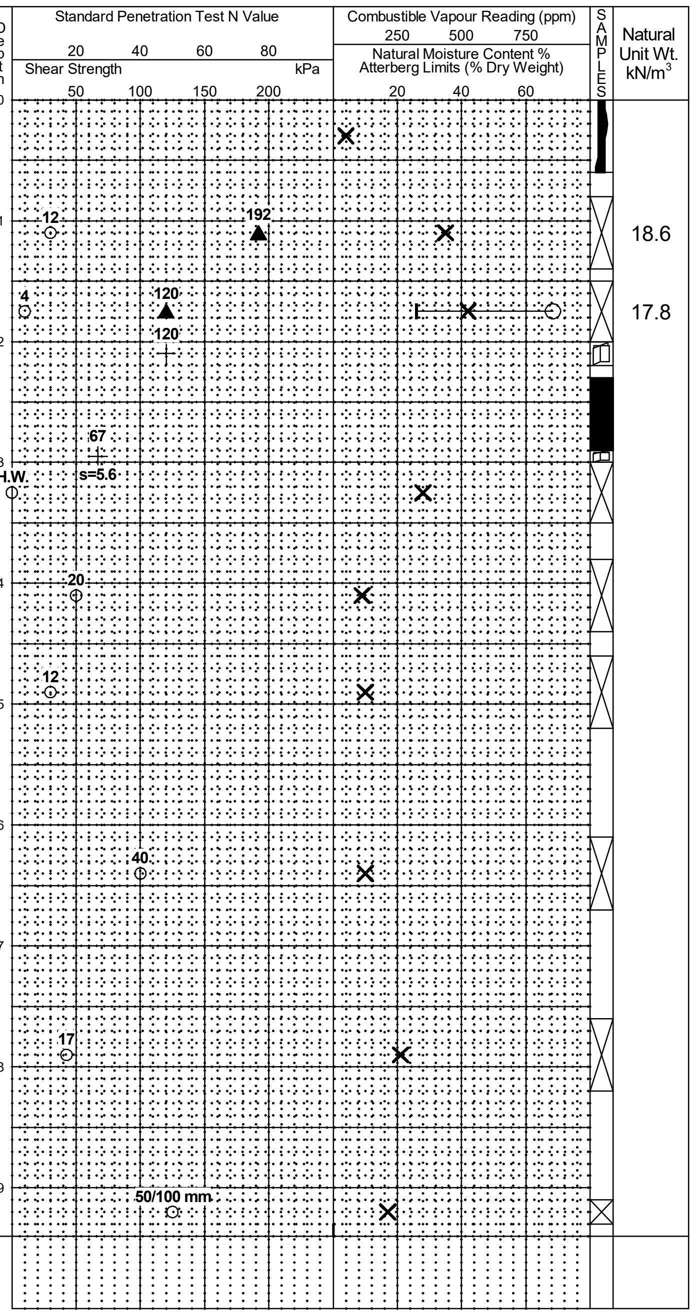
Drill Type: CME-75 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: A.N. Checked by: S.P.

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

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			74.83	0
		ASPHALTIC CONCRETE ~ 75 mm	74.7	
		FILL Silty sand and gravel, brown, damp		
			73.9	
		CLAY High plasticity, brown, damp, (very stiff)		1
			72.5	2
▼			72.33	
		CLAY Sensitive, grey, wet, (stiff)		
			71.5	3
		GLACIAL TILL Silty sand with gravel, cobbles, boulders, grey, wet, (compact to dense) difficult augering from 3.8 m to 9.3 m depths		4–9
			65.5	
		Dynamic Cone Penetration Test (DCPT) from 9.3 m to Cone Refusal at 9.4 m Depth	65.4	
		Cone Refusal at 9.4 m Depth		



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 m diameter standpipe peizometer installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245894-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	6.1	
13 days	2.5	

CORE DRILLING RECORD

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

Log of Borehole BH 4

Project No: OTT-00245894-A0

Project: Geotechnical Investigation - Proposed Vertical Extension of Best Western Plus

Location: 1274 Carling Avenue, Otawa, Ontario

Date Drilled: 'July 25, 2018

Drill Type: Manual Drilling Equipment

Datum: Geodetic Elevation

Logged by: M.L. Checked by: S.P.

Figure No. 7

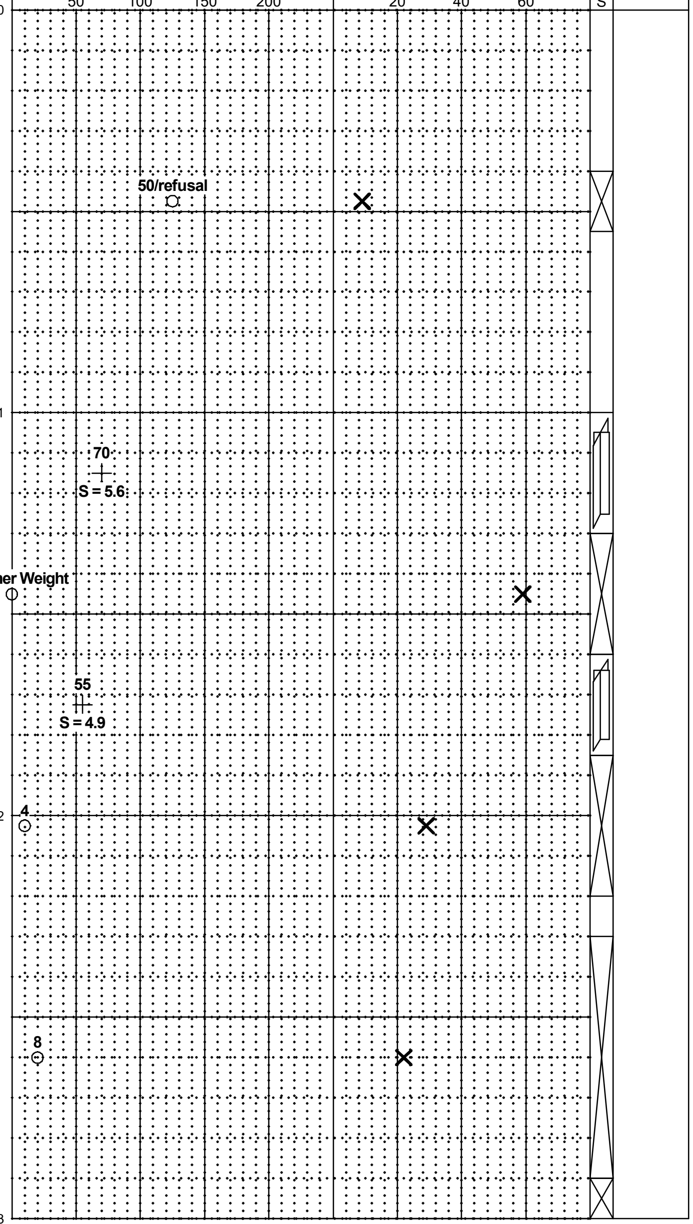
Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◧	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

Geodetic Elevation (m)	Depth (m)	SOIL DESCRIPTION
72.55	0	
72.5		**FLOOR TILE** ~6 mm
72.4		**CONCRETE BASEMENT FLOOR SLAB** ~110 mm Reinforced with wire mesh located 40 mm from top of slab
72.2		**GRANULAR FILL** ~205 mm of crusher run limestone
		FILL Sand, some gravel, asphalt pieces, brown, moist
72.0		**CONCRETE FOOTING** ~450 mm Reinforced with 15 M size steel bars located 130 mm and 305 mm from the top of the footing
71.6	1	**CLAY** Sensitive, grey, wet, (stiff)
70.7		**CLAY** Trace to some sand and gravel, grey, wet, (firm)
70.5 70.45	2	**GLACIAL TILL** Silty sand with gravel to sand with silt and gravel, cobbles, boulders, grey, wet, (loose)
	3	

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



Continued Next Page

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-00245894-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	2.1	

CORE DRILLING RECORD

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

Log of Borehole BH 4

Project No: OTT-00245894-A0

Project: Geotechnical Investigation - Proposed Vertical Extension of Best Western Plus

Figure No. 7

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	Standard Penetration Test N Value (20 40 60 80) / Shear Strength kPa (50 100 150 200)	Combustible Vapour Reading (ppm) (250 500 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20 40 60)	SAMPLES	Natural Unit Wt. kN/m³
		GLACIAL TILL Silty sand with gravel to sand with silt and gravel, cobbles, boulders, grey, wet, (loose) *(continued)*	69.55	3	7			23.2
					7			
				4				
			68.5					
		Borehole Terminated at 4.1 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-00245894-A0

WATER LEVEL RECORDS

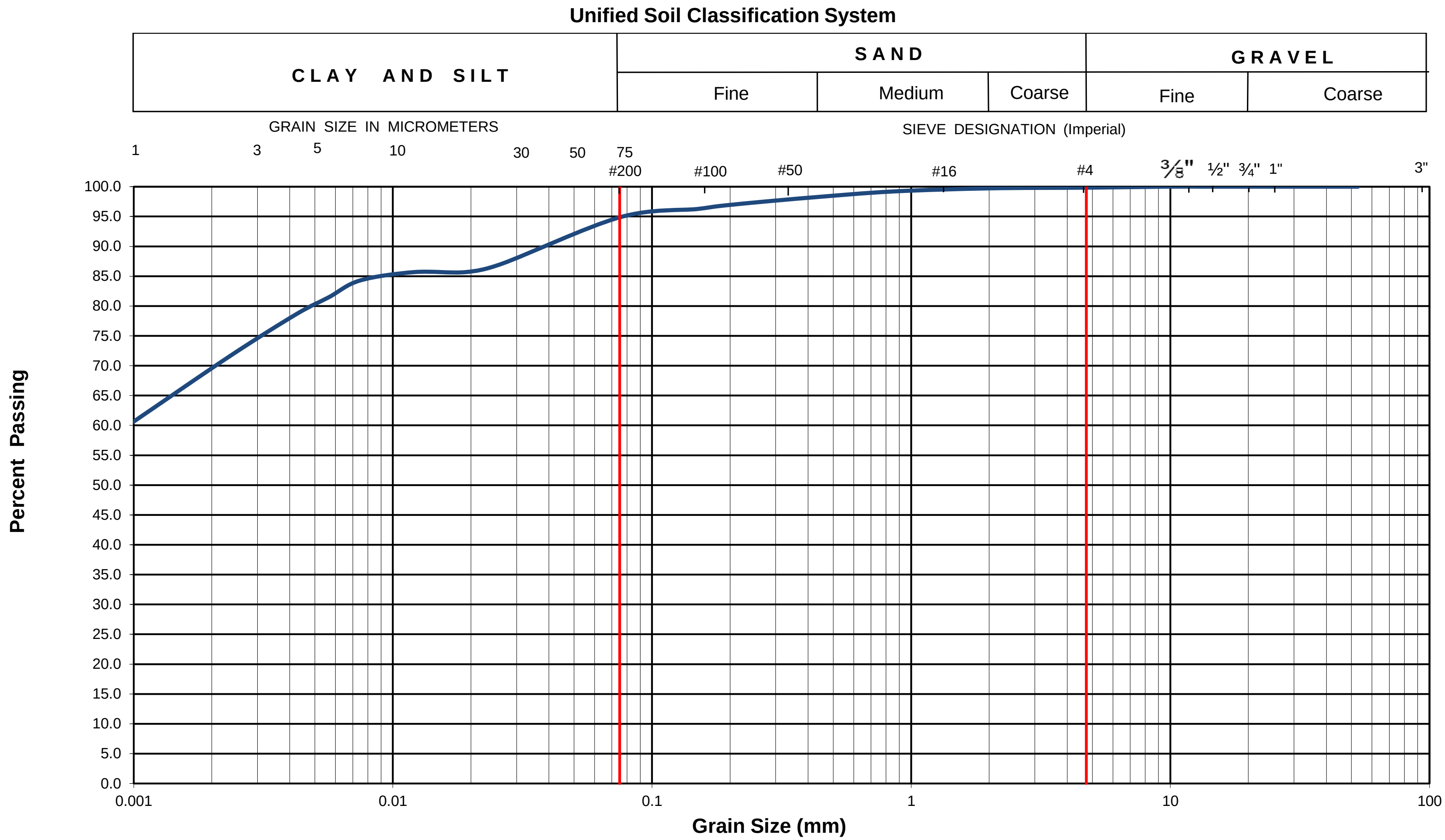
Elapsed Time	Water Level (m)	Hole Open To (m)
On Completion	2.1	

CORE DRILLING RECORD

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

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

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



EXP Project No.:	**OTT-00245894-A0**	**Project Name :**	**Geotechnical Investigation - Vertical Extension of Best Western Plus**				
Client :	**Best Western Plus**	**Project Location :**	**1274 Carling Avenue, Ottawa, ON.**				
Date Sampled :	**May 4, 2018**	**Borehole No:**	**BH3**	**Sample No.:**	**SS3**	**Depth (m) :**	**1.5-2.0**
Sample Description :		**% Silt and Clay**	**95**	**% Sand**	**5**	**% Gravel**	**0**
Sample Description :	**Clay (CH)**					**Figure :**	**8**

EXP Services Inc.
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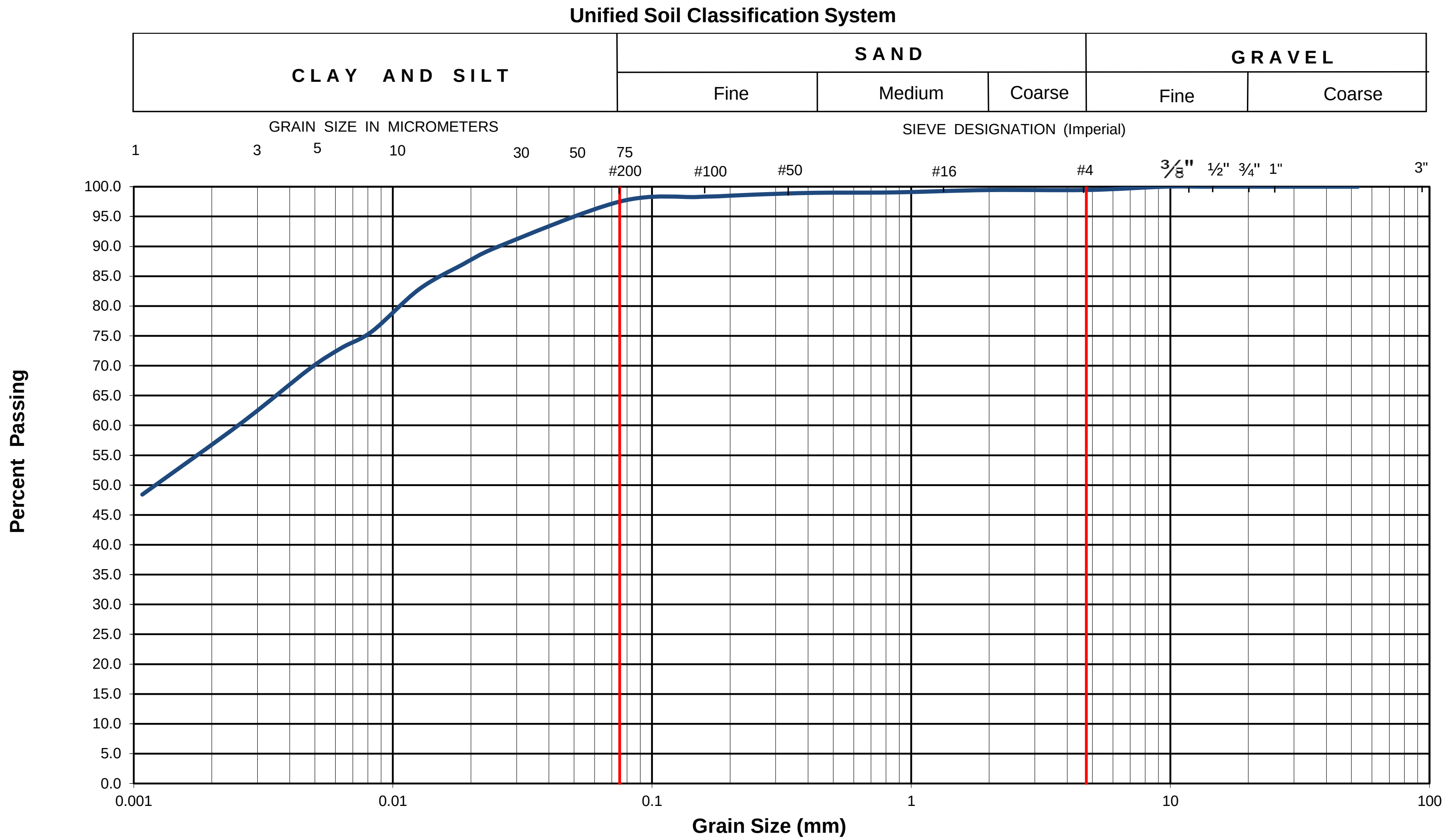
Ottawa, ON K2B 8H6

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



EXP Project No.:	OTT-00245894-A0	Project Name :	Geotechnical Investigation - Vertical Extension of Best Western Plus					
Client :	Best Western Plus	Project Location :	1274 Carling Avenue, Ottawa, ON.					
Date Sampled :	May 4, 2018	Borehole No:	BH1	Sample No.:	SS5		Depth (m) :	3.0-3.5
Sample Description :		% Silt and Clay	97	% Sand	2	% Gravel 1	Figure :	9
Sample Description :		Clay (CH)						

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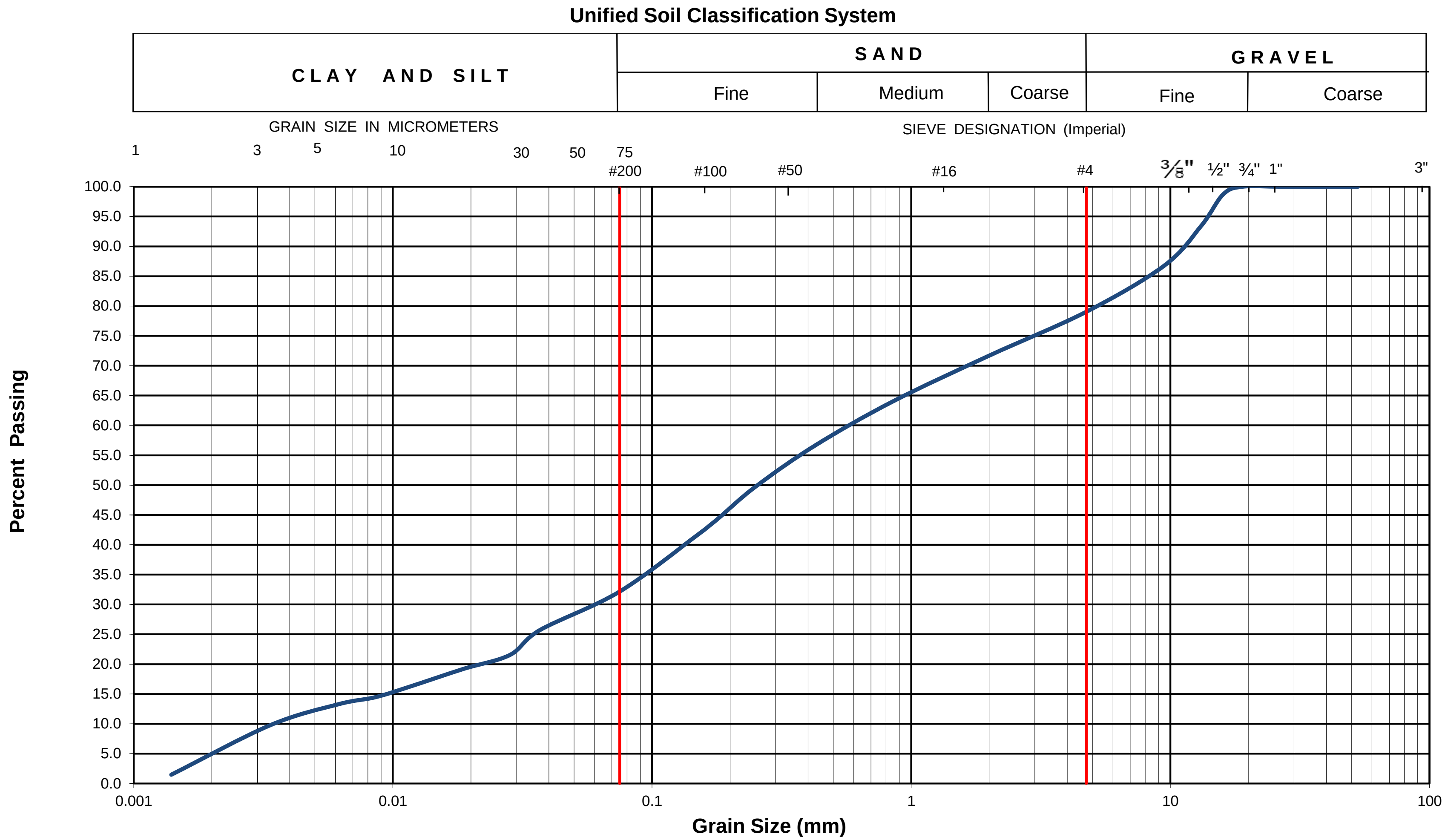
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-00245894-A0	Project Name :	Geotechnical Investigation - Vertical Extension of Best Western Plus					
Client :	Best Western Plus	Project Location :	1274 Carling Avenue, Ottawa, ON.					
Date Sampled :	May 4, 2018	Borehole No:	BH1	Sample No.:	SS7		Depth (m) :	4.6-5.2
Sample Description :		% Silt and Clay	32	% Sand	47	% Gravel 21	Figure :	10
Sample Description :		Glacial Till - Silty Sand with Gravel (SM)						

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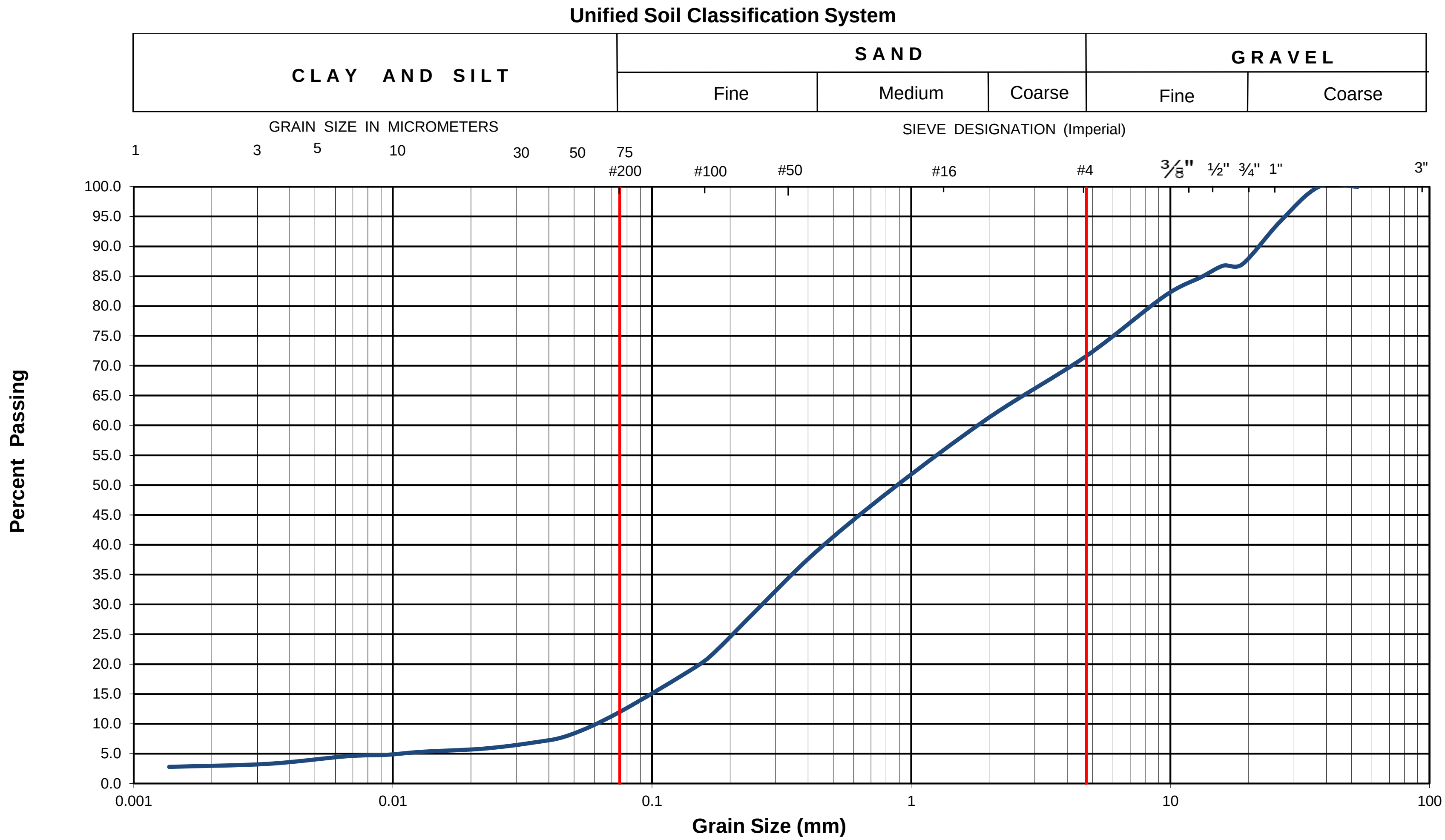
Ottawa, ON K2B 8H6

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



EXP Project No.:	**OTT-00245894-A0**	**Project Name :**	**Geotechnical Investigation - Vertical Extension of Best Western Plus**						
Client :	**Best Western Plus**	**Project Location :**	**1274 Carling Avenue, Ottawa, ON.**						
Date Sampled :	**May 4, 2018**	**Borehole No:**		**BH1**	**Sample No.:**	**SS10**		**Depth (m) :**	**9.1-9.7**
Sample Description :		**% Silt and Clay**	**12**	**% Sand**		**60**	**% Gravel**	**28**	**Figure : 11**
Sample Description :	**Glacial Till - Sand with Silt and Gravel (SW-SM)**								

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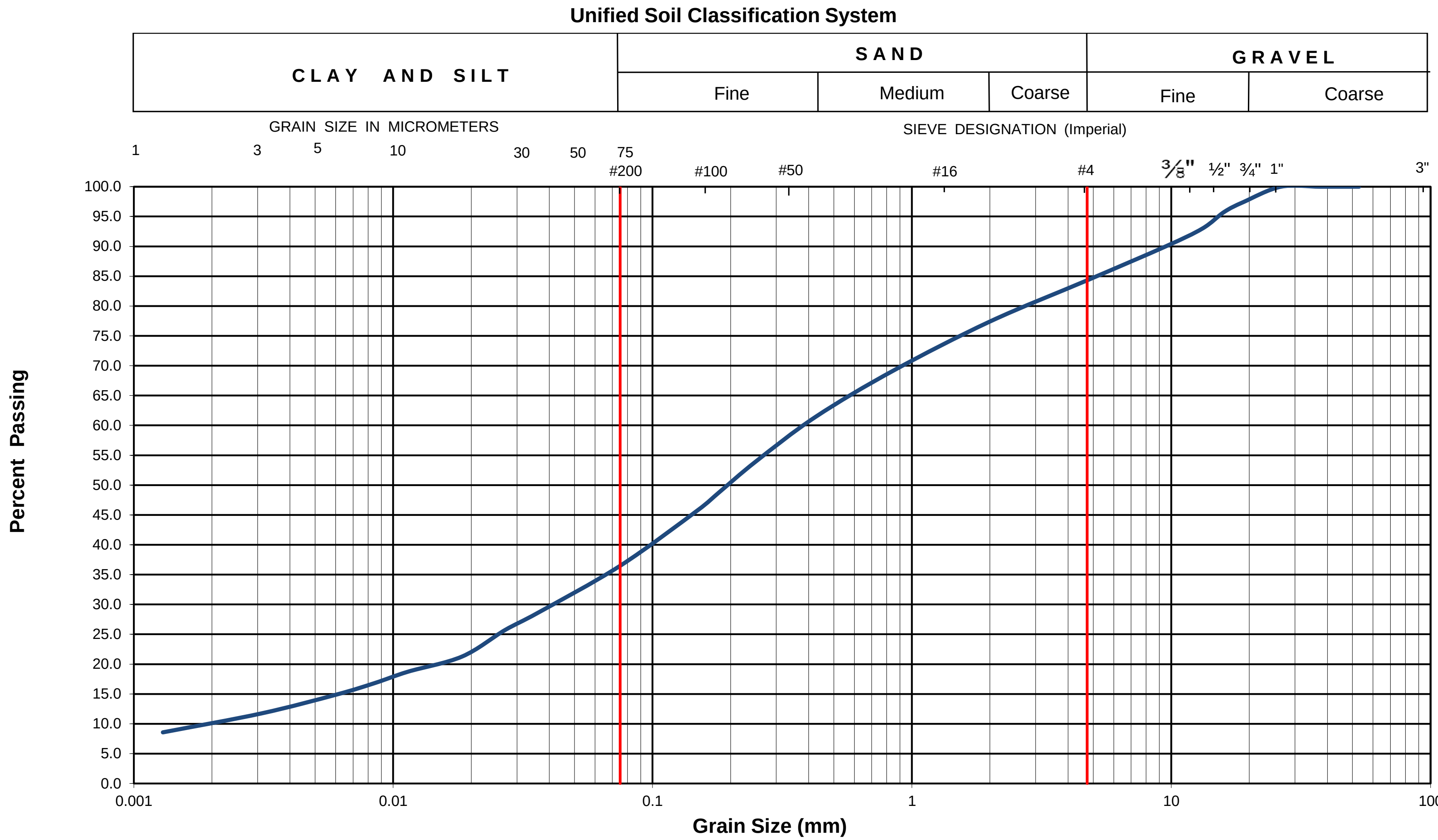
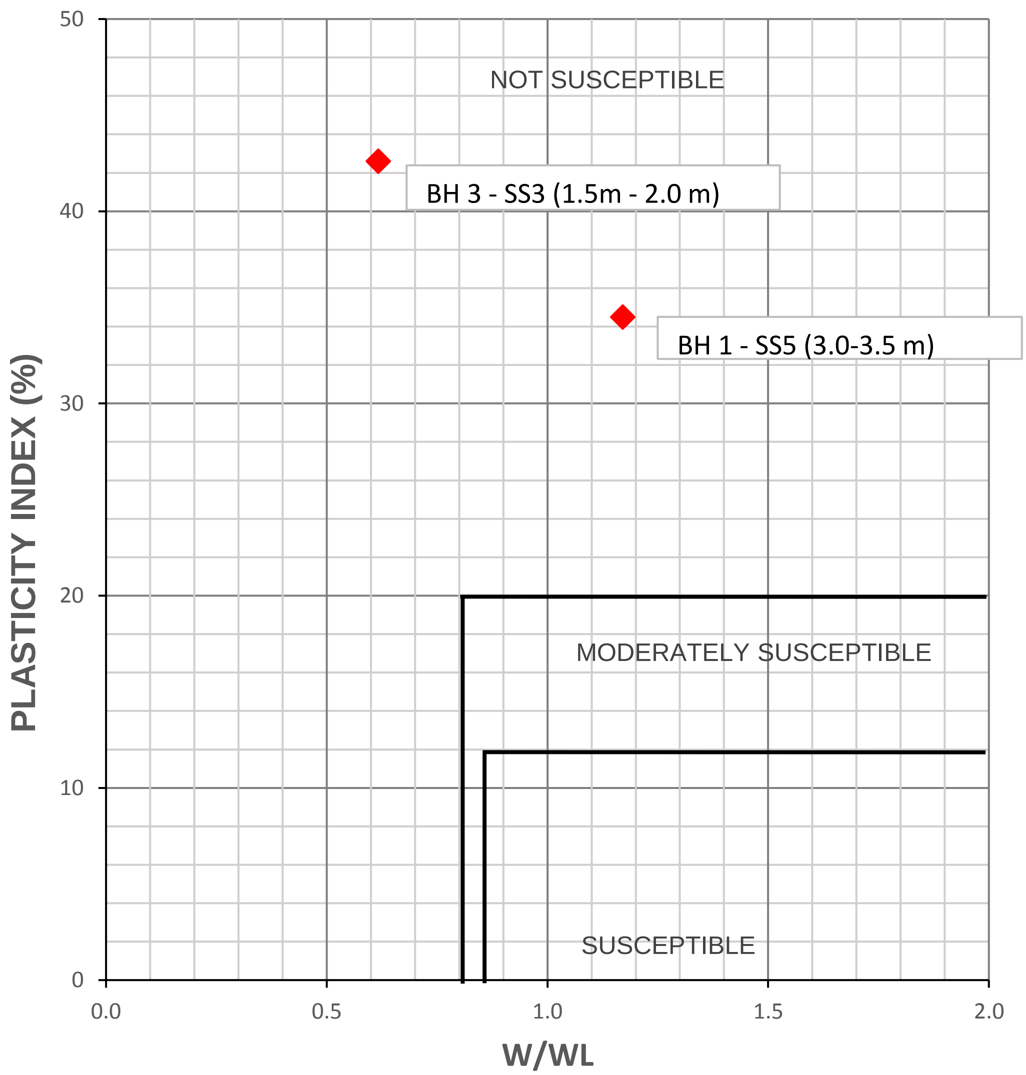
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245894-A0**	**Project Name :**	**Geotechnical Investigation - Vertical Extension of Best Western Plus**					
Client :	**Best Western Plus**	**Project Location :**	**1274 Carling Avenue, Ottawa, ON.**					
Date Sampled :	**May 4, 2018**	**Borehole No:**	**BH2**	**Sample No.:**	**SS9**		**Depth (m) :**	**7.6-8.2**
Sample Description :		**% Silt and Clay**	**37**	**% Sand**	**47**	**% Gravel 16**	**Figure :**	**12**
Sample Description :	**Glacial Till - Silty Sand with Gravel (SM)**							



BRAY ET AL. (2004) CRITERIA FOR LIQUEFACTION ASSESSMENT OF FINE GRAINED SOILS

exp Services Inc.
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2650 Queensview Drive, Suite 100
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Canada
www.exp.com

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• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

scale: N.T.S.	CLIENT: Best Western Plus	project no. OTT-00245894-A0
date: May 2018	TITLE: Geotechnical Investigation - Proposed Vertical Extension of Best Western Plus 1274 Carling Avenue, Ottawa, ON.	FIG. 13
drawn by: M.L.		

Appendix A: Rock Core Photographs

DRY BEDROCK CORES

WET BEDROCK CORES

exp Services Inc.
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borehole no.	core runs		project no.
BH-1	Run 1: 15.0m-16.2m Run 2: 16.2m-17.7m Run 3: 17.7m-19.2m	Geotechnical Investigation Proposed Vertical Extension of Best Western Plus	OTT-00245894-A0
date cored May 03, 2018		Rock Core Photographs	FIG. A-1

Appendix B: MASW Survey Results

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7
Tel. : (450) 679-2400
Fax : (514) 521-4128
info@geophysicsgpr.com
www.geophysicsgpr.com

June 11th, 2018

Transmitted by email: susan.potyondy@exp.com
Our Ref.: GPR-18-00642-c

Mrs. Susan Potyondy, P.Eng.
Senior Project Manager
exp Services inc.
100 - 2650 Queensview Drive
Ottawa (ON) K2B 8H6

Subject: **Shear Wave Velocity Sounding for Site Class Determination**
1274, Carling Avenue, Ottawa (ON)

[Project: OTT-00245894-A0]

Dear Madam,

Geophysics GPR International inc. has been requested by **exp** Services inc. to carry out seismic shear wave surveys at the Best Western Plus, located at 1274 Carling Avenue, 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 for the soil and the rock.

The surveys were carried out, on May 23rd, by Mr. Alexis Marchand and Mrs. Jasmine-Sophie Papineau, trainee. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the 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 "noises" produced far away. The method can also be used with "active" seismic source records. The dispersion properties are expressed as a change of phase velocities with frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding). Figure 3 schematically outlines the basic operating procedure for the MASW method.

Figure 4 illustrates an example of one of the MASW/ESPAC records, the corresponding spectrogram analysis and resulting 1D V_S model. The ESPAC method allows deeper Vs soundings, but generally with a lower resolution for the surface portion. Its dispersion curve can then be merged with the higher frequency one from the MASW to calculate a more complete inversion.

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 considered seismic wave's arrival times were identified for each geophone. 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 measurements were realised to calculate the rock depth, and its seismic velocity (using P waves). The rock seismic velocities (V_S) were calculated using two methods: the reduced travel-times (the Hobson and Overton method) and the opposite apparent velocities. The first one allows independence from the surface and rock topography effect, as well as the overburden lateral variation of its seismic velocity, but remains limited to common geophones. Its application remains however limited to shallow to intermediate depths refractors. The second one can use longer segments of opposite directions signals, improving the linear regressions accuracy, but remains affected by the surface and rock topography effect, as well as the overburden lateral variation of the seismic velocity. Conversely to the MASW method, the seismic rock velocity calculated by seismic refraction is only representative of its superior part, due to the evanescent nature of the refracted wave.

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 the northern parking, starting from Merivale Road. The geophone spacing for the main spread was of 3 metres, using 24 geophones. Shorter seismic spreads, with geophone spacings of 0.5 and 1 metre, were dedicated to the near surface 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

From seismic refraction surveys, the rock was calculated between 7.5 and 10 to 12 metres deep (± 1 metre), dipping West. Its seismic velocity was calculated between 1725 and 1750 m/s for the upper portion (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.

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.

Considering an average rock depth of 11 metres, the calculated $\overline{V}_{S30}$ value of the actual site is 599.2 m/s (cf. Table 1), corresponding to the Site Class “C”. A $\overline{V}_{S30}$ of 633.8 m/s could be calculated considering an average rock depth of 9 meters, leading to the same Site Class. Nevertheless, low seismic velocities were calculated from the surface to approximately 2 metres deep.

As the existing footings of the existing wing of the building are located between 2.3 to 3.0 metres deep, a $\overline{V}_{S30}{}^*$ value was calculated considering an average depth of 2.65 metres. This calculated value is 838.7 m/s, corresponding to Site Class “B”. However, the Site Classes A and B are not to be used if there is more than 3 metres of unconsolidated material between the rock and the lower portion of the footings. In the case of an average rock depth of 9 meters, the $\overline{V}_{S30}{}^*$ value would be 890.8 m/s, corresponding also to the Site Class “B”, but with the same NBC constraint.

CONCLUSION

Geophysical surveys were carried out at the Best Western Plus, located at 1274 Carling Avenue, in Ottawa (ON). The seismic surveys used the MASW, ESPAC analysis methods, as well as the complementary seismic refraction method, to calculate the $\overline{V}_{S30}$ value for the Site Class determination. The $\overline{V}_{S30}$ calculation is presented in Table 1.

The calculated $\overline{V}_{S30}$ value of the actual site is 599 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 footings of the existing wing of the building being known between 2.3 to 3.0 metres deep, a $\overline{V}_{S30}$* value was calculated considering an average depth of 2.65 metres. The corresponding calculated value is 839 m/s, corresponding to the Site Class "B". However, the Site Classes A and B are not to be used if there is more than 3 metres of unconsolidated material between the rock surface and the bottom of the spread footing or mat foundation. This restrictive condition was encountered from the geotechnical boreholes, as well as from the geophysical seismic refraction results.

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.

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

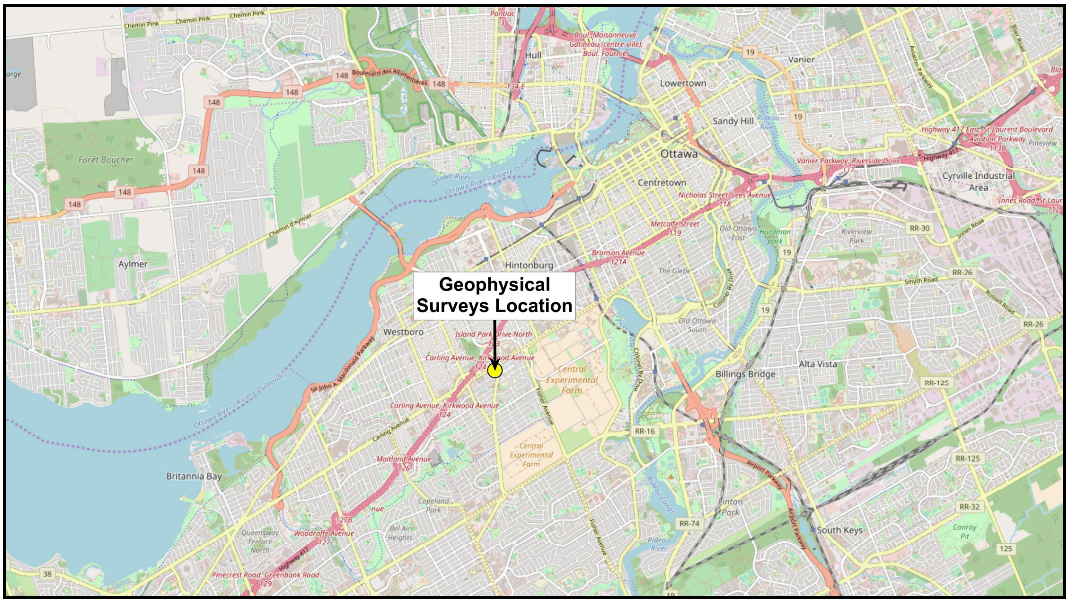


Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

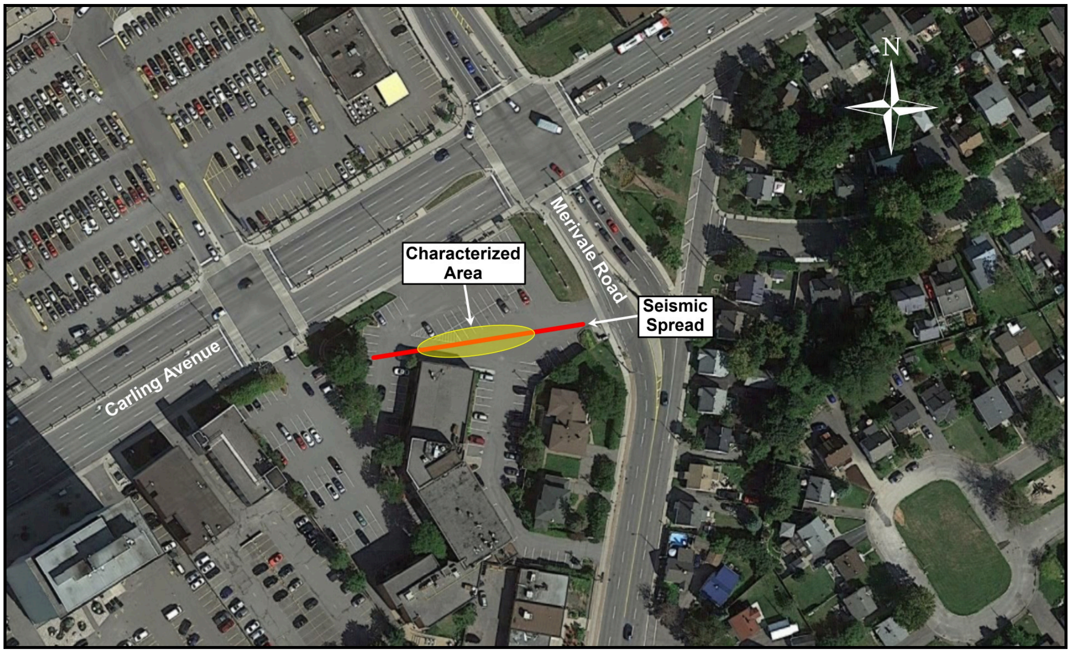


Figure 2: Location of the seismic spread
(source: *Google Earth™*)

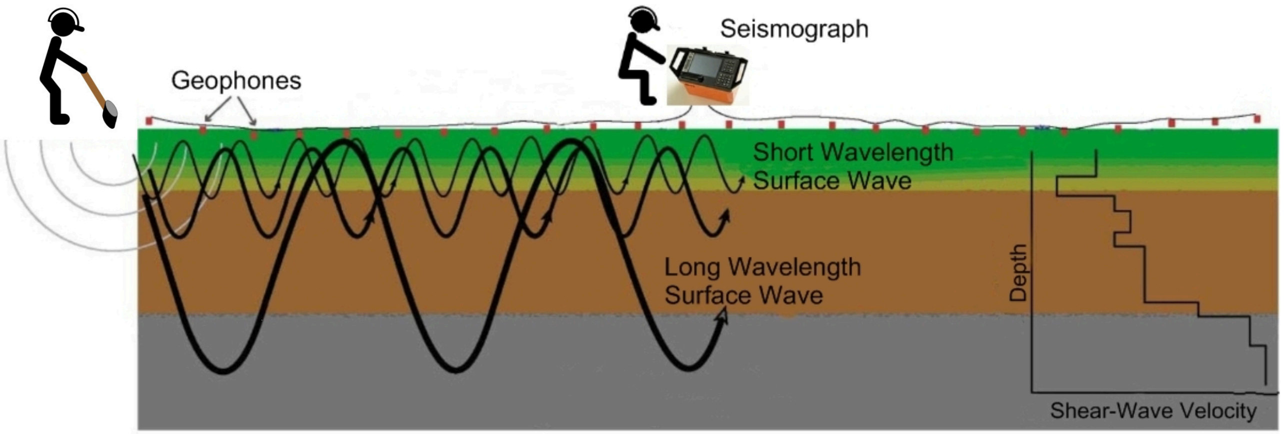
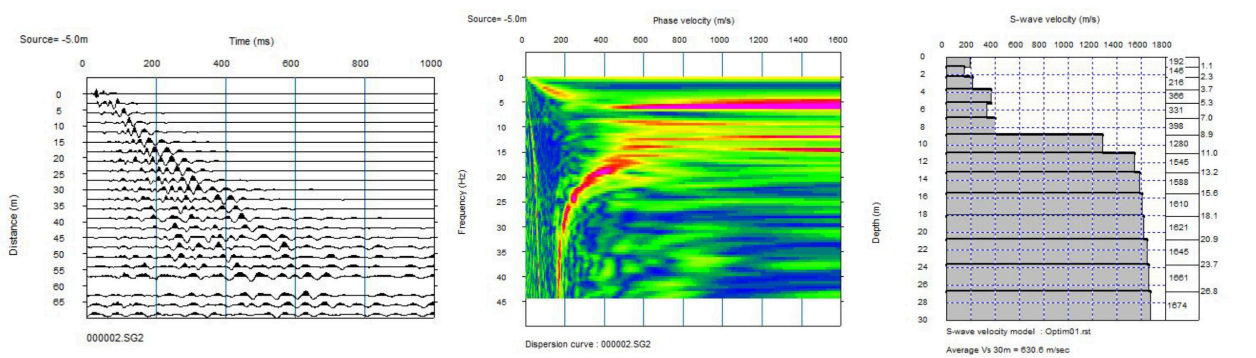


Figure 3: MASW Operating Principle



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

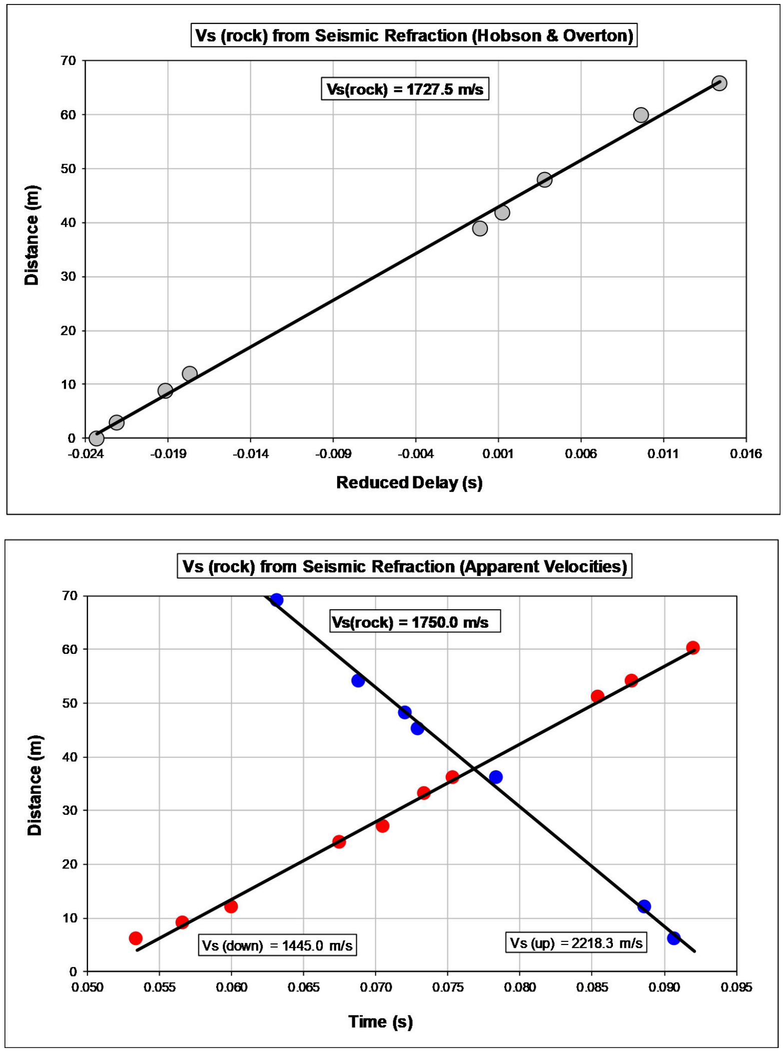


Figure 5: Rock V_S from Seismic Refraction

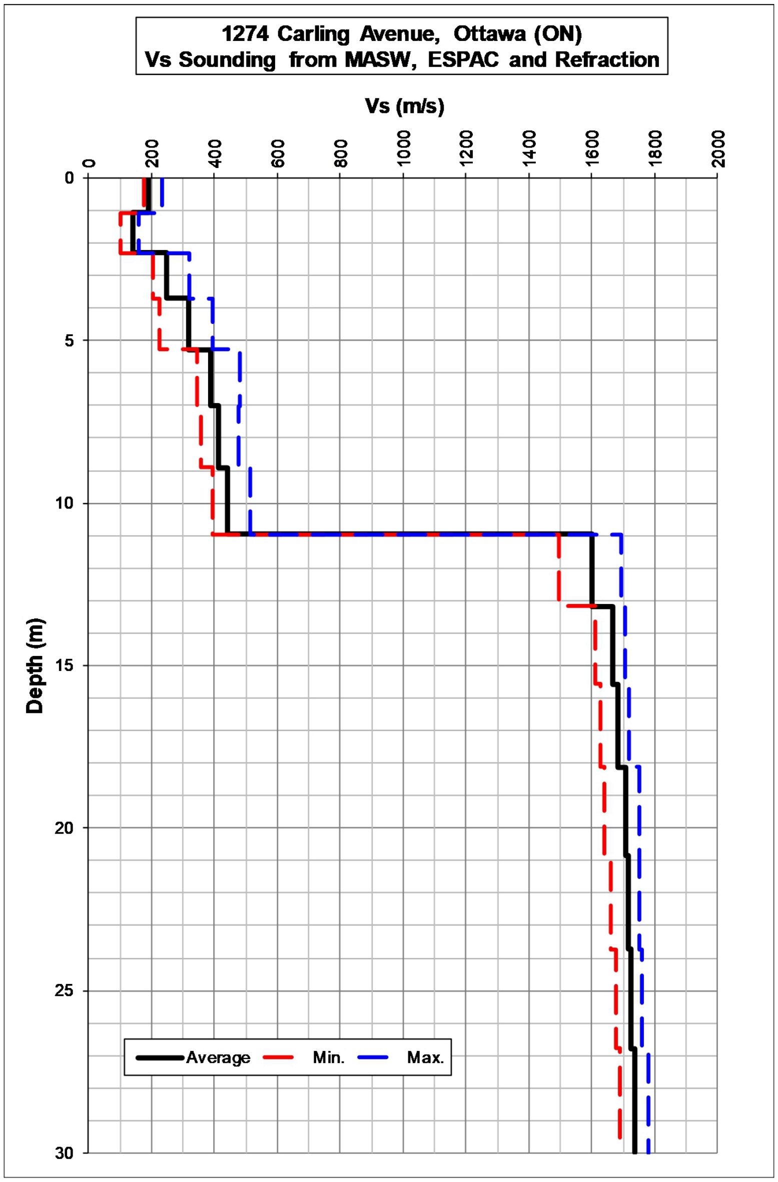


Figure 6: MASW Shear-Wave Velocities Sounding

TABLE 1

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

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	173.4	190.8	232.3					
1.07	99.2	140.2	158.7	1.07	1.07	0.005616	0.005616	190.8
2.31	203.0	245.6	317.9	1.24	2.31	0.008816	0.014432	159.9
3.71	224.9	316.2	394.2	1.40	3.71	0.005705	0.020136	184.2
5.27	345.4	387.9	478.7	1.57	5.27	0.004952	0.025088	210.2
7.01	355.2	410.9	474.8	1.73	7.01	0.004462	0.029551	237.1
8.90	394.3	441.9	515.4	1.90	8.90	0.004614	0.034164	260.5
10.96	1493.9	1600.3	1694.8	2.06	10.96	0.004662	0.038827	282.3
13.19	1610.0	1667.3	1705.3	2.23	13.19	0.001390	0.040217	327.9
15.58	1628.8	1681.2	1717.6	2.39	15.58	0.001433	0.041651	374.0
18.13	1638.4	1705.6	1750.0	2.55	18.13	0.001520	0.043171	420.0
20.85	1661.3	1716.2	1750.0	2.72	20.85	0.001595	0.044765	465.8
23.74	1675.9	1724.8	1757.7	2.88	23.74	0.001681	0.046446	511.0
26.79	1689.4	1734.4	1779.5	3.05	26.79	0.001768	0.048214	555.6
30				3.21	30.00	0.001853	0.050067	599.2

V_{S30} (m/s)	**599.2**
Class	**C**

TABLE 2

V_{S30}* Calculation for the Site Class (actual footings at 2.65 metres deep)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	173.4	190.8	232.3	**Considering footings at 2.65 metres deep**				
1.07	99.2	140.2	158.7					
2.31	203.0	245.6	317.9					
2.65	203.0	245.6	317.9					
3.71	224.9	316.2	394.2	1.06	1.06	0.004311	0.004311	245.6
5.27	345.4	387.9	478.7	1.57	2.62	0.004952	0.009263	283.4
7.01	355.2	410.9	474.8	1.73	4.36	0.004462	0.013725	317.3
8.90	394.3	441.9	515.4	1.90	6.25	0.004614	0.018339	340.9
10.96	1493.9	1600.3	1694.8	2.06	8.31	0.004662	0.023001	361.3
13.19	1610.0	1667.3	1705.3	2.23	10.54	0.001390	0.024392	432.0
15.58	1628.8	1681.2	1717.6	2.39	12.93	0.001433	0.025825	500.6
18.13	1638.4	1705.6	1750.0	2.55	15.48	0.001520	0.027345	566.2
20.85	1661.3	1716.2	1750.0	2.72	18.20	0.001595	0.028940	628.9
23.74	1675.9	1724.8	1757.7	2.88	21.09	0.001681	0.030621	688.6
26.79	1689.4	1734.4	1779.5	3.05	24.14	0.001768	0.032389	745.2
32.65				5.86	30.00	0.003381	0.035770	838.7

V_{S30}* (m/s)	**838.7**
Class	**B [(1)]**

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

Appendix C: 2015 National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

May 23, 2018

Site: 45.386 N, 75.733 W User File Reference: 1274 Carling Avenue Ottawa ON.

Requested by: , EXP Services Inc.

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.442	0.518	**0.434**	0.330	**0.234**	**0.117**	**0.056**	**0.015**	**0.0054**	**0.278**	**0.194**

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. ***These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.043	0.145	0.243
Sa(0.1)	0.060	0.183	0.295
Sa(0.2)	0.054	0.158	0.251
Sa(0.3)	0.043	0.122	0.192
Sa(0.5)	0.031	0.087	0.137
Sa(1.0)	0.015	0.044	0.069
Sa(2.0)	0.0060	0.020	0.032
Sa(5.0)	0.0012	0.0047	0.0080
Sa(10.0)	0.0006	0.0019	0.0032
PGA	0.032	0.100	0.161
PGV	0.021	0.067	0.109

References

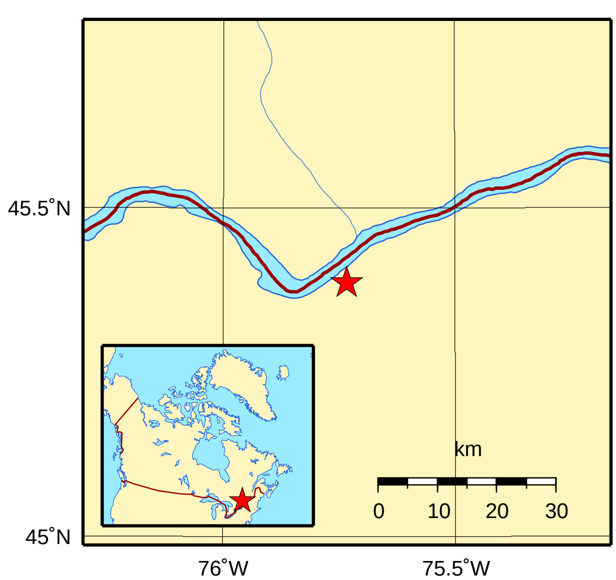
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français



Natural Resources Canada Ressources naturelles Canada

Canada

Appendix D: AGAT 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-245894-AO**

AGAT WORK ORDER: **18Z338119**

SOIL ANALYSIS REVIEWED BY: **Amanjot Bhela, Inorganic Coordinator**

DATE REPORTED: **May 18, 2018**

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.

AGAT Laboratories

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

Certificate of Analysis

AGAT WORK ORDER: 18Z338119

PROJECT: OTT-245894-AO

CLIENT NAME: EXP SERVICES INC

SAMPLING SITE:1274 Carling Ave.

ATTENTION TO: Susan Potyondy

SAMPLED BY:exp

Inorganic Chemistry (Soil)

DATE RECEIVED: 2018-05-10

DATE REPORTED: 2018-05-17

			SAMPLE DESCRIPTION:	BH1 SS5		BH1 SS9
			SAMPLE TYPE:	Soil		Soil
			DATE SAMPLED:	2018-05-03		2018-05-03
Parameter	**Unit**	**G / S**	**RDL**	**9235030**	**RDL**	**9235031**
pH (2:1)	pH Units		N/A	7.58	N/A	7.98
Sulphate (2:1)	µg/g		8	161	2	93
Chloride (2:1)	µg/g		8	1540	2	278
Resistivity (2:1)	ohm.cm		1	457	1	1830

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

9235030 EC/Resistivity, pH,Chloride and Sulphate were determined on the DI water extract obtained from the 2:1 leaching procedure (2 parts DI water:1 part soil).
Elevated RDL indicates the degree of sample dilution prior to the analysis in order to keep analytes within the calibration range of the instrument and to reduce matrix interference.

9235031 EC/Resistivity, pH,Chloride and Sulphate were determined on the DI water extract obtained from the 2:1 leaching procedure (2 parts DI water:1 part soil).

Certified By: Amanjot Bhela

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

Quality Assurance

CLIENT NAME: EXP SERVICES INC | **AGAT WORK ORDER: 18Z338119**

PROJECT: OTT-245894-AO | **ATTENTION TO: Susan Potyondy**

SAMPLING SITE:1274 Carling Ave. | **SAMPLED BY:exp**

Soil Analysis

RPT Date:			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
Inorganic Chemistry (Soil)															
pH (2:1)	9235030	9235030	7.58	7.62	0.5%	N/A	100%	90%	110%	NA			NA		
Sulphate (2:1)	9235030	9235030	161	156	3.2%	< 2	97%	70%	130%	102%	70%	130%	106%	70%	130%
Chloride (2:1)	9235030	9235030	1540	1480	4.0%	< 2	102%	70%	130%	104%	70%	130%	106%	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.

Certified By: Amanjot Bhela

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

Method Summary

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-245894-AO
SAMPLING SITE:1274 Carling Ave.

AGAT WORK ORDER: 18Z338119
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
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	EC METER

AGAT Laboratories

1Kg

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

Laboratory Use Only

Work Order #: 18Z338119

Cooler Quantity: one-noice

Arrival Temperatures: 16.5 16.8 16.2
/26 | /3/ | /34

Custody Seal Intact: ☐Yes ☐No ☐N/A

Notes:

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: 100-2650 Queensview dr
Ottawa Ontario

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-245894-AO

Site Location: 1274 Carling Avenue

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)	O. Reg 153: ☐ Hydride Metals ☐ 153 Metals (Incl. Hydrides)	ORPs: ☐ B-HWS ☐ Cl ☐ CN⁻ ☐ Cr^{6+} ☐ EC ☐ FOC ☐ Hg ☐ pH ☐ SAR	Full Metals Scan	Regulation/Custom Metals	Nutrients: ☐ TP ☐ NH_3 ☐ TKN ☐ NO_3 ☐ NO_2 ☐ NO_3+NO_2	Volatiles: ☐ VOC ☐ BTEX ☐ THM	PHCs F1 - F4	ABNs	PAHs	PCBs: ☐ Total ☐ Aroclors	Organochlorine Pesticides	TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	Sewer Use	PH	Sulphate	Chloride	Resistivity	
BH 1 SS 5	May 3																					✓	✓	✓	✓	
BH 1 SS 9	2.																					✓	✓	✓	✓	
																						✓	✓	✓	✓	

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Ryan DiGiuseppe	May 10	5:00	B. Mehret	18-05-11	8h35
LB 1050 to Pete	18-05-11	16h00	Simon	18/5/12	10:40

Page 1 of 1

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