Conseil des ecoles catholiques du Centre - Est (CECCE)

Geotechnical Investigation

Type of Document
Final (supersedes June 27, 2018 report)

Project Name
Proposed New Riverside South Catholic Elementary School
925 Ralph Hennessy Avenue
Ottawa, Ontario

Project Number
OTT-00245869-A0

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Date Submitted:
August 28, 2018

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Executive Summary

A geotechnical investigation was recently completed at the site of the proposed new Riverside South Catholic Elementary School to be located in the southeast corner of the Ralph Hennessy Avenue and Mount Nebo Way in Ottawa, Ontario.

The project under consideration is the design and construction of a new two-storey slab-on-grade school without a basement. The new school development will also include the associated access roads, parking lots and playground areas. The proposed design floor slab elevation of the school building will be at Elevation 93.75 m, resulting in a grade raise within the building footprint of approximately 0.4 m to 2.0 m. In the northwest corner of the proposed school building, there will be a localized higher grade raise of up to approximately 2.2 m to 2.7 m.

The fieldwork for the geotechnical investigation was undertaken in two stages, i.e. between April 2 and 5, 2018 and on July 6, 2018 and consisted of the drilling of twenty (20) boreholes (Borehole Nos. 1 to 20) to depths ranging between 1.8 m and 14.4 m. The second stage of the fieldwork was completed to collect additional data on the depth of the liquefiable soils requiring removal from the building envelope, as the location of the building was re-located from its original location identified at the start of the investigation.

Based on a review of the engineering properties of the underlying compressible clay, it is considered that the grade raise at the site should be restricted to 1.8 m. The balance of the required site grade raise above 1.8 m in the interior of the building may be achieved using lightweight fill (LWF).

Foundation options for the proposed building include supporting the building on footings designed to bear on an engineered fill pad and the floor slab designed as a slab-on-grade. This option involves the excavation and removal of the topsoil (surficial and buried), fill, silty sand, sandy silt and potentially liquefiable sandy clay down to the clay and replacing the excavated soils with engineered fill. The second foundation option is to support the building by pile foundation and design the floor slab as a structural slab.

Caisson foundation is not considered suitable due to the shallow groundwater level and silt seams within the soils that may result in difficulties in dewatering the caisson and preventing cave of the caisson sidewalls.

The results of the seismic shear-wave survey and liquefaction analysis indicate that the upper sandy clay is susceptible to liquefaction during a seismic event. The post liquefaction settlement of this layer was estimated to range from 50 mm to 110 mm. For the pile foundation and structural slab option, the site has been classified as Class F for seismic site response in accordance with the 2012 Ontario Building Code (OBC). Class C may be used for the proposed school building if the fundamental period for vibration for the building is less than or equal to 0.5 seconds, as per the 2012 OBC. For the option where the liquefiable soils are removed from the proposed building area and the building is supported by footings with a slab-on-grade all designed to bear on engineered fill, the site class is Class C.

Excavations for the construction of the proposed school building and installation of underground services should be undertaken in accordance with the current Occupational Health and Safety Act (OHSA). De-watering of excavations may be undertaken using conventional pumping techniques.

It is anticipated that all of the excavated soils are not suitable for use as backfill material beneath structural elements. Therefore, any materials required for backfilling purposes will have to be imported to site as per the recommendation of this report. The on-site soils may be used as fill in landscaped areas of the proposed school development.

Normal Portland cement may be used in the subsurface concrete at this site.

For the pile foundation option, it is recommended that additional deep boreholes be undertaken within the footprint of the proposed school building to provide geographic coverage of the building footprint to better delineate the subsurface soil, rock and groundwater conditions and to confirm the geotechnical design parameters presented in this report.

Prior to tendering and to minimize potential contractor claims, consideration should be given to conducting an additional test pit investigation throughout the site, to collect additional data on the quality and depth of the fill.

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

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1 Introduction

EXP Services Inc. (EXP) recently completed a geotechnical investigation at the site of the proposed new Riverside South Catholic Elementary School to be located in the southeast corner of Ralph Hennessy Avenue and Mount Nebo Way intersection in Ottawa, Ontario. This work was authorized by Mr. Luc Poulin on behalf of the Conseil des ecoles catholiques du Centre-Est (CECCE) in a letter dated March 19, 2018.

The project under consideration is the design and construction of a new two-storey slab-on-grade school without a basement. The new school development will also include associated access roads, parking lots and playground areas. The proposed design floor slab elevation of the school building will be at Elevation 93.75 m, resulting in a grade raise within the building footprint of approximately 0.4 m to 2.0 m. In the northwest corner of the proposed school building there will be a localized higher grade raise up to approximately 2.2 m to 2.7 m.

The investigation was undertaken to:

a) Establish the subsurface soil and groundwater conditions at the borehole locations;

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

c) Comment on grade-raise restrictions;

d) Make recommendations regarding the most suitable types 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 types;

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

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

g) Provide pipe bedding requirements for municipal underground services;

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

i) Recommend pavement structure thickness for access roads and parking areas; and

j) Comment on subsurface concrete requirements.

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

Phase I and II Environmental Site Assessments (ESAs) of the proposed school site were undertaken by EXP in conjunction with this geotechnical investigation. The results of the Phase I and II ESAs are presented in two (2) separate reports.

2 Site Description

The site is located on the east side of Ralph Hennessy Avenue and south side of Mount Nebo Way in Ottawa as shown on Figure 1. The site has an area of approximately 2.0 hectares and is currently vacant land. The ground surface elevations at the borehole locations vary between Elevation 93.35 m and 91.98 m. At the time of this investigation, the site was occupied by soil fill piles scattered across the site and low-lying areas of poor drainage with ponded water. Photographs of the site are shown in Appendix A.

3 Procedure

The fieldwork for the geotechnical investigation was undertaken in two stages, i.e. between April 2 and 5, 2018 and on July 6, 2018 and consisted of the drilling of twenty (20) boreholes (Borehole Nos. 1 to 20) to depths ranging between 1.8 m and 14.4 m. The second stage of the fieldwork was completed to collect additional data on the depth of the liquefiable soils requiring removal from the building envelope, as the location of the building was re-located from its original location identified at the start of the investigation.

Ten (10) boreholes (Borehole Nos. 1, 3, 4 and 14 to 20) were located within the proposed building footprint and advanced to depths ranging from 4.0 m to 14.4 m (Elevation 88.3 m to 82.3 m). The remaining ten (10) boreholes (Borehole Nos. 2 and 5 to 13) were located within the access road, parking lot and playground areas and were advanced to depths ranging from 1.8 m to 12.0 m (Elevation 91.0 m to 83.4 m). The borehole locations and geodetic elevations were determined on site by EXP. The geodetic elevation of Borehole No. 7 was not determined. The borehole locations are shown in Figure 2. The fieldwork was supervised on a full-time basis by a representative from EXP.

The borehole locations were cleared of private and public underground services, prior to the start of drilling operations. The boreholes were drilled with a CME-75 track-mounted drill rig equipped with continuous flight hollow-stem auger equipment and rock coring capabilities. Standard penetration tests (ASTM 1586) were performed in all the boreholes at 0.75 m and 1.5 m depth intervals and soil samples retrieved by split-barrel sampler. In addition, relatively undisturbed thin wall tube samples (Shelby tube samples) of the clayey soil were obtained from selected depths in some of the boreholes. The undrained shear strength of the cohesive soils was determined by conducting in-situ field vane tests (ASTM 273). The presence of the bedrock in Borehole Nos. 2 and 4 was proven by conventional coring techniques using an NQ-size core bit. A record of wash water return, colour of wash and any sudden drop of the drill rods were kept during rock coring operations. Borehole Nos. 1 and 6 were advanced to cone refusal depths by conducting the dynamic cone penetration test (DCPT) from 6.7 m to 10.5 m cone refusal depth in Borehole No. 1 and from 6.7 m to 8.8 m cone refusal depth in Borehole No. 6. Borehole No. 5A was advanced by power augering technique from ground surface to a 1.5 m depth and conducting the DCPT from 1.5 m to cone refusal at 9.6 m depth.

Groundwater levels were measured in the open boreholes upon completion of drilling. In addition, 19 mm diameter slotted standpipe piezometers and 50 mm diameter monitoring wells were installed in Borehole Nos. 1, 2, 5 and 6 for long-term monitoring and sampling of the groundwater. The standpipe piezometers and monitoring wells were installed in accordance with EXP standard practice and their installation configuration is documented on the respective borehole log.

On completion of the fieldwork, all the soil samples and rock cores were transported to the EXP laboratory located in the City of Ottawa. All the borehole 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. The tests were undertaken in accordance with the American Society for Testing and Materials (ASTM).

Soil Samples:

Natural Moisture Content	112 tests
Natural Unit Weight	25 tests
Grain-Size Analysis	16 tests
Atterberg Limits	17 tests
Consolidation Test	2 tests
pH, Sulphate and Chlorides Analyses	3 tests

Bedrock Cores:

Unit Weight and Unconfined Compressive Strength Tests	2 tests

4 Subsurface Soil and Groundwater Conditions

A detailed description of the subsurface soil, rock and groundwater conditions determined from the boreholes are given on the attached Borehole Logs, Figure Nos. 3 to 23 inclusive. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted. Boreholes were drilled to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions. Reference is made to the Phase I and II ESA reports regarding the subsurface 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 rock conditions with depth and groundwater conditions.

4.1 Shallow and Deep Boreholes Within Building Footprint (Borehole Nos. 1, 3, 4 and 14 to 20) and Deep Boreholes Outside the Building Footprint (Borehole Nos. 2, 5 and 6)

4.1.1 Topsoil

A surficial 100 mm and 150 mm thick topsoil layer was contacted in Borehole Nos. 4 and 16.

4.1.2 Fill

A surficial fill was encountered in Borehole Nos. 1 to 3, 5, 6, 14, 15, 19 and 20 and beneath the topsoil in Borehole No. 16. The fill extends to depths ranging 0.4 m to 2.4 m (Elevation 92.3 m to 89.8 m). The fill consists of a mixture of silty sand and silty clay with gravel, roots, topsoil and wood debris. Based on the standard penetration test (SPT) N-values of 2 to 14, the fill is in a very loose to compact state. The natural moisture content of the fill is 19 percent to 37 percent. The natural unit weight of the fill ranges from 18.0 kN/m^3 to 20.8 kN/m^3 .

4.1.3 Buried Topsoil

A 100 mm to 200 mm thick topsoil layer exists below the fill in Borehole Nos. 1, 2 and 5.

4.1.4 Silty Sand

The surficial and buried topsoil in Borehole Nos. 1, 2 and 4 are underlain by a 500 mm to 1.3 m thick silty sand which extends to a 1.4 m depth (Elevation 91.4 m to 90.8 m). Based on the SPT N-values of 4 to 12, the silty sand is in a loose to compact state. The natural moisture content of the silty sand is 25 percent to 43 percent. The natural unit weight ranges from 17.4 kN/m^3 to 19.6 kN/m^3.

4.1.5 Sandy Clay

Sandy clay was contacted beneath the buried topsoil, fill and silty sand in all boreholes. The sandy clay was surficially contacted in Borehole Nos. 17 and 18 and is considered to be a possible fill due to its re-worked appearance in Borehole No. 18. Sandy clay is not present in Borehole No. 20. The sandy clay extends to depths ranging from 2.2 m to 3.7 m (Elevation 91.5 m to 89.6 m). Based on the undrained shear strength measurements from the in-situ vane tests of 80 kPa to 120 kPa in Borehole Nos. 1 to 3, the sandy clay has a stiff to very stiff consistency. Based on SPT N-values of 0 to 6 in Borehole Nos. 4 to 6, the sandy clay has a soft to firm consistency. The natural moisture content of the sandy clay ranges from 23 percent to 39 percent and the natural unit weight is 18.5 kN/m^3 to 19.8 kN/m^3.

Grain-size analysis and Atterberg limit determination tests were conducted on several samples of the sandy clay and the results are summarized in Tables I and II. The grain-size distribution curves are shown in Figures 24 to 30.

Table I: Summary of Results from Grain-size Analysis – Sandy Clay Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain-size Analysis (%)		
		Gravel	Sand	Fines
BH1 – SS3	1.5 – 2.1 (91.4 – 90.8)	0	43	57
BH2 – SS4	2.3 – 2.9 (90.3 – 89.7)	0	36	64
BH3 – SS2	0.8 – 1.4 (91.8 – 91.2)	2	44	54
BH4 – SS3	1.5 – 2.1 (90.7 – 90.1)	0	40	60
BH5 – SS4	2.3 – 2.9 (90.7 – 90.1)	0	33	67
BH6 – SS2	0.8 – 1.4 (91.5 – 90.9)	0	37	63
BH16 – SS3	1.5 – 2.1 (90.8 – 90.2)	0	34	66

Table II: Summary of Atterberg Limit Results – Sandy Clay Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Atterberg Limit Results (%)			
		w_n	LL	PL	PI
BH1 - SS3	1.5 – 2.1 (91.4 – 90.8)	39	28	17	11
BH2 – SS4	2.3 – 2.9 (90.3 – 89.7)	28	25	17	8
BH3 – SS2	0.8 – 1.4 (91.8 – 91.2)	25	31	18	13
BH4 – SS3	1.5 – 2.1 (90.7 – 90.1)	34	31	16	15
BH5 – SS4	2.3 – 2.9 (90.7 – 90.1)	35	29	17	12
BH6 – SS3	1.5 – 2.1 (90.8 – 90.2)	34	34	16	18
BH16 – SS3	1.5 – 2.1 (90.8 – 90.2)	36	35	16	19

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 Atterberg limits, the soil may be classified as a low to medium plastic sandy clay (CL) in accordance with the Unified Soil Classification System (USCS).

4.1.6 Sandy Silt

A sandy silt was contacted beneath the sandy clay (possible fill) of Borehole No. 18. The sandy silt extends to 2.2 m depth (Elevation 90.0 m). Based on the SPT-N values of 4 and 5, the sandy silt is in a loose state. The natural moisture content of the sandy silt is 35 percent and 36 percent.

Grain-size analysis and Atterberg limit determination tests were conducted on one (1) sample of the sandy silt and the results are summarized in Table III. The grain-size distribution curve is shown in Figure 31.

Table III: Summary of Results from Grain-size Analysis – Sandy Silt Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain-size Analysis (%)		
		Gravel	Sand	Fines
BH18 – SS2	0.8 – 1.4 (91.4 – 90.8)	0	46	54

The Atterberg limit determination indicated the soil is non-plastic.

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

4.1.7 Clay

The topsoil, fill and sandy clay in all boreholes are underlain by clay to silty clay comprising of an upper desiccated brown clay crust underlain by grey clay.

Upper Desiccated Brown Clay Crust

The upper crust is present in only Borehole Nos. 1,4 and 6. The clay crust was contacted at 2.2 m and 3.0 m depths (Elevation 90.1 m to 89.8 m) and extends to depths ranging from 2.2 m to 3.6 m (Elevation 89.4 m to 89.2 m). Based on SPT N-values of 0 and 1, the consistency of the clay is very soft. Based on undrained shear strength value of greater than 120 kPa, the clay has a very stiff consistency. The natural moisture content of the clay crust is 34 percent to 59 percent.

Atterberg limit determination was conducted on one (1) sample of the brown clay crust and the results are presented in Table IV.

Table IV: Summary of Atterberg Limit Results – Brown Clay Crust Sample

Borehole No. - Sample No.	Depth (Elevation) (m)	Atterberg Limit Results (%)			
		w_n	LL	PL	PI
BH6 – SS4	2.3 – 2.9 (90.0 – 89.4)	34	49	21	28

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

The results from the Atterberg limit test indicate the clay crust may be classified as a medium plastic clay (CL) in accordance with the Unified Soil Classification System (USCS).

Grey Clay to Silty Clay

Th grey clay was contacted at 2.2 m to 3.7 m depths (Elevation 90.4 m to 89.2 m) below the topsoil, fill, sandy clay, sandy silt and brown clay crust. The grey clay extends to depths ranging from 3.7 m to 9.8 m (Elevation 88.2 m to 82.4 m). The grey clay contains shell fragments below the 5.6 m depth (Elevation 87.0 m) in Borehole No. 3. In Borehole Nos. 4 and 6, the clay grades to a silty clay containing silt seams below
5.7 m and 6.1 m depths (Elevation 86.5 m and 86.2 m), respectively. The undrained shear strength of the grey clay is 19 kPa to 67 kPa indicating a soft to stiff consistency. The undrained shear strength of the silty clay is 67 kPa to greater than 120 kPa in Borehole No. 4 indicating a stiff to very stiff consistency. The higher undrained shear strength measurements in the silt clay may be attributed to the presence of silt seams. In Borehole No. 6, the SPT N-value of the silty clay is 5 indicating a firm consistency. The natural moisture content of the grey clay is 36 percent to 88 percent. The natural moisture content of the silty clay is 29 percent to 46 percent.

Grain-size analysis and Atterberg limit determination tests were conducted on several samples of the grey clay and the results are summarized in Tables V and VI respectively. The grain-size distribution curves are shown in Figures 32 to 34.

Table V: Summary of Results from Grain-size Analysis – Grey Clay to Silty Clay Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain-size Analysis (%)		
		Gravel	Sand	Fines
BH3 – SS8	6.1 – 6.7 (86.5 – 85.9)	1	8	91
BH4 – SS9	7.6 – 8.2 (84.6 – 84.0)	2	2	96
BH6 – SS8	6.1 – 6.7 (86.2 – 85.6)	1	2	97

Table VI: Summary of Atterberg Limit Results – Grey Clay to Silty Clay Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Atterberg Limit Results (%)			
		w_n	LL	PL	PI
BH1 – SS7	4.6 – 5.2 (88.2 – 87.6)	67	52	22	30
BH3 – SS6	3.8 – 4.4 (88.8 – 88.2)	72	48	23	25
BH4 – SS9	7.6 – 8.2 (84.6 – 84.0)	37	30	17	13
BH6 – SS6	3.8 – 4.4 (88.5 – 87.9)	76	47	22	25
BH6 – SS8	6.1 – 6.7 (86.2 – 85.6)	29	30	18	12

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 Atterberg limits, the grey clay may be classified in accordance with the USCS as a clay to silty clay of low to medium plasticity (CL) and a clay of high plasticity (CH).

One-dimensional oedometer test was performed on two (2) relatively undisturbed samples of the grey clay and the test results are summarized in Table VII. The test results are shown in Figures 35 and 36.

Table VII: Summary of Results from One-Dimensional Oedometer Tests on Grey Clay Samples

Borehole No.-Sample No.	Depth (Elevation) (m)	σ'_{v0} (kPa)	W_c (%)	γ (kN/m^3)	σ'_p (kPa)	e_o	c_r	c_c	OC (kPa)	OCR
BH 1 – ST6	3.8 – 4.4 (89.0 – 88.4)	58	68	15.7	95	1.873	0.042	1.56	37	1.6
BH6 – ST5	3.3 – 3.4 (89.0 – 88.9)	43	67	15.8	95	1.856	0.038	1.49	52	2.2

σ'_{v0} = calculated effective overburden pressure (kPa); Wc: natural moisture content (%), γ: estimated natural unit weight σ'_p = pre-consolidation pressure (kPa), e_o = initial void ratio, ; c_r = re-compression index; c_c = compression index; OC= available over-consolidation pressure (kPa); OCR -Over-Consolidation Ratio

Note: σ'_{v0} calculated using May 7, 2018 groundwater level measurement and assuming an average groundwater level measurement of 1.3 m (Elevation 91.0 m) for Borehole No. 6.

The test results indicate the grey clay is over-consolidated.

4.1.8 Sandy Clay

The grey silty clay in Borehole No. 4 grades to a sandy clay below the 9.8 m depth (Elevation 82.4 m) to a 10.5 m depth (Elevation 81.7 m). Based on the SPT N-value of 3, the consistency of the sandy clay is soft. The natural moisture content is 22 percent.

The results from the grain-size analysis conducted on one (1) sample of the grey sandy clay is summarized in Table VIII. The grain-size distribution curve is shown in Figure 37.

Table VIII: Summary of Results from Grain-size Analysis – Sandy Clay Sample

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain-size Analysis (%)		
		Gravel	Sand	Fines
BH4 – SS10	9.9 – 10.5 (82.3 – 81.7)	9	43	48

Based on a review of the results from the grain-size analysis, the soil may be classified as a sandy clay (CL) as per the USCS.

4.1.9 Glacial Till

The clay is underlain by glacial till contacted at 3.7 m to 10.5 m depths (Elevation 89.0 m to 81.7 m) in Borehole Nos. 1,2 4 and 5. The till extends to depths of 10.1 m and 11.3 m (Elevation 82.5 m and 80.9 m) in Borehole Nos. 2 and 4. Based on the SPT N-values of 13 to 85, the glacial till is in a compact to very dense state. Higher N values are likely a result of the presence of cobbles and boulders within the glacial till. The natural moisture content of the glacial till is 7 percent to 17 percent.

The results of the grain-size analysis conducted on one (1) sample of the glacial till are summarized in Table IX. The grain-size distribution curve is shown in Figure 38.

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

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain-size Analysis (%)		
		Gravel	Sand	Fines
BH2 – SS9	7.6 – 8.2 (85.0 – 84.4)	31	40	29

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

Atterberg limits determination were conducted on one (1) sample from Borehole No. 2 (SS8: 6.1 m - 6.7 m, Elevation 86.5 m – 85.9 m) and one (1) sample from Borehole No. 4 (SS11: 10.7 m – 11.3 m, Elevation 81.5 m – 80.9 m). The results indicate the glacial till is non-plastic.

4.1.10 Bedrock

Inferred Boulders and Bedrock

Based on the auger and dynamic cone refusal criteria, boulders and bedrock are inferred in Borehole Nos. 1, 5, 5A and 6 at 7.7 m to 10.5 m depths (Elevation 85.3 m to 82.3 m).

The presence of limestone bedrock was confirmed by coring the bedrock in Borehole Nos. 2 and 4. Bedrock was contacted at 10.1 m and 11.3 m depths (Elevation 82.5 m and 80.9 m) in Borehole Nos. 2 and 4, respectively. The Total Core recovery (TCR) is 64 percent and 100 percent. The Rock Quality Designation (RQD) ranges from 31 percent to 91 percent indicating the bedrock is of a poor to excellent quality.

The results of the unit weight determination and unconfined compressive strength test conducted on two (2) rock core sections are summarized in Table X.

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

Borehole No.- Sample No.	Depth (Elevation) (m)	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)
BH2 – Run 2	10.6 – 10.8 (82.0 – 81.8)	25.8	179.2
BH4 – Run 3	13.1 – 13.3 (79.1 – 78.9)	25.4	191.5

The unconfined compressive strength test results indicate the strength of the rock may be classified as very strong in accordance with the Canadian Foundation Engineering Manual (CFEM), Fourth Edition, 2006.

Based on a review of published geology maps, the site lies near the transition zone between dolomite and limestone bedrock of the Oxford formation and interbedded sandstone and sandy dolomite bedrock of the March formation. The majority of the site appears to be underlain by the Oxford formation with the northeast corner of the site possibly underlain by the March formation.

4.1.11 Groundwater Levels

Groundwater level observations were made in the boreholes during drilling and in standpipe piezometers and monitoring wells installed in boreholes subsequent to the completion of drilling operations. The recent groundwater level measurements made in the standpipe piezometers and monitoring wells in the boreholes are summarized in Table XI.

Table XI: 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 1	92.81	April 2, 2018	May 7, 2018 (35 days)	2.2	90.6
BH 2	92.55	April 3, 2018	May 7, 2018 (34 days)	1.6	91.0
BH 5	92.97	April 2, 2018	May 7, 2018 (35 days)	1.5	91.5
BH 6	92.32	April 2, 2018	May 7, 2018 (35 days)	0.3	92.0

The measured groundwater level in Borehole No. 6 is high compared to the measured groundwater levels in the remaining standpipe piezometers. The high groundwater level may be a result of a broken bentonite seal in the standpipe piezometer installation. Therefore, the groundwater level measurement in Borehole No. 6 is not considered reliable.

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.

4.2 Shallow Boreholes Outside the Building Footprint (Borehole Nos. 7 to 13)

4.2.1 Topsoil

A surficial 300 mm and 400 mm thick topsoil layer was identified in Borehole Nos. 9, 11 and 12.

4.2.2 Fill

Fill was contacted beneath the topsoil in Borehole No. 11 and at ground surface in the remaining boreholes. The fill extends to depths ranging from 0.6 m to 1.2 m depths (Elevation 92.2 m to 91.4 m). The fill consists of a mixture of gravel, sand, silt, clay, decayed grass shoots and topsoil. The fill contains a 200 mm thick topsoil layer at a 0.3 m depth. Based on the SPT N values of 2 to 9, the fill is in a very loose to compact state. The natural moisture content of the fill is 27 percent to 57 percent and the natural unit weight of the fill is 17.5 kN/m^3 .

The results of the grain-size analysis conducted on one (1) sample of the fill are summarized in Table XII. The grain-size distribution curve is shown in Figure 39.

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

Borehole No. - Sample No.	**Depth (Elevation) (m)**	**Grain-size Analysis (%)**		
		Gravel	**Sand**	**Fines**
BH11 – SS2	0.6 – 1.2 (91.9 – 91.3)	0	38	62

Based on a review of the results of the grain-size analysis, the fill may be classified as sandy clay (CL) in accordance with the USCS.

4.2.3 Buried Topsoil Layer

A topsoil layer was identified beneath the fill in Borehole Nos. 7, 8 and 13. The topsoil ranges in thickness from 150 m to 250 mm.

4.2.4 Sandy Clay

The surficial and buried topsoil and fill are underlain by sandy clay contacted at 0.3 m to 1.2 m depths (Elevation 92.2 m to 91.3 m) in Borehole Nos. 7 to 12. The sandy clay in Borehole No. 7 may be a possible fill. The sandy clay contains roots and rootlets from 0.3 to 1.2 m depths in Borehole No. 12. Based on the SPT N-values of 3 to 9, the sandy clay has a soft to stiff consistency. The natural moisture content of the sandy clay ranges from 29 percent to 63 percent. The natural unit weight of the sandy clay is 17.8 kN/m^3 to 19.3 kN/m^3 .

The results of the grain-size analysis conducted on two (2) samples of the soil are summarized in Table XIII. The grain-size distribution curves ae shown in Figures 40 and 41.

Table XIII: Summary of Results from Grain-size Analysis – Sandy Clay Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain-size Analysis (%)		
		Gravel	Sand	Fines
BH7 – SS3	1.2 – 1.8	0	35	65
BH8 – SS2	0.6 – 1.2 (91.5 – 90.9)	0	39	61

Based on a review of the results of the grain-size analysis, the soil may be classified as sandy clay (CL) in accordance with the USCS.

4.2.5 Silty Clayey Sand

The buried topsoil in Borehole No. 13 is underlain by silty clayey sand contacted at 0.9 m depth (Elevation 91.9 m). Based on the SPT N-values of 4 and 7, the sand is in a compact state and has a firm consistency. The natural moisture content of the sand is 34 percent and 51 percent.

The results of the grain-size analysis conducted on one (1) sample of the soil are summarized in Table XIV. The grain-size distribution curve is shown in Figure 42.

Table XIV: Summary of Results from Grain-size Analysis – Sandy Clay Samples

Borehole No. - Sample No.	Depth (Elevation) (m)	Grain-size Analysis (%)		
		Gravel	Sand	Fines
BH13 – SS3	1.2 – 1.8 (91.6 – 91.0)	0	55	45

Based on a review of the results of the grain-size analysis, the soil may be classified as a silty clayey sand (SC-SM) in accordance with the USCS.

4.2.6 Groundwater Levels

All boreholes remained dry upon completion of drilling operations.

5 Site Classification for Seismic Site Response and Potential for Liquefaction

5.1 Liquefaction Potential

The geotechnical investigation revealed that the subsurface conditions at the site consist of topsoil (surficial and buried) and fill underlain by native silty sand, sandy silt, sandy clay, clay, silty clay and glacial till overlying limestone bedrock. The groundwater level as measured at 1.5 m to 2.2 m depths below existing grade.

Within the building envelope, the silty sand and sandy silt were not considered in the liquefaction potential evaluation, since these soils require excavation and removal due to being unsuitable to support the proposed building on footings.

The results of the natural moisture contents and Atterberg limits of the fine-grained soils including the sandy clay (upper and lower layers), the underlying brown and grey clay and grey silty clay within the proposed building footprint were evaluated for the potential to liquefy during a seismic event using the criteria for liquefaction potential of fine-grained soils by Bray et al. (2004) and the chart is shown in Figure 43. The chart indicates that the upper sandy clay is susceptible to moderately susceptible to liquefaction during a seismic event. The underlying brown and grey clay are not susceptible to liquefaction. The deeper grey silty clay in Borehole Nos. 4 and 6 are susceptible to moderately susceptible to liquefaction.

Further assessment of the liquefaction potential of the subsurface soils was undertaken by conducting a seismic shear wave survey using Multi-channel Analysis of Surface Waves (MASW), the Extended Spatial AutoCorrelation (ESPAC) and seismic refraction methods. The results of the survey are shown in Appendix B. The results of the survey indicate that very low to low seismic velocities of less than 200 m/s were calculated from the ground surface to approximately a 5.0 m depth below existing grade along the survey alignment indicating the soils in the upper approximate 5.0 m may have the potential liquefy during a seismic event. Since the upper silty sand and sandy silt have been eliminated for reasons previously discussed, the remaining soil types within the upper 5.0 m of very low to low seismic velocities that are susceptible to moderately susceptible to liquefaction is the upper sandy clay layer, the underlying brown and grey clay and the grey silty clay.

Based on a review of the grain-size analysis and the Atterberg limits, the sandy clay has a sufficient amount of sand and clay particle sizes to cause the sandy clay to exhibit not only cohesive clay-like behavior but also cohesionless sand-like behavior making it susceptible to liquefaction during a seismic event. Therefore, the liquefaction potential for the upper sandy clay and the underlying soils was further evaluated by determining the characteristics of the materials through the ground response analysis using equivalent-linear total stress approach, in according with the 2006 Canadian Foundation Engineering Manual, Fourth Edition. The empirical methods on the basis of NCEER (2001) and Seed and Idriss (1971) were utilized to compare the cyclic resistance ratio (CRR) of the soils to the cyclic stress ratio (CSR) caused by an earthquake, respectively. The risk of soil liquefaction was assessed based on a characteristic earthquake of magnitude 6.5. The analysis results indicated that the calculated factors of safety against liquefaction were less than 1.0 for the upper sandy clay and greater than 1.0 for the remaining underlying soils. Therefore, the upper sandy clay is considered to have a potential to liquefy during a seismic event, while the underlying soils are not considered to have a potential to liquefy during a seismic event.

The volumetric reconsolidation strain of the upper sandy clay layer is 6.3 percent to 6.9 percent and the anticipated total post liquefaction settlement is estimated to be 50 mm to 110 mm.

The following two (2) foundation and floor slab options were considered to address the potentially liquefiable sandy clay:

1) Footings on an Engineered Fill Pad – Excavate and remove the topsoil (surficial and buried), fill, silty sand, sandy silt and liquefiable sandy clay down to the clay from within the building footprint and replace with engineered fill. Support the proposed building on spread and strip footings founded within the engineered fill pad and design the floor slab as a slab-on-grade also founded on the engineered fill pad. With this foundation option, the potentially liquefiable sandy clay is removed from the building envelope and liquefication becomes a non-issue.

2) Pile Foundations – Support the proposed building on pile foundations driven into the bedrock and design the floor slab as a structural slab supported by the pile foundation.

5.2 Seismic Site Classification

For the pile foundation and structural slab option, since the upper sandy clay layer is considered to have a potential to liquefy during a seismic event, the site classification for seismic site response would be **Class F**. However, if the fundamental period of vibration for the proposed school building is equal to or less than 0.5 seconds, the site may be classified as **Class C**. If the fundamental period of vibration for the structures is greater than 0.5 seconds, this office should be contacted to provide revised parameters for seismic design.

For the option of removing the liquefiable soils, replacing with engineered fill and supporting the proposed building on footings and the slab-on-grade all on the engineered fill pad, the seismic site class will be **Class C**.

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

6 Grade Raise Restriction

The proposed design floor slab elevation will be at Elevation 93.75 m. The existing ground surface elevations of the site are shown by spot elevations on Drawing No. A01.1 titled, “Site Plan, Nouvelle Ecole Elementaire a Riverside-Sud”, dated May 30, 2018 and prepared by Pye and Richards Architects Inc. Based on a review of the spot elevations shown on Drawing No. A01.1 and the ground surface elevations of the boreholes located within the proposed building footprint, the proposed site grade raise under the building will range from approximately 0.4 m to 2.0 m. In the northwest corner of the building, the site grade raise will range from approximately 2.2 m to 2.7 m.

The investigation has revealed that the grey clay at the site is slightly over-consolidated to over-consolidated. Consequently, large settlements of the clay will result if it is loaded beyond its pre-consolidation pressure. To maintain the settlements within normally tolerated limit of 25 mm total, the site grade raise is restricted to 1.8 m using conventional soil fill. The balance of the proposed site grade raise above 1.8 m may be achieved by the placement of lightweight fill (LWF).

At entrances to the building, a combination of soil fill and LWF should be used to minimize differential settlement between the floor and sidewalk entranceways. The LWF should be placed over a horizontal distance of 2.4 m from the exterior side of the building.

For the parking lot and access road areas, the total site grade raise may be raised by the placement of soil fill. Some consolidation settlement of the clay should be expected over time requiring future maintenance of the pavement structures.

The estimated total settlement of the underground services from the placement of the anticipated site grade raise of approximately 1.6 to 2.7 m using soil fill is estimated to range from 19 mm to 160 mm. Therefore, provided this range of settlement can be tolerated by the underground services, no special measures are required for the service pipes.

The LWF may consist of extended polystyrene (EPS) blocks conforming to ASTM C578 specification with a normal density of 21.6 kg/m^3, a compressive strength of 103 kPa – 145 kPa at 10 percent strain, water absorption of 1.0 to 3.5 percent and tolerances within 0.5 percent for thickness, flatness and squareness. The LWF blocks should be tightly fitted to the walls of the excavation without voids. The LWF blocks should be fixed on all sides to the adjacent blocks with Building Grip PL300 construction adhesive. If another layer of lightweight blocks is required, it should be installed at right angles to the previous layer with blocks fitting tightly leaving no voids. The LWF should be covered with geotextile (such as Terrafix 270R or equivalent) prior to placement of soil or granular fill.

An allowance for groundwater lowering was not required as part of the grade raise review, since measures will be employed in new service trenches to minimize the permanent lowering of the groundwater level (use of clay seals), as recommended in Section 10.

The final grading plan should be reviewed by EXP to confirm that the grade raise is in accordance with the recommendations of this report.

7 Site Grading

Site grading within the footprint of the proposed building, playgrounds, future portable, etc. and paved areas should consist of the excavation and removal of all existing topsoil (surficial and buried), fill and organic stained soils from the site down to the native undisturbed soil. For the option of supporting the proposed building by footings designed to bear on an engineered fill pad, the existing topsoil (surficial and buried), fill, silty sand, sandy silt and sandy clay would have to be excavated and removed down to the clay from within the building footprint. In addition, the stockpiles of fill currently present on-site should be removed and disposed off site. For budgeting purposes, the contractor should assume that all existing fill in the building area, playgrounds, portable, parking and access road areas would require removal and replacement with well-compacted fill as indicated below.

For guidance, the approximate excavation depths at the borehole locations within the proposed building footprint to remove the topsoil (surficial and buried), fill, silty sand, sandy silt and sandy clay down to the native clay surface are summarized in Table XV.

Table XV: Summary of Excavation Depths (Elevations) Within Building Footprint

Borehole No.	Ground Surface Elevation (m)	Approximate Excavation Depth (Elevation), m
1	92.81	3.0 (89.8)
3	92.64	2.9 (89.7)
4	92.20	2.2 (90.0)
14	93.35	3.7 (89.7)
15	92.73	2.3 (90.4)
16	92.31	2.6 (89.7)
17	91.98	2.2 (89.8)
18	92.21	2.2 (90.0)
19	92.10	2.2 (89.9)
20	92.21	2.4 (89.8)

The exposed native subgrade should be examined by a geotechnical engineer. Following approval, the grades at the site may be raised to the permissible level by the placement of engineered fill consisting of Ontario Provincial Standard Specification (OPSS) Granular B Type II placed in 300 mm thick lifts, with each lift compacted to 100 percent standard Proctor maximum dry density (SPMDD) within the building footprint beneath the footings, to 98 percent SPMDD beneath the floor slab and 95 percent SPMDD elsewhere.

For the construction of the engineered fill pad for footing and slab-on-grade support, the excavation of the topsoil (surficial and buried), fill, silty sand, sandy silt and sandy clay should extend sufficient distance beyond the limits of the proposed building footprint to accommodate a 1.0 m wide bench of engineered fill around the perimeter of the proposed building, which should thereafter be sloped at an inclination of 1H:1V. As noted above, the exposed native subgrade should be examined by a geotechnical engineer. Following approval, the engineered fill should be placed in 300 mm thick lifts and each lift should be compacted.

The engineered fill should be placed under the full-time supervision of a geotechnician working under the direction of a geotechnical engineer. In-place density tests should be undertaken on each lift of the engineered fill to ensure that it is properly compacted prior to placement of subsequent lift.

8 Foundation Considerations

The geotechnical investigation has revealed that the site is underlain by topsoil (surficial and buried), fill, silty sand which overlies a sandy clay that may undergo an estimated post liquefaction settlement of 50 mm to 110 mm as indicated in Section 5. Due to the estimated post liquefaction settlement, it is not considered feasible to support the proposed school building by shallow footings designed to bear on the upper sandy clay. The following two (2) foundation options are considered feasible to support the proposed building:

- Footings on engineered fill, following the removal of the liquefiable soils and replacement with well compacted engineered fill; and
- Pile foundation.

Caisson foundation is not considered suitable at this site due to the high-water table and due to the presence of possible water bearing silt seams throughout the site, which will make dewatering of the caissons as well as preventing the side walls from caving in very difficult.

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

8.1 Footings on Engineered Fill

For this option, the existing topsoil (surficial and buried), fill, silty sand, liquefiable sandy silt and sandy clay will have to be excavated and removed to the surface of the native undisturbed clay and replaced with engineered fill as described in Sections 5 and 7 of this report.

The proposed floor slab elevation will be Elevation 93.75 m and the footings will be founded at Elevation 92.10 m on the engineered fill pad. For strip footings having a maximum width of 1.0 m and square footings having a maximum 3.0 m by 3.0 m founded on the engineered fill at the proposed founding elevation of Elevation 92.10 m, the footings may be designed for a bearing capacity at serviceability limit state (SLS) of 150 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 225 kPa. The factored ULS value includes a resistance factor of 0.5. The SLS and ULS values are valid provided the site grade restriction of 1.8 m is respected for this project. Settlement of the footings designed for the SLS bearing pressure recommended above and properly constructed are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential movements.

A minimum of 1.5 m of earth cover should be provided to the footings of heated structures to protect them from damage due to frost protection. The frost cover should be increased to 2.1 m for unheated structures if snow will not be removed from their vicinity. If snow will be removed from the vicinity of the unheated structures, the frost cover should be increased to 2.4 m depth. In addition, it is recommended that 100 mm thick HI-40 insulation should be placed at entrances and doors and extending a distance of 2.4 m from the edge of the structure to minimize differential frost heave during the freeze-thaw cycles.

All the footings beds should be examined by a geotechnical engineer to ensure the founding surfaces are capable of supporting the design bearing pressures and that the footing beds have been properly prepared.

8.2 Pile Foundation

The proposed building may be supported by closed end concrete filled steel pipe or steel H pile foundation. Within the proposed building location, the piles are expected to meet practical refusal on bedrock at an approximate 11.3 m depth below the existing ground surface, i.e. Elevation 80.9 m based on the information from Borehole No. 4 located within the building footprint. As noted previously, only three (3) boreholes are currently situated within the envelope of the proposed building due to its relocation by the design team following the completion of the fieldwork.

Closed-end concrete filled pipe piles or steel H piles are considered to be the most suitable type of pile sections. However, closed end concrete filled pipe piles are expected to be more economical.

The factored geotechnical resistance at ULS has been given on Table XIV. Since the piles are expected to meet refusal in the bedrock, the factored geotechnical resistance at ULS will govern the design. The factored ULS values include a resistance factor of 0.40. The factored ULS values should be considered preliminary and will have to be confirmed from bedrock information obtained from the additional boreholes.

The increase in the site grades will cause consolidation settlement of the clay. This will result in down-drag forces on the piles which will have to be taken into consideration in the design of the piles. The unfactored down-drag forces on the piles have been estimated and are shown in Table XVI.

Table XVI: Factored Geotechnical Resistance at Ultimate Limit State (ULS) of Steel Pipe and H Piles

Type of Pile	Size	Factored Geotechnical Resistance at ULS (kN)	Unfactored Negative Skin Friction (kN)
Steel Pipe	245 mm O.D. by 10 mm wall thickness	1,275	235
	245 mm O.D. by 12 mm wall thickness	1,445	235
	324 mm O.D. by 12 mm wall thickness	2,120	315
Steel H	HP 310 x 79	1,260	370
	HP 310 x 110	1,775	380
	HP 310 x 125	2,000	385

The above factored geotechnical resistance at ULS was based on steel piles with a yield strength of 350 MPa and concrete compressive strength of 35 MPa.

Total settlement of piles designed for the recommended factored ULS values and installed as noted below are expected to be less than 10 mm.

In accordance with the 2006 Canadian Foundation Engineering Manual (CFEM), down-drag or negative skin friction forces and transient live loads should not be combined. Two (2) separate loading conditions must be considered in design; permanent load plus drag load but no transient live load, permanent load and transient live load but no drag load.

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

The glacial till is expected to contain cobbles and boulders. It is therefore recommended that the pile tips should be reinforced with a 25-mm thick steel plate and equipped with a driving shoe in accordance with Ontario Provincial Standard Drawing (OPSD) 3001.100, Type II, dated November 2010 and shown in Appendix D.

A number of test piles (5 percent of the total number of piles) should be monitored with the Pile Driving Analyzer (PDA) during the initial driving and re-striking at the beginning of the project and 3 percent of the piles tested should be subjected to CAPWAP analysis. This monitoring will allow for the evaluation of transferred energy into the pile from the hammer, determination of driving criteria and an evaluation of the geotechnical resistance at ULS of the piles. Depending on the results of the pile driving analysis, the pile capacity may have to be proven by at least one pile load test for each pile type before production piling begins. If necessary, the pile load test should be performed in accordance with ASTM D 1143.

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

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

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

A granular mat at least 600 mm thick will be required to provide access to the pile driving rig following the removal of the topsoil. The thickness of the required granular mat would have to be established by the piling contractor based on the type of piling rig that will be used on-site.

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

8.2.1 Uplift Capacity

The estimated factored ULS geotechnical uplift resistance of the piles are given in Table XV. The factored ULS geotechnical uplift resistance is based on an embedment length in the clay of 9.1 m and includes a factored geotechnical resistance of 0.30. The uplift capacities listed on Table XVII do not include the dead weight of the piles.

Table XVII: Factored Ultimate Limit State (ULS) Uplift Resistance of Steel Pipe and H Piles

Type of Pile	Size	Factored Geotechnical Uplift Resistance at ULS (kN)
Steel Pipe	245 mm OD x 10 mm wall thickness	90
	245 mm OD x 12 mm wall thickness	90
	324 mm OD x 12 mm wall thickness	115
Steel H	HP 310 x 79	140
	HP 310 x 110	140
	HP310 x 125	145

8.2.2 Lateral Resistance of Piles

The ultimate lateral resistance (capacity), H_u, and deflection, y_g, at ground level of the piles may be estimated using the charts provided by Broms' (1964) shown below for short and long piles and for free head and restrained head conditions.

The pile is determined to be long or short in cohesive soil by the following criteria:

$$\beta h = \beta = \left(\frac{kb}{4EI}\right)^{1/4}$$

$\beta h\ D > 2.25$ (long pile)

$\beta h\ D < 2.25$ (short pile)

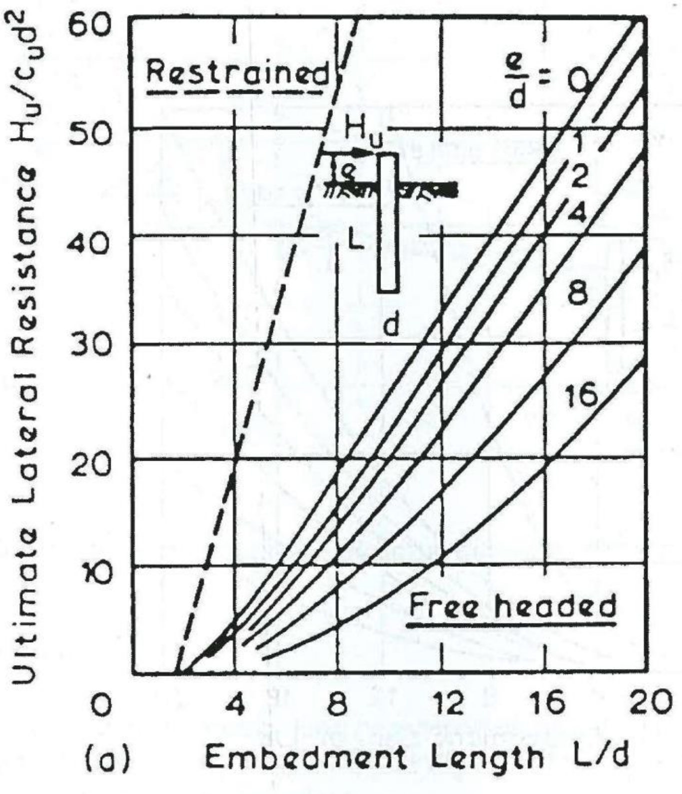
Where: b = diameter or width of the pile, m
D= embedment depth of the pile, m
E = modulus of elasticity of pile, MPa
I = moment of inertia of pile, m^4
k = k_h = coefficient of horizontal subgrade reaction = 2,880/b where b is in metres and k_h is in kPa /m^3

The following geotechnical parameters may be used in determining the lateral resistance:

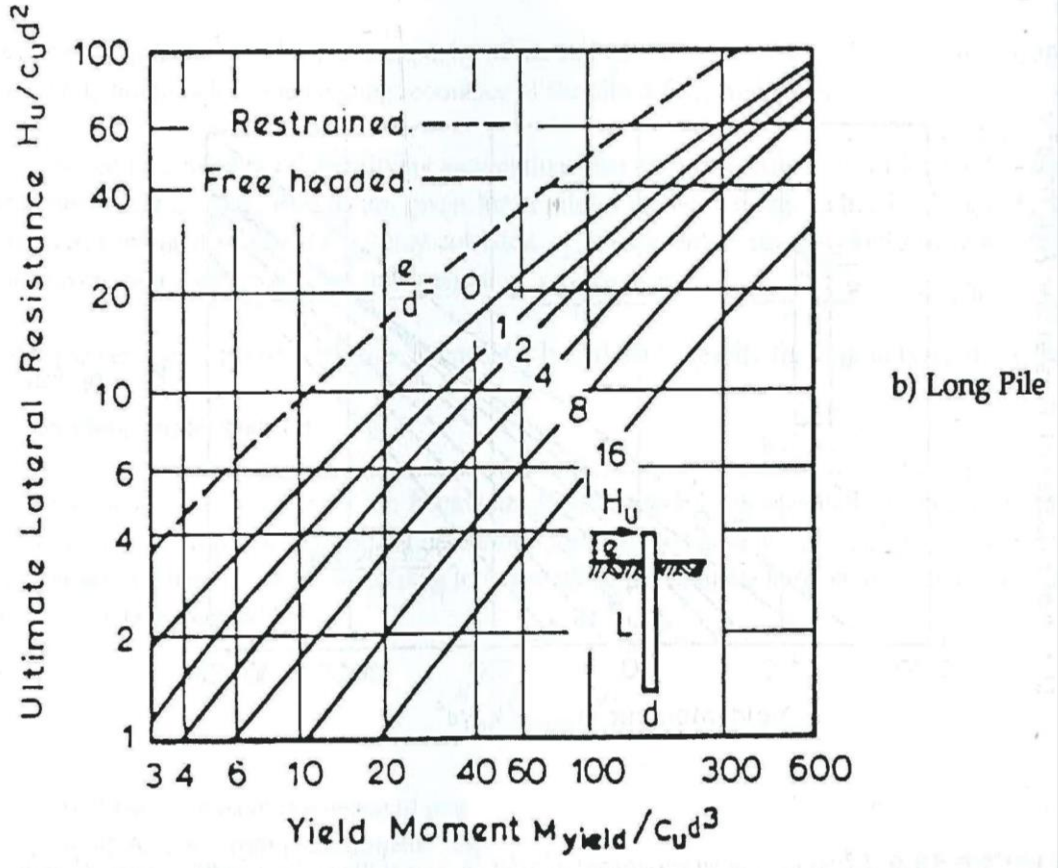
H_u = Lateral load capacity of pile
C_u = Clay Cohesion = 50 kPa
d = Diameter or width of pile, m
L = Embedded length, m
γ = unit weight of soil = 16 kN/m^3
M yield = Yield moment of pile
e = Height of lateral load above ground surface

The computed resistance should be multiplied by a geotechnical resistance factor of 0.5.

For βhD values between 2.0 and 2.5, both long and short pile criteria should be considered and the smaller value used in design.

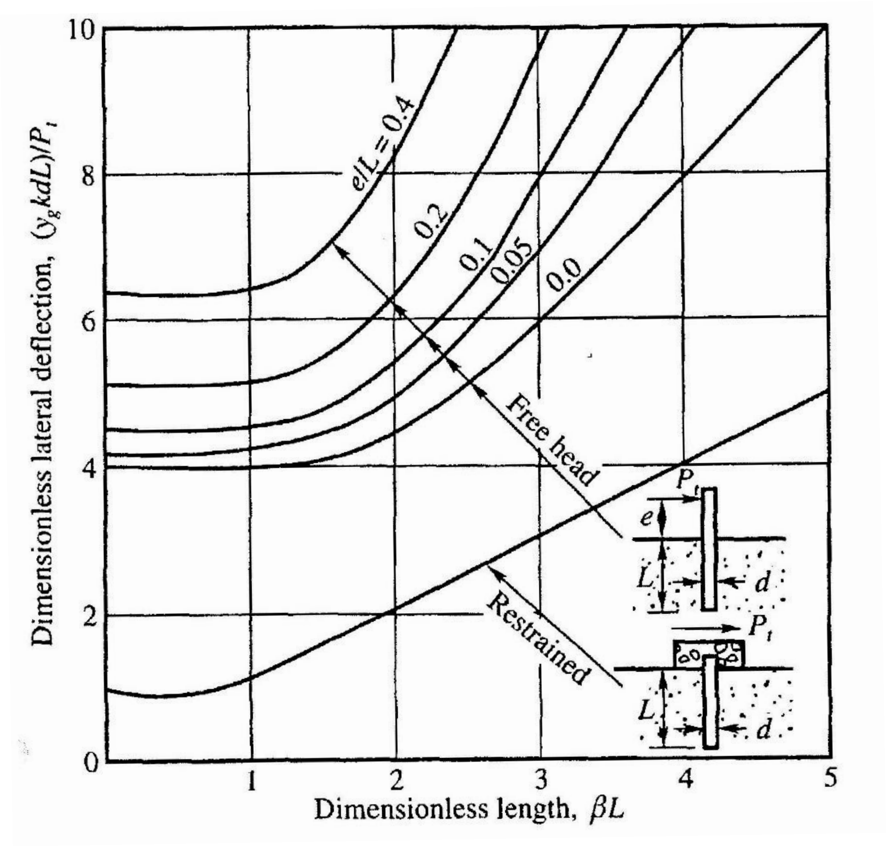


a) Short Pile



Ultimate lateral resistance of piles in cohesive soils (after Broms, 1964a)

The deflection, y_g, at ground surface of the pile maybe calculated from the following graph:



Lateral Deflection at the Ground Surface of Horizontally Loaded Pile in Cohesive Soil (Broms, 1964 a)

Where:

$$\beta h = \beta$$

P_t = Lateral load applied at or above the ground level

The lateral capacity of a single pile may also be computed by using computer software such as L-pile.

8.3 Additional Comments

For the pile foundation option, it is recommended that additional deep boreholes be undertaken within the footprint of the proposed school building to also provide geographic coverage of the building footprint to better delineate the lateral extent and depth of the subsurface soils and bedrock and to confirm the pile depth as well as uplift capacity and lateral resistances of the piles.

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

9 Floor Slab and Drainage Requirements

For the foundation option of supporting the proposed building on footings constructed on engineered fill, the floor slab may be designed as a slab-on-grade provided it is placed on a well compacted bed of 19 mm clear stone at least 200 mm thick set on the engineered fill pad constructed as per the recommendations provided in Sections 5 and 7 of this report. The clear stone will prevent the capillary rise of moisture from the engineered fill pad to the floor slab. The grade raise restriction noted in Section 6 should be respected, i.e. by the use of combination of mineral fill as well as light weight fill.

The structural slab should also be placed on a 200 mm thick bed of well packed 19 mm clear stone.

A perimeter drainage system should be provided for the slab-on-grade or structural slab around the proposed building if the final exterior site grades around the building will be less than 150 mm below the floor slab elevation.

The groundwater level is at Elevation 92.0 m to 90.6 m, which is 1.8 m to 3.2 m below the proposed floor slab elevation of Elevation 93.75 m. In this case, an underfloor drainage system is not required.

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

10 Pipe Bedding Requirements

It is recommended that the bedding for the underground services including material specifications, thickness of cover material and compaction requirements conform to City of Ottawa requirements and/or Ontario Provincial Standard Specification and Drawings (OPSS and OPSD).

Due to the presence of the sandy clay and clay and shallow groundwater level, it is recommended the pipe bedding consist of 300 mm thick OPSS 1010 Granular B Type II sub-bedding material overlain by 150 mm thick OPSS 1010 Granular A bedding material. The bedding materials should be compacted to at least 95 percent SPMDD.

The bedding thickness may be further increased in areas where the sandy clay and clay subgrades become disturbed. Trench base stabilization techniques, such as removal of loose/soft material, placement of crushed stone sub-bedding (Granular B Type II), completely wrapped in a non-woven geotextile, may also be used if trench base disturbance becomes a problem in wet or soft areas.

The service pipes in the exterior of the building should be equipped with flexible joints and/or backfilled with LWF if the service pipes cannot tolerate the estimated settlements of 19 mm to 160 mm, as indicated in Section 6.

If the backfill for the service trenches will consist of granular fill, clay seals should be installed in the service trenches at select intervals as per City of Ottawa Drawing No. S8. The seals should be 1 m wide, extend over the entire trench width and from the bottom of the trench to the underside of the pavement structure. The clay should be compacted to 95 percent SPMDD. The purpose of the clay seals is to prevent the permanent lowering of the groundwater level.

If structural slab supported on piles is used, the services in the interior of the building will required to be hanged from the structural slab to minimize damage due the settlement of the fill in areas to be raised in excess of the maximum allowable grade raise.

11 Excavations and De-Watering Requirements

11.1 Excavations

Excavations for construction of the proposed building and installation of any underground services at the site are expected to extend to a maximum depth of approximately 3.7 m below the existing ground surface. These excavations will terminate within sandy clay to clay and will likely be below the groundwater level.

It is anticipated that excavations may be undertaken using conventional equipment capable of removing possible debris within the existing on-site fill. All excavation work should be completed in accordance with the Occupational Health and Safety Act, Ontario, Reg. 213/91. As per OHSA, the subsurface soils are considered to be Type 3 and as such the walls of the open-cut excavations must be sloped back at 1H:1V from the bottom of the excavation. For excavations that extend below the groundwater level, the side slopes should be cut back at 2H:1V to 3H:1V from the bottom of the excavation. If space restrictions prevent open-cut excavations), the excavations may be undertaken within the confines of a prefabricated support system (trench box) and/or engineered support system designed and installed in accordance with the above noted regulation.

Excavations up to a 3.7 m depth below existing grade are not expected to experience ‘base heave’ type failure. Deeper excavations may be susceptible to ‘base heave’ type failure and EXP should be contacted to review and provide comment regarding the potential for ‘base heave’ failure.

The sandy clay to clay are susceptible to disturbance due to the movement of construction equipment, and personnel on its surface. It is therefore recommended that the excavation at the site should be undertaken by equipment that does not travel on the excavated surface, such as a gradall or mechanical shovel. It is anticipated that temporary granular roads (600 mm thick granular base) may be required to gain access to the site by construction equipment and the pile driving rig. The exposed subgrade for the temporary construction roads should be inspected by a geotechnical engineer prior to placement of the granular roads.

Prior to tendering and to minimize potential contractor claims, consideration should be given to conducting an additional test pit investigation throughout the site, to collect additional data on the quality and depth of the fill.

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

11.2 De-Watering Requirements

Seepage of the surface and subsurface water into these excavations is anticipated. However, it should be possible to collect water entering the excavations at low points and to remove it by conventional pumping techniques. In areas of high infiltration or in areas where more permeable soil layers may exist, a higher seepage rate should be anticipated. Therefore, the need of high capacity pumps to keep the excavation dry should not be ignored. Drainage of ponded surface water in low lying areas will also be required and can be accomplished by perimeter ditching and pumping from sumps.

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

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

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

12 Backfilling Requirements

The on-site soils to be excavated are anticipated to consist of topsoil (surficial and buried), fill, silty sand, sandy silt, sandy clay and clay. From a geotechnical perspective, these soils are not considered suitable for re-use as backfill material. Therefore, it is anticipated that the majority of the material required for backfilling purposes and for subgrade preparation would have to be imported and should preferably conform to the following specifications. Portions of the above noted soils may be used as fill in the landscape areas of the proposed development as noted below.

- Engineered fill under footings – OPSS 1010 Granular B Type II for the interior of the building compacted to 100 percent SPMDD.
- Underfloor fill and backfill of footing trenches, grade beams, pile caps, (building interior and exterior) – OPSS 1010 Granular B Type II placed in 300 mm thick lifts and each lift compacted to 98 percent of the SPMDD in the interior of the building and to 95 percent of the SPMDD in the exterior of the building;
- Underfloor Fill in areas where grade raise will be in excess of 1.8 m, Light Weight Fill (LWF) as per the specification described in Section 6 of this report.
- Trench backfill and subgrade fill in playground areas, areas of future portables, parking areas and access roadways – OPSS 1010 Select Subgrade Material (SSM) placed in 300 mm thick lifts and each lift compacted to 95 percent of the SPMDD; and
- Landscaped area, clean fill free of organic and deleterious material placed in 300 mm thick lifts and each lift to compacted to 92 percent of the SPMDD.

To minimize settlement of the pavement structure over services trenches, the trench backfill material within the frost zone should match the existing material along the trench walls to minimize differential frost heaving of the subgrade soil, provided this material is compactible. Otherwise, frost tapers may be required.

13 Corrosion Potential

Chemical tests limited to pH, chloride and sulphate content determinations were performed on three (3) selected soil samples. The certificate of the laboratory analysis is attached in Appendix E and the results are summarized in Table XVIII below.

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

Borehole/ Sample No.	**Soil**	**Depth (Elevation) (m)**	**pH**	**Sulphate (%)**	**Chloride (%)**
Threshold Values			**<5**	**>0.1**	**>0.04**
BH 2; SS3	Sandy Clay	1.5 – 2.1 (91.1 – 90.5)	7.47	0.0042	0.0004
BH 3; SS5	Grey Clay	3.0 – 3.6 (89.6 – 89.0)	8.01	0.0104	0.0003
BH 6; SS3	Sandy Clay	1.5 – 2.1 (90.8 – 90.2)	7.91	0.0084	0.0003

The results indicate a soil with a sulphate and chloride content of less than 0.1 percent and 0.04 percent respectively. These concentrations of sulphate and chloride in the soil would have a negligible potential of sulphate and chloride attack on subsurface concrete. The concrete should be designed in accordance with Table Nos. 3 and 6 of CSA A.23.1-14. However, the concrete should be dense, well compacted and cured.

14 Access Roads and Parking Areas

The subgrade at the site is anticipated to primarily consist of imported granular fill such as OPSS Granular B Type II and Select Subgrade Material (SSM) used to raise the grades at the site. Pavement structure thicknesses required for the access roads and parking areas set on the imported fill were computed and are shown on Table XIX. The pavement structure thicknesses are based upon an estimate of the soil properties of the imported fill and functional design life of twelve to fifteen years. The proposed functional design life represents the number of years to the first rehabilitation, assuming regular maintenance is carried out.

Table XIX: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Light Duty Parking Areas	Heavy Duty Parking Areas and Access Roads
Asphaltic Concrete (PG 58-34)	92 to 97 % MRD	65 mm – SP12.5 Cat B or HL3	40 mm – 12.5 Cat B/HL3 50 mm – 19 Cat B/HL8
Granular A Base (OPSS 1010) (crushed limestone)	100% SPMDD	150 mm	150 mm
Granular B Sub-base, Type II (OPSS 1010)	100% SPMDD	450 mm	600 mm

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

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

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

1. As part of the subgrade preparation for the areas to be paved, the proposed parking and access roadway should be stripped of topsoil and other obviously unsuitable material. The subgrade should be properly shaped, crowned, then proofrolled with a heavy vibratory roller in the full-time presence of a representative of this office. Any soft or spongy subgrade areas detected should be sub excavated and properly replaced with suitable OPSS 1010 Granular B Type II compacted to 95 percent SPMDD (ASTM D698). To prevent overstressing the clay subgrade, coarser material may be required in the lower 300 mm of the subgrade fill such as OPSS 1010 Granular B Type II or well graded blast-shattered bedrock.

2. The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved. The need for adequate drainage cannot be over-emphasized. Subdrains should be installed on both sides of the access road(s). Subdrains must be installed in the proposed parking area at low points and should be continuous between catchbasins to intercept excess surface and subsurface moisture and to prevent subgrade softening. This will ensure no water collects in the granular course, which could result in pavement failure during the spring thaw. The location and extent of sub drainage required within the paved areas should be reviewed by this office in conjunction with the proposed site grading.

3. To minimize the problems of differential movement between the pavement and catchbasins/ manhole due to frost action, the backfill around the structures should consist of free-draining granular preferably conforming to OPSS 1010 Granular B, Type II material. Weep holes should be provided in the catchbasins/manholes to facilitate drainage of any water that may accumulate in the granular fill.

4. The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted lanes, half-loads during paving, temporary construction roadways, etc., may be required, especially if construction is carried out during unfavorable weather.

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

6. Relatively weaker subgrade may develop over service trenches at subgrade level. These areas may require the use of thicker/coarser sub-base material and the use of a geotextile at the subgrade level. if this is the case, it is recommended that additional 150 mm of granular sub-base Granular B should be provided in these areas in addition to the use of a geotextile at the subgrade level. On-site excavated soils should not be used as backfill of the service trenches.

7. The granular materials used for pavement construction should conform to OPSS 1010 for Granular A and Granular B, Type II and should be compacted to 100 percent of the SPMDD (ASTM D698). The asphaltic concrete used and its placement should meet OPSS requirements. It should be compacted to 92 to 97 percent of the maximum relative density in accordance with ASTM D2041.

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

15 Tree Planting Restrictions

The modified plasticity index of the upper sandy clay and underlying brown and grey clay was calculated and determined to be less than 40 percent. Based on a review of the City of Ottawa document titled, "Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines", soils that exhibit a modified plasticity index less than 40 percent are considered to have a low to medium potential for soil volume change.

For low to medium potential volume change soil types, the tree planting restrictions and setbacks from structures should follow the above noted 2017 guidelines.

A landscape architect should be consulted to ensure the applicable tree planting restrictions and setbacks for the development of this site are in accordance with the applicable City of Ottawa guideline and policy.

16 General Comments

For the pile foundation option, it is recommended that additional deep boreholes be undertaken within the footprint of the proposed school building to provide geographic coverage of the building footprint to better delineate the subsurface soil, rock and groundwater conditions and to confirm the geotechnical design parameters presented in this report.

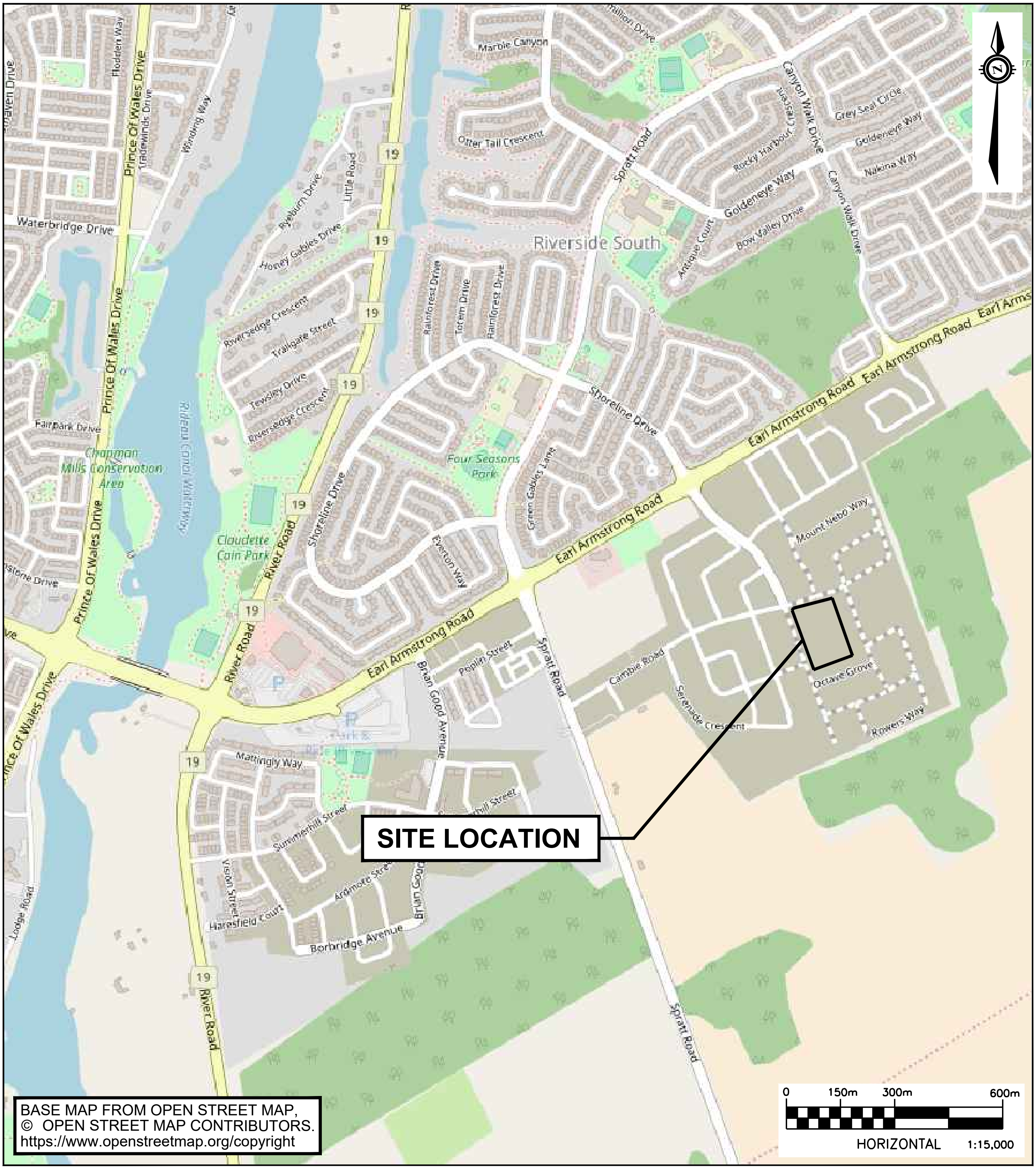
Prior to tendering and to minimize potential contractor claims, consideration should be given to conducting an additional test pit investigation throughout the site, to collect additional data on the quality and depth of the fill.

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 1:15 000

date APRIL 2018

drawn by J.R.

PROJECT:
CONSEIL DES ÉCOLES CATHOLIQUES
DU CENTRE-EST
PROPOSED NEW RIVERSIDE SOUTH CATHOLIC ELEMENTARY SCHOOL

TITLE:
SITE LOCATION PLAN

project no. OTT-00245869-A0

design

checked

FIG 1

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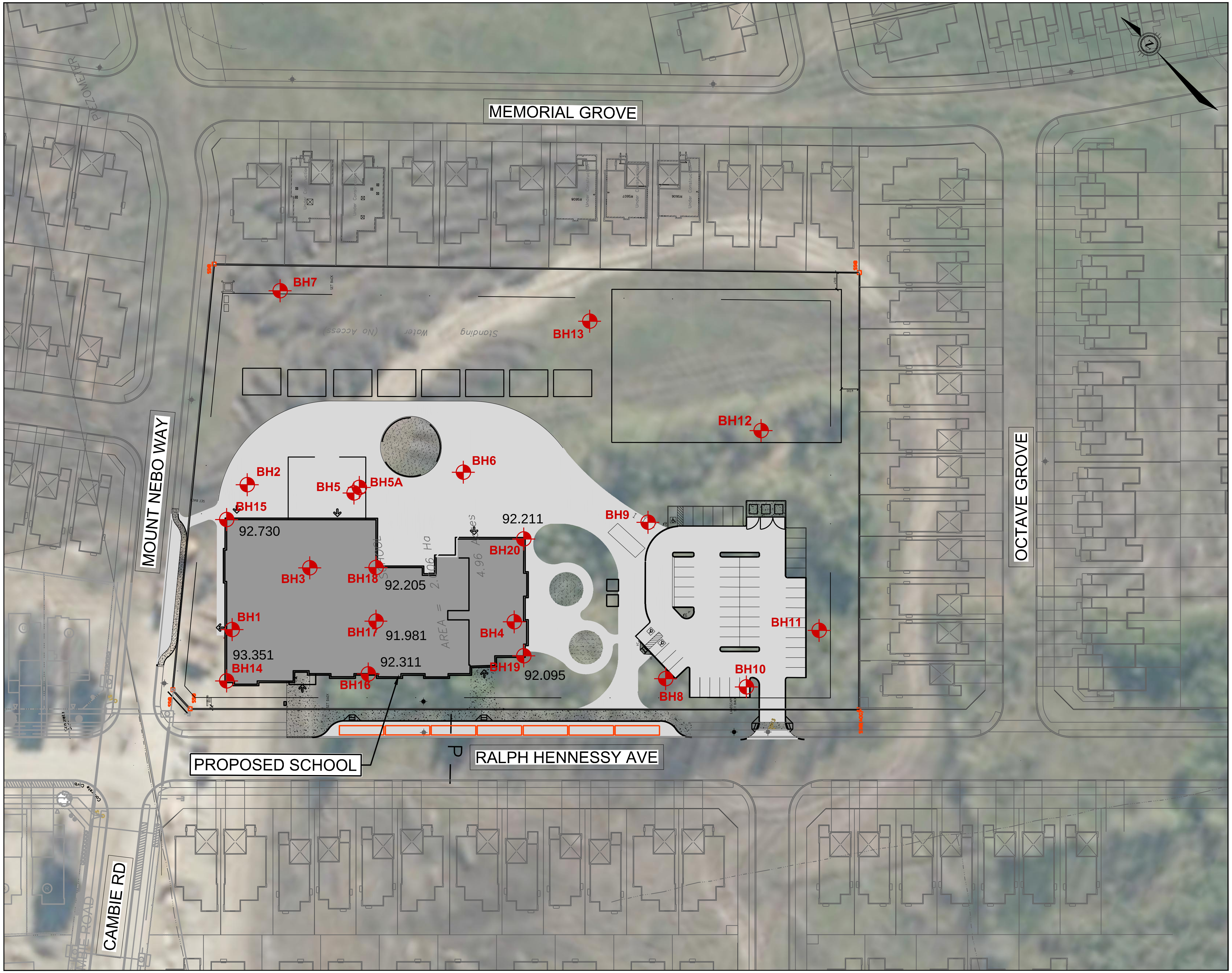

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NOTES :

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.

THE BACKGROUND IMAGE CONTAINS INFORMATION LICENSED UNDER THE OPEN GOVERNMENT LICENSE - CITY OF OTTAWA. SCHOOL SITE PLAN AND SURROUNDING DEVELOPMENT PLAN FROM PYE & RICHARDS ARCHITECTS, INC.

LEGEND

⊕ BH12   BOREHOLE LOCATION AND NUMBER

SCALE
0   10m   20m   40m
HORIZONTAL   1:1000

CLIENT
CONSEIL DES
ÉCOLES CATHOLIQUES
DU CENTRE-EST

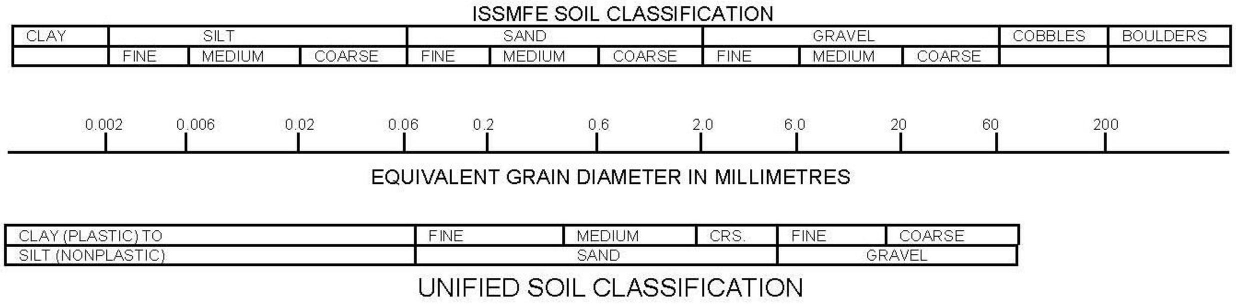
PROJECT
PROPOSED NEW RIVERSIDE SOUTH
CATHOLIC ELEMENTARY SCHOOL

TITLE
BOREHOLE LOCATION PLAN

date APRIL 2018	project no. OTT-00245869-A0
design by I.T.	drawing no.
prepared by J.R.	FIG 2
reviewed by I.T.	

Notes On Sample Descriptions

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



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

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

Log of Borehole BH-1

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 3

Page. 1 of 2

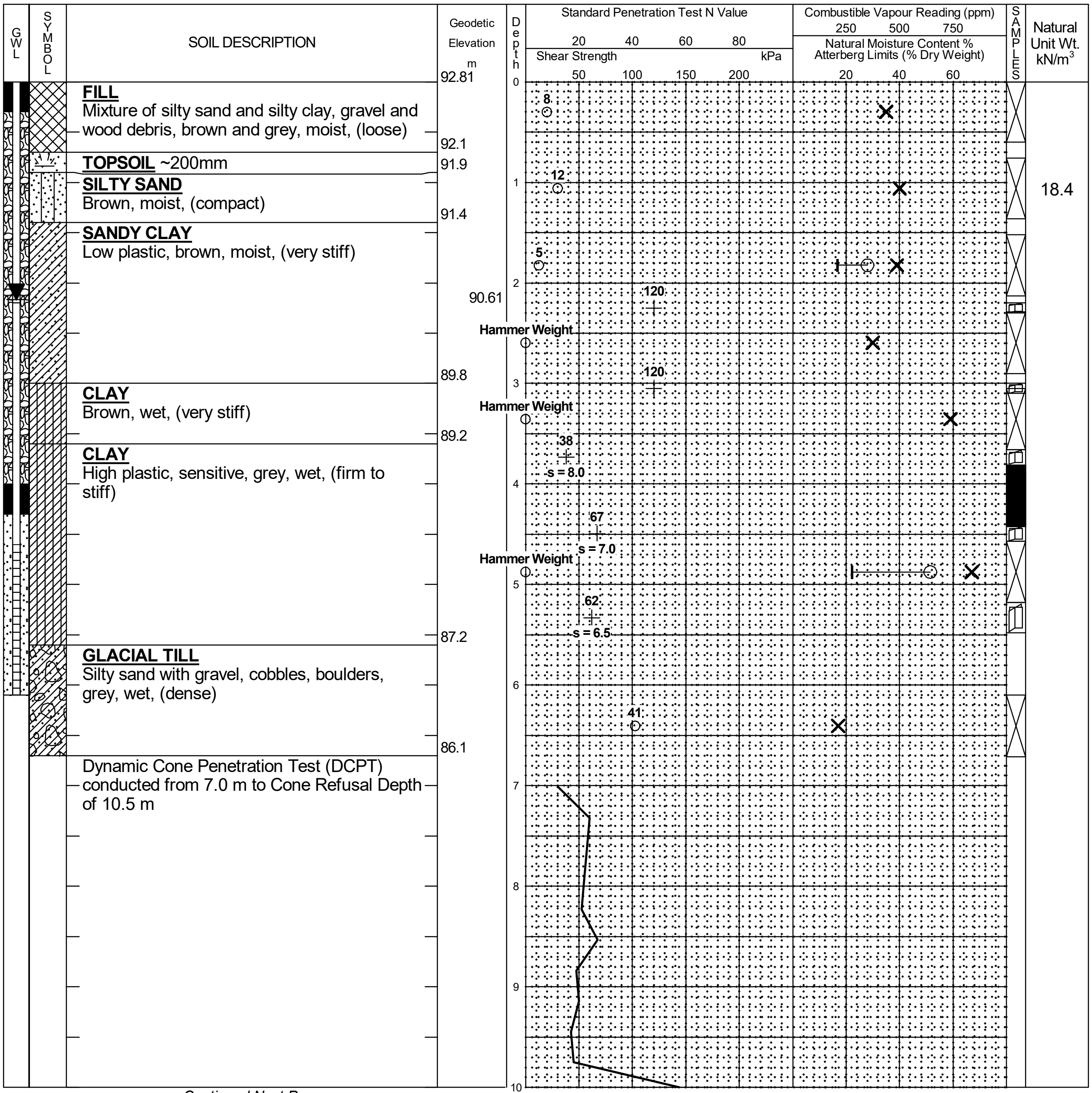
Date Drilled: 'April 2, 2018

Drill Type: CME-75 Track Mount 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		



FILL
Mixture of silty sand and silty clay, gravel and wood debris, brown and grey, moist, (loose)

TOPSOIL ~200mm

SILTY SAND
Brown, moist, (compact)

SANDY CLAY
Low plastic, brown, moist, (very stiff)

CLAY
Brown, wet, (very stiff)

CLAY
High plastic, sensitive, grey, wet, (firm to stiff)

GLACIAL TILL
Silty sand with gravel, cobbles, boulders, grey, wet, (dense)

Dynamic Cone Penetration Test (DCPT) conducted from 7.0 m to Cone Refusal Depth of 10.5 m

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe piezometer installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	2.7	6.1
9 days	2.3	
35 days	2.2	

CORE DRILLING RECORD

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

Log of Borehole BH-1

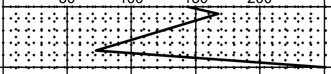
Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Figure No. 3

Page. 2 of 2

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/m^3
			82.81 82.3	10				
		Cone Refusal at 10.5 m Depth						



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe piezometer installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	2.7	6.1
9 days	2.3	
35 days	2.2	

CORE DRILLING RECORD

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

Log of Borehole BH-2

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 4

Page. 1 of 2

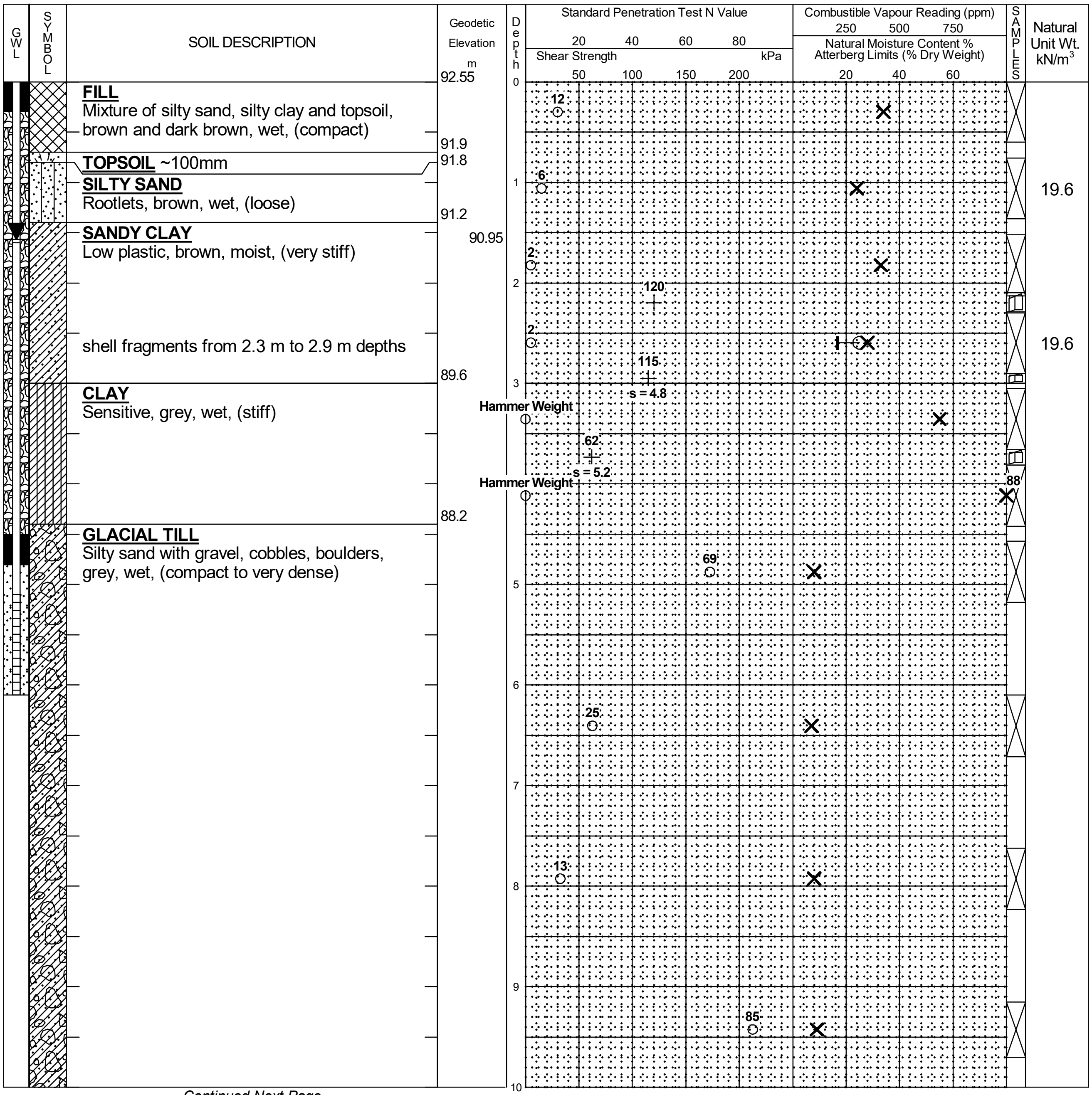
Date Drilled: 'April 3, 2018

Drill Type: CME-75 Track Mount 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲



FILL
Mixture of silty sand, silty clay and topsoil, brown and dark brown, wet, (compact)

TOPSOIL ~100mm

SILTY SAND
Rootlets, brown, wet, (loose)

SANDY CLAY
Low plastic, brown, moist, (very stiff)

shell fragments from 2.3 m to 2.9 m depths

CLAY
Sensitive, grey, wet, (stiff)

GLACIAL TILL
Silty sand with gravel, cobbles, boulders, grey, wet, (compact to very dense)

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe piezometer installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.0	10.1
8 days	1.6	
34 days	1.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	10.1 - 10.4	100	33
2	10.4 - 12	100	97

Log of Borehole BH-2

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Figure No. 4

Page. 2 of 2

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
			82.55	10				
		LIMESTONE BEDROCK Horizontal and vertical fractures, grey, (poor and exellent quality)	82.5					Run 1
				11				Run 2
			80.6	12				
		Borehole Terminated at 12.0 m Depth						

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe piezometer installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.0	10.1
8 days	1.6	
34 days	1.6	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	10.1 - 10.4	100	33
2	10.4 - 12	100	97

Log of Borehole BH-3

Project No: OTT-00245869-A0

Figure No. 5

Page. 1 of 1

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

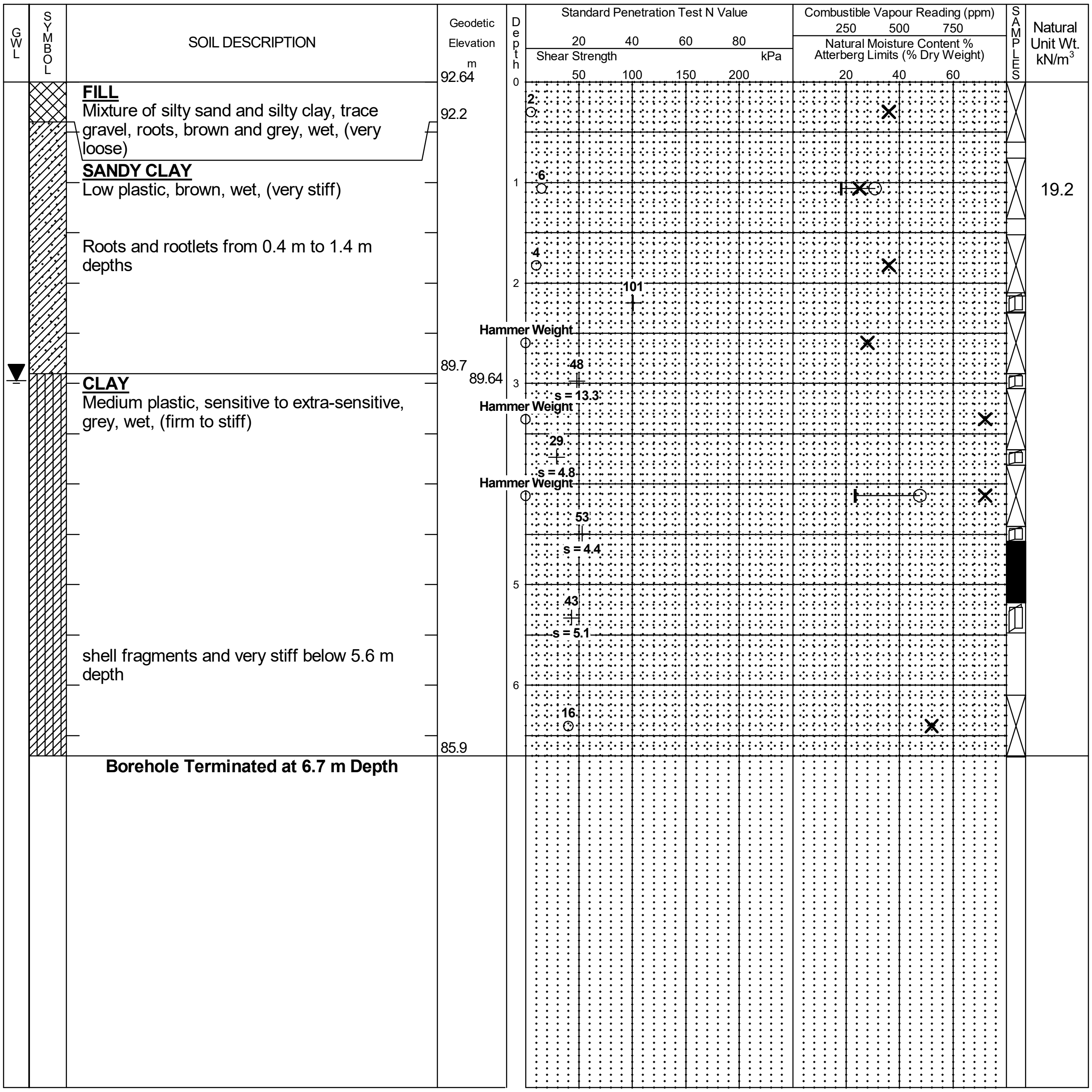
Date Drilled: 'April 2, 2018

Drill Type: CME-75 Track Mount 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		



FILL
Mixture of silty sand and silty clay, trace gravel, roots, brown and grey, wet, (very loose)

SANDY CLAY
Low plastic, brown, wet, (very stiff)

Roots and rootlets from 0.4 m to 1.4 m depths

CLAY
Medium plastic, sensitive to extra-sensitive, grey, wet, (firm to stiff)

shell fragments and very stiff below 5.6 m depth

Borehole Terminated at 6.7 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.0	6.1

CORE DRILLING RECORD

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

Log of Borehole BH-4

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 6

Page. 1 of 2

Date Drilled: 'April 3, 2018

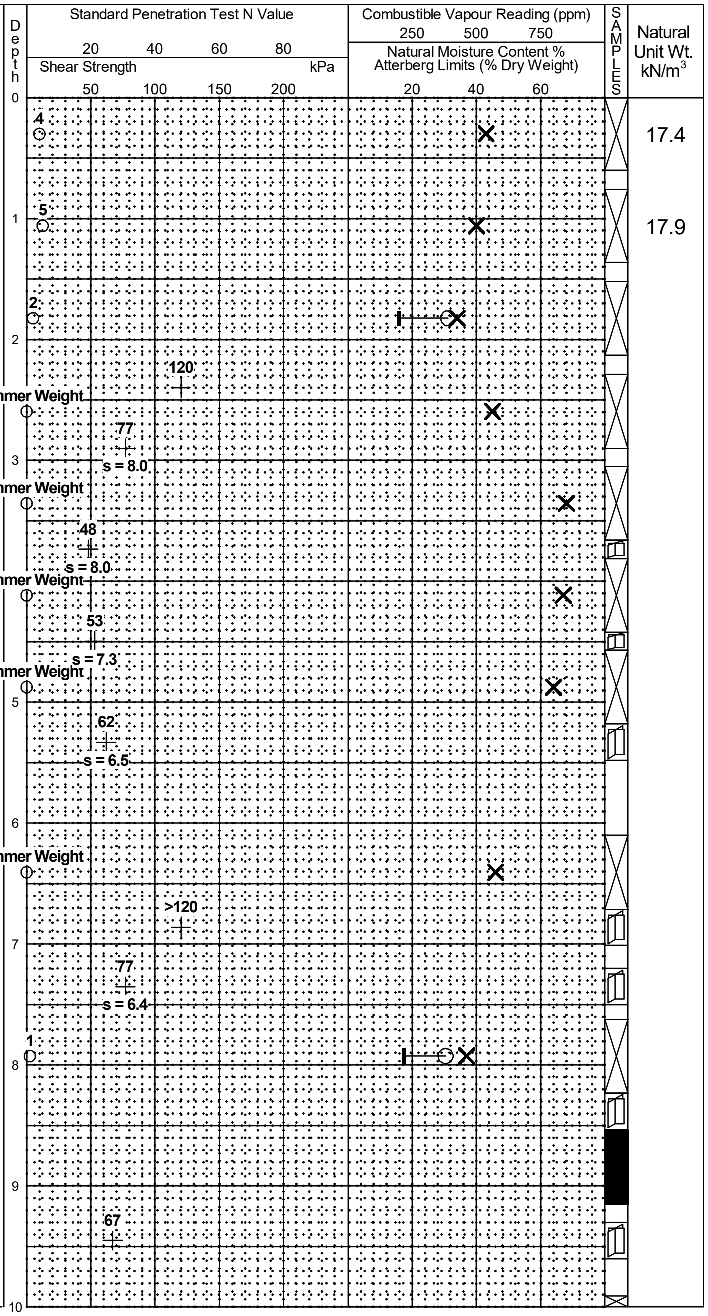
Drill Type: CME-75 Track Mount Drill Rig

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
			92.2
		TOPSOIL ~100mm	92.1
		SILTY SAND Brown, moist, (loose)	
		SANDY CLAY Medium plastic, brown, moist, (soft)	90.8
		CLAY Brown, wet, (stiff to very stiff)	90.0
		CLAY Grey, wet, (firm to stiff)	89.2
		SILTY CLAY Silt seams, low plastic, sensitive, grey, wet, (stiff to very stiff)	86.5
			82.4



Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	11.3 - 11.7	100	31
2	11.7 - 13.1	64	34
3	13.1 - 14.4	100	61

Log of Borehole BH-4

Project No: OTT-00245869-A0

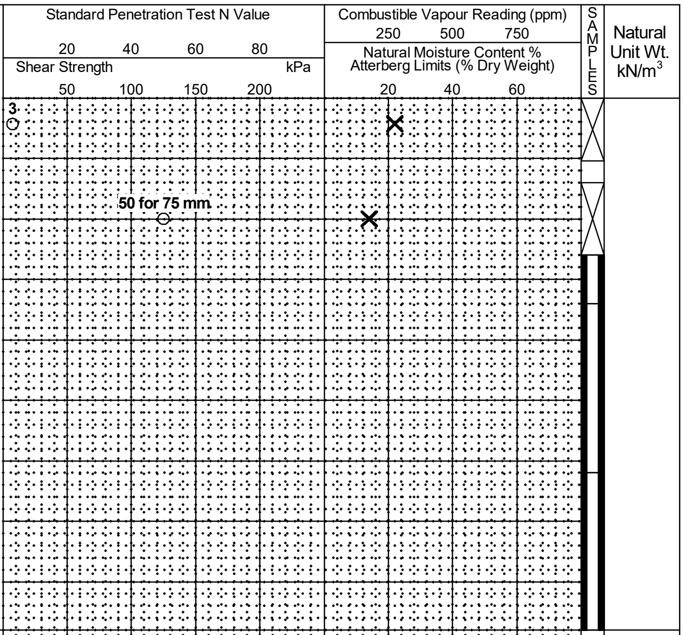
Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Figure No. 6

Page. 2 of 2

SOIL DESCRIPTION	Geodetic Elevation m	Depth
	82.2	10
SANDY CLAY Grey, wet, (soft) *(continued)*		
	81.7	
GLACIAL TILL Silty sand with gravel, cobbles and boulders, grey, wet		11
	80.9	
LIMESTONE BEDROCK Horizontal and vertical fractures, (poor and fair quality)		12 13 14
	77.8	

Borehole Terminated at 14.4 m Depth



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	11.3 - 11.7	100	31
2	11.7 - 13.1	64	34
3	13.1 - 14.4	100	61

Log of Borehole BH-5

Project No: OTT-00245869-A0

Figure No. 7

Page. 1 of 1

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

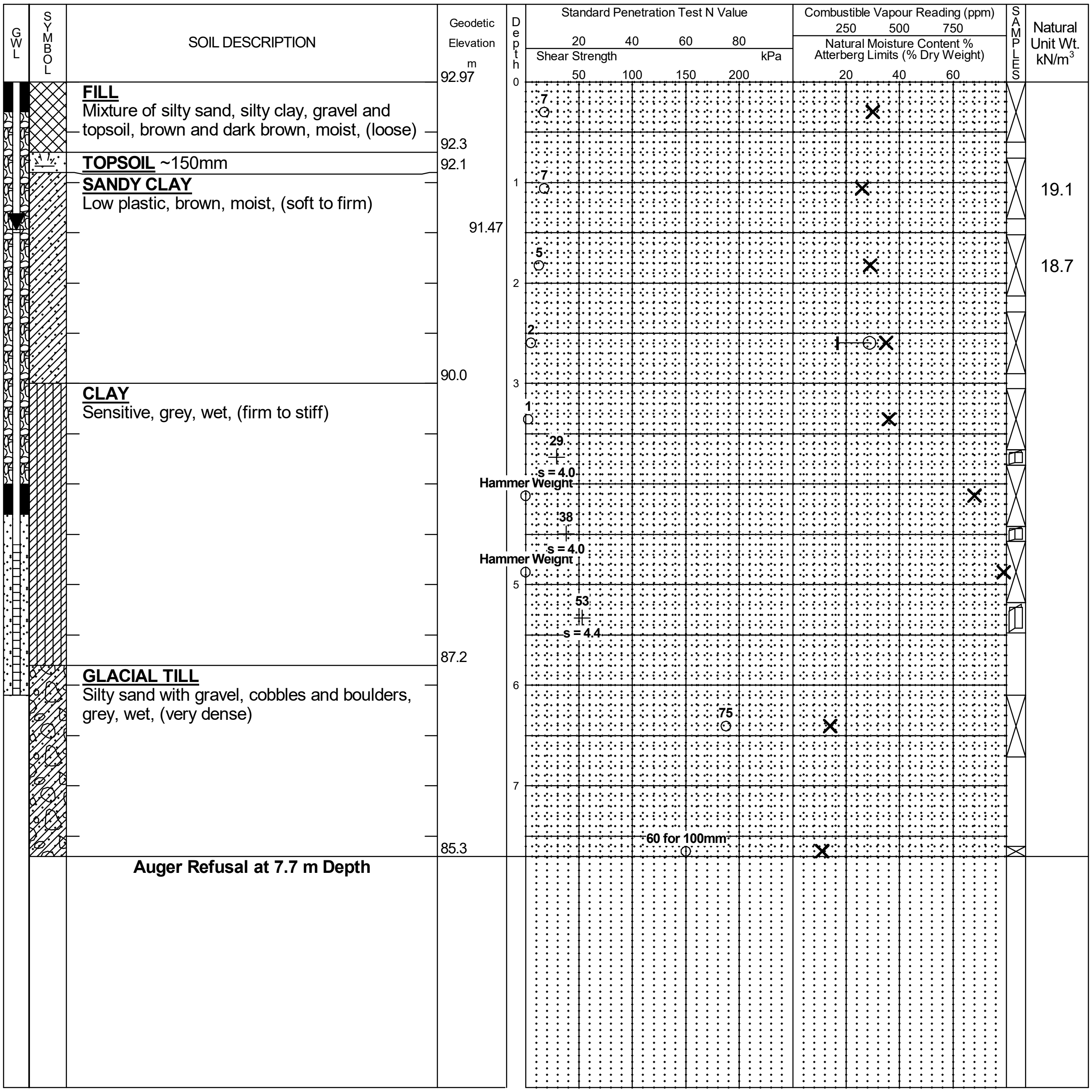
Date Drilled: 'April 2, 2018

Drill Type: CME-75 Track Mount 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲



FILL
Mixture of silty sand, silty clay, gravel and topsoil, brown and dark brown, moist, (loose)

TOPSOIL ~150mm

SANDY CLAY
Low plastic, brown, moist, (soft to firm)

CLAY
Sensitive, grey, wet, (firm to stiff)

GLACIAL TILL
Silty sand with gravel, cobbles and boulders, grey, wet, (very dense)

Auger Refusal at 7.7 m Depth

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe piezometer installed in borehole as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.3	7.6
9 days	1.5	
35 days	1.5	

CORE DRILLING RECORD

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

Log of Borehole BH-5A

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 8

Page. 1 of 1

Date Drilled: 'April 4, 2018

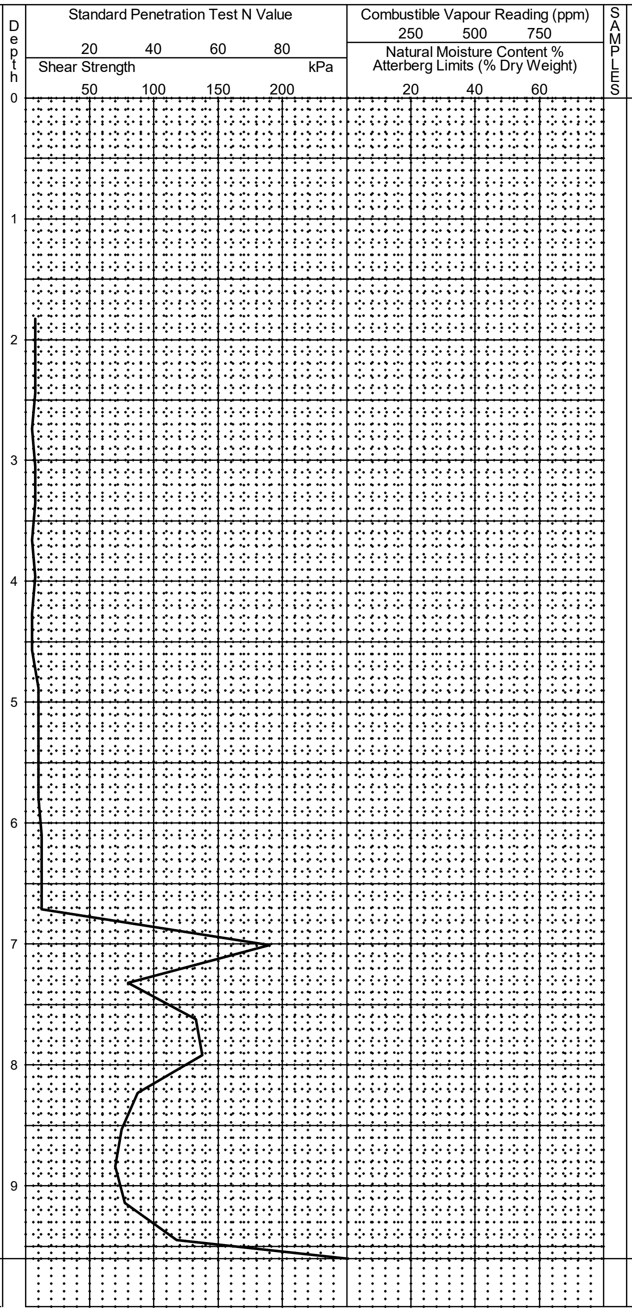
Drill Type: CME-75 Track Mount Drill Rig

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
			92.97	0
	▨	Power Augered to a 1.5 m Depth		
			91.5	
		Dynamic Cone Penetration Test (DCPT) conducted from 1.8 m to Cone Refusal Depth of 9.6 m		
			83.4	
		Cone Refusal at 9.6 m Depth		



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

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

LOG OF BOREHOLE BH LOGS - 245869 - RIVERSIDE SOUTH SCHOOL.GPJ TROW OTTAWA.GDT 7/24/18

Log of Borehole BH-6

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 9

Page. 1 of 1

Date Drilled: 'April 2, 2018

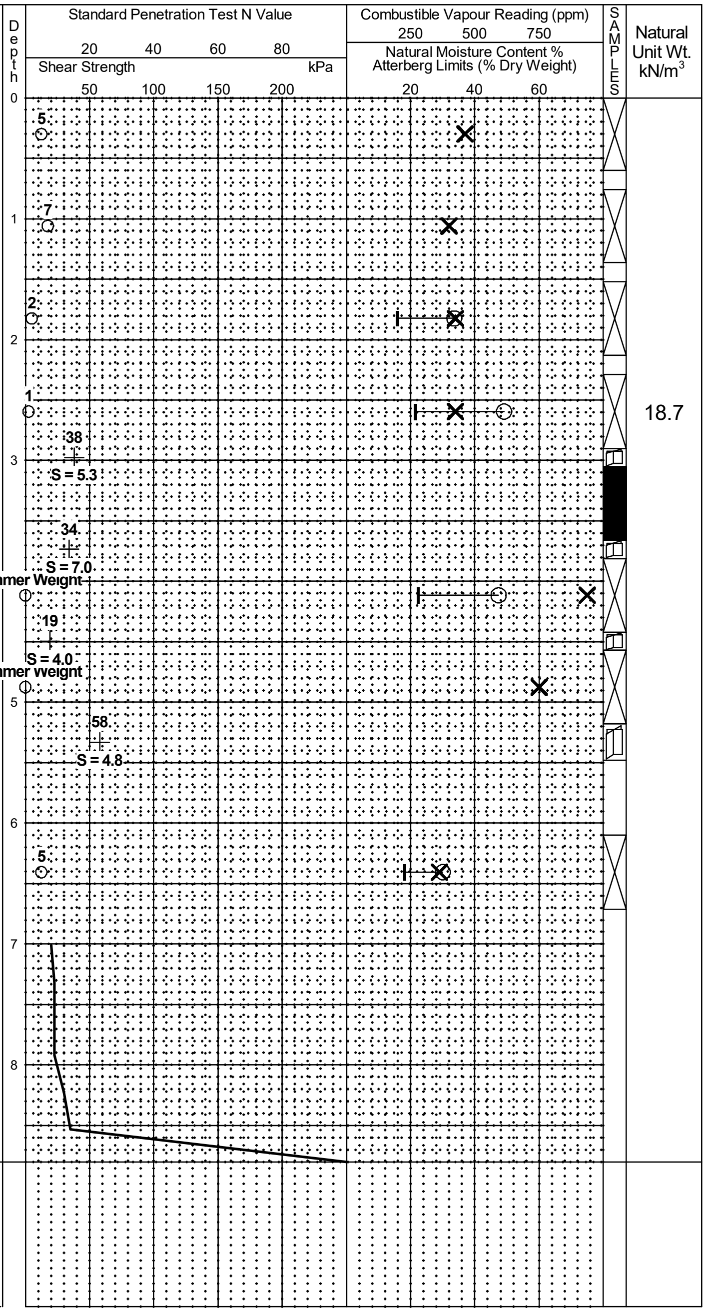
Drill Type: CME-75 Track Mount Drill Rig

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		

SOIL DESCRIPTION	Geodetic Elevation m
	92.32
FILL Mixture of silty sand and silty clay, roots, brown and dark brown, moist, (loose)	92.02
SANDY CLAY Medium plastic, brown, moist, (soft to firm)	91.6
CLAY Medium plastic, brown, moist, (very soft)	90.1
CLAY Medium plastic, sensitive, grey, wet, (soft to stiff)	89.4
SILTY CLAY Low plastic, grey, wet, (firm)	86.2
Dynamic Cone Penetration Test (DCPT) conducted from 7.0 m to Cone Refusal Depth of 8.8 m	85.6
Cone Refusal at 8.8 m Depth	83.5



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50 mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	2.4	6.1
9 days	0.0	
35 days	0.3	

CORE DRILLING RECORD

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

Log of Borehole BH-7

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 10

Page. 1 of 1

Date Drilled: 'April 5 2018

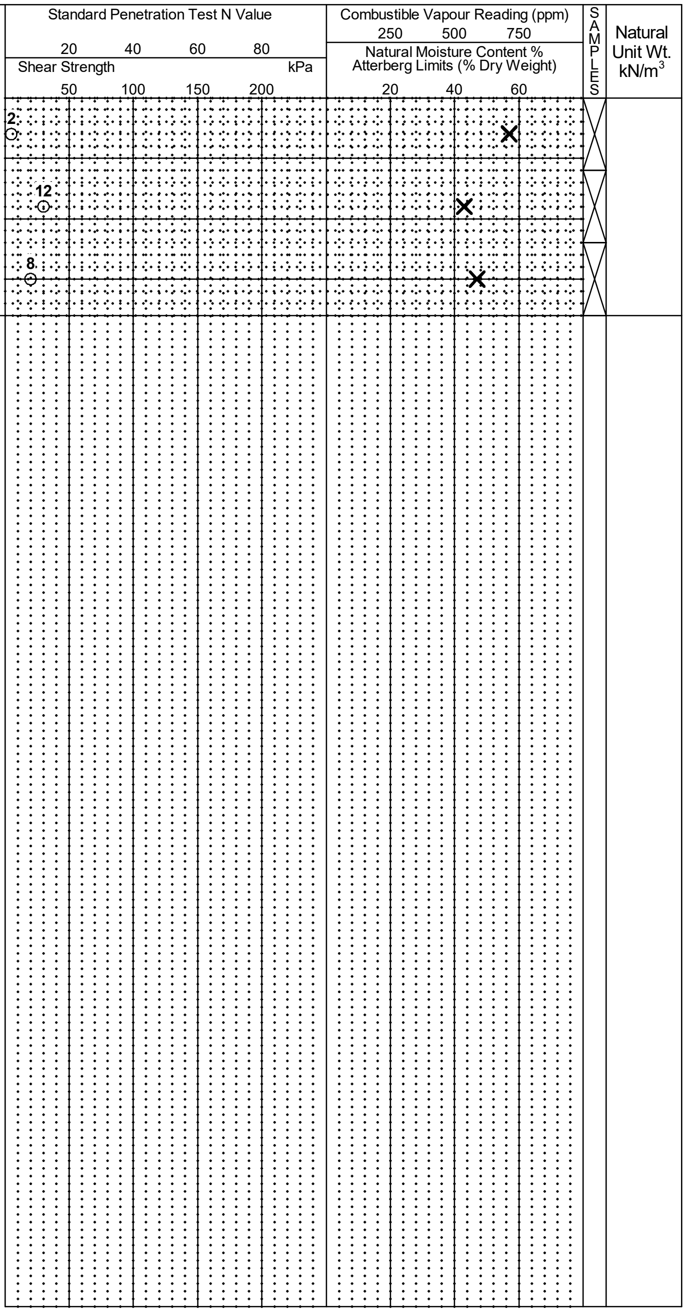
Drill Type: CME-75 Track Mount Drill Rig

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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
			0	0
	▨	**FILL** Mixture of sand, gravel and clay, brown, wet, (very loose)	-0.6	
		TOPSOIL ~200mm	-0.8	
	▨	**SANDY CLAY (possible fill)** Brown, wet, (stiff)	-1.8	1
		Borehole Terminated at 1.8 m Depth		



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	dry	

CORE DRILLING RECORD

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

LOG OF BOREHOLE BH LOGS - 245869 - RIVERSIDE SOUTH SCHOOL.GPJ TROW OTTAWA.GDT 7/24/18

Log of Borehole BH-8

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 11

Page. 1 of 1

Date Drilled: 'April 4, 2018

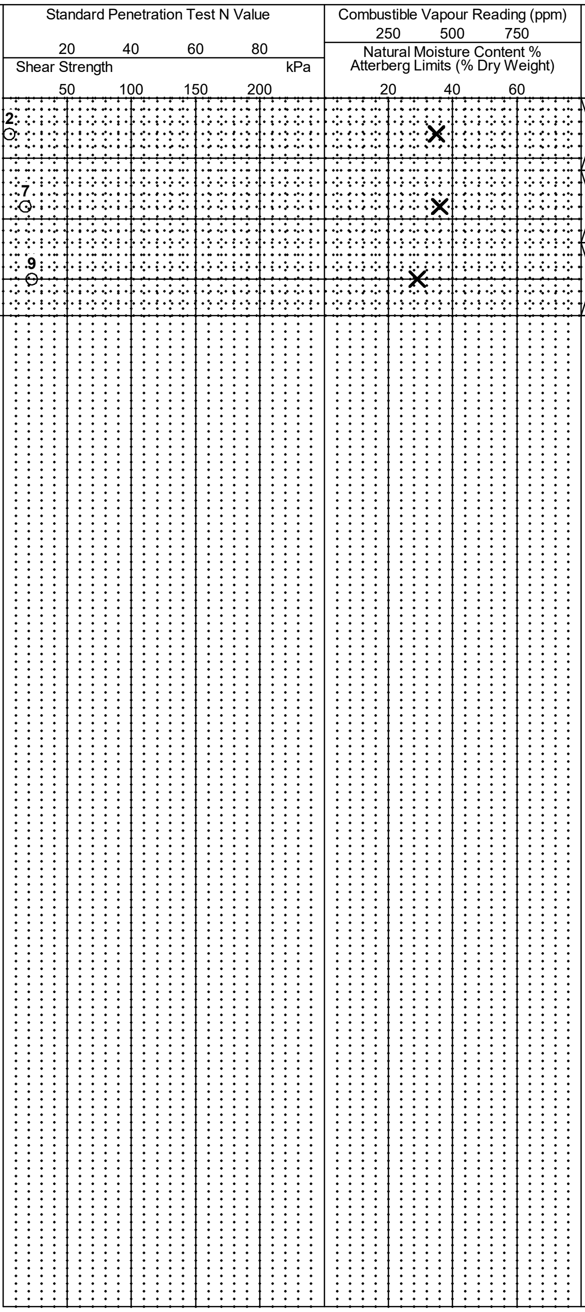
Drill Type: CME-75 Track Mount Drill Rig

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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	SAMPLES	Natural Unit Wt. kN/m³
	▩	**FILL** Mixture of silty sand and silty clay, brown, moist, (very loose)	92.13 91.5	0	☒	
		TOPSOIL ~150mm	91.3			
	▨	**SANDY CLAY** Brown, moist to wet, (firm to stiff)	90.3	1	☒ ☒	19.0 19.3
		Borehole Terminated at 1.8 m Depth				



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	dry	

CORE DRILLING RECORD

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

Log of Borehole BH-9

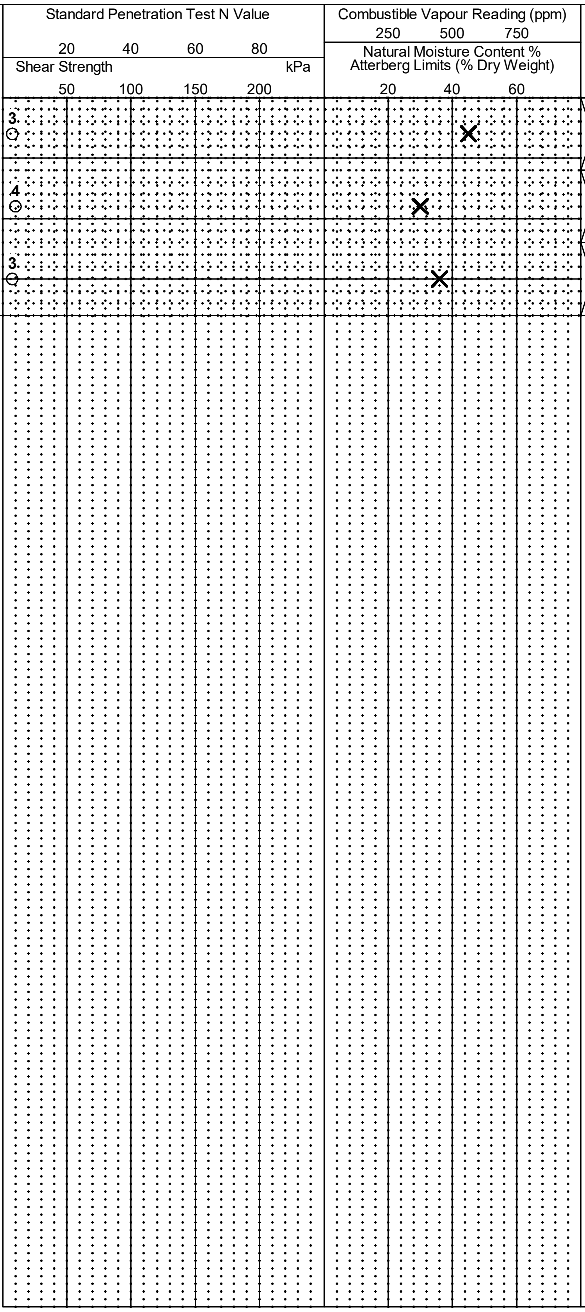
Project No: OTT-00245869-A0
Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School
Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 12
Page. 1 of 1

Date Drilled: 'April 4, 2018
Drill Type: CME-75 Track Mount 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	SAMPLES	Natural Unit Wt. kN/m³
		TOPSOIL ~400mm	92.4	0	☒	
		SANDY CLAY Brown, wet, (soft)	92.0	1	☒ ☒	18.6
			90.6			
		Borehole Terminated at 1.8 m Depth				



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	dry	

CORE DRILLING RECORD

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

Log of Borehole BH-10

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 13

Page. 1 of 1

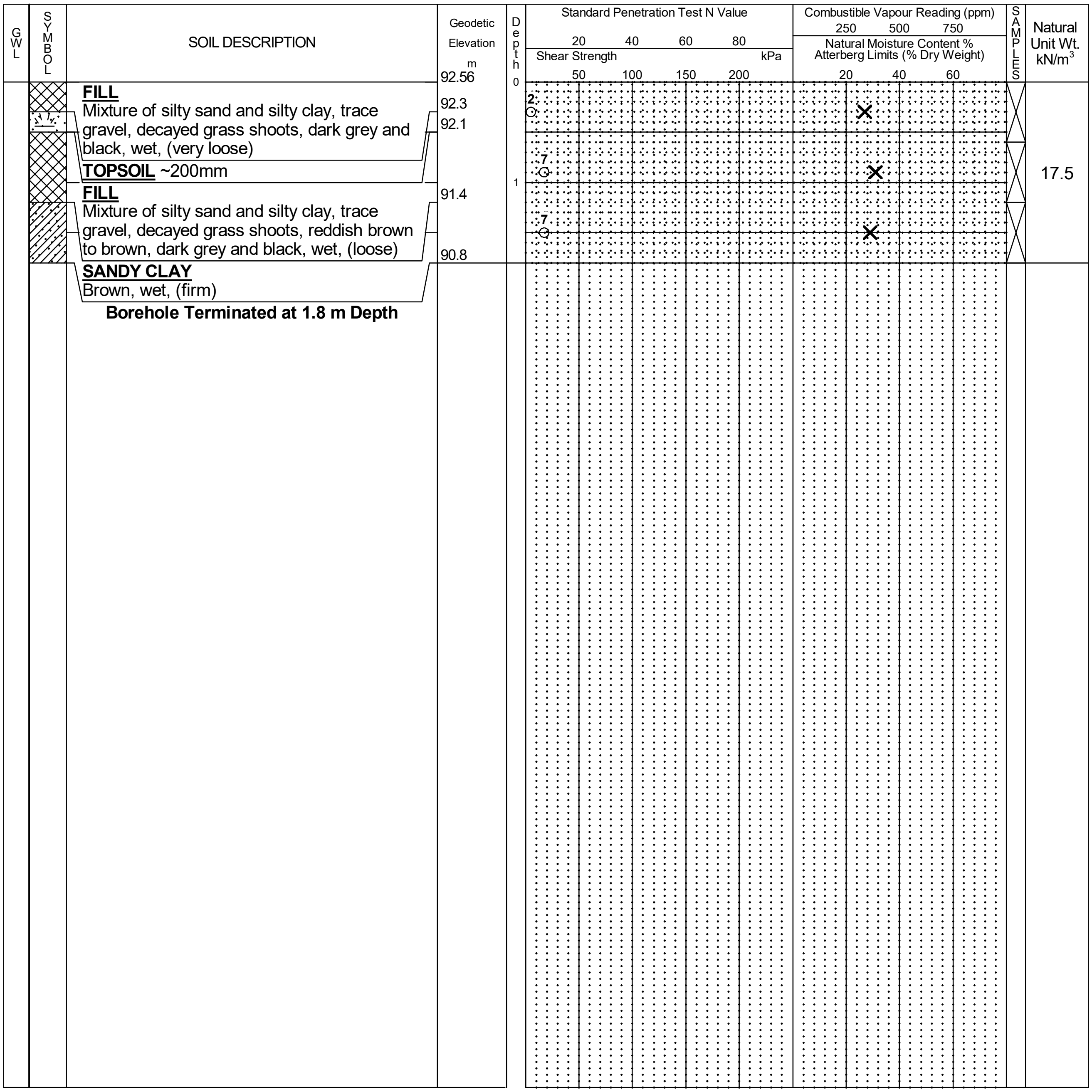
Date Drilled: 'April 4, 2018

Drill Type: CME-75 Track Mount 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		



FILL
Mixture of silty sand and silty clay, trace gravel, decayed grass shoots, dark grey and black, wet, (very loose)

TOPSOIL ~200mm

FILL
Mixture of silty sand and silty clay, trace gravel, decayed grass shoots, reddish brown to brown, dark grey and black, wet, (loose)

SANDY CLAY
Brown, wet, (firm)

Borehole Terminated at 1.8 m Depth

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	dry	

CORE DRILLING RECORD

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

Log of Borehole BH-11

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 14

Page. 1 of 1

Date Drilled: 'April 4, 2018

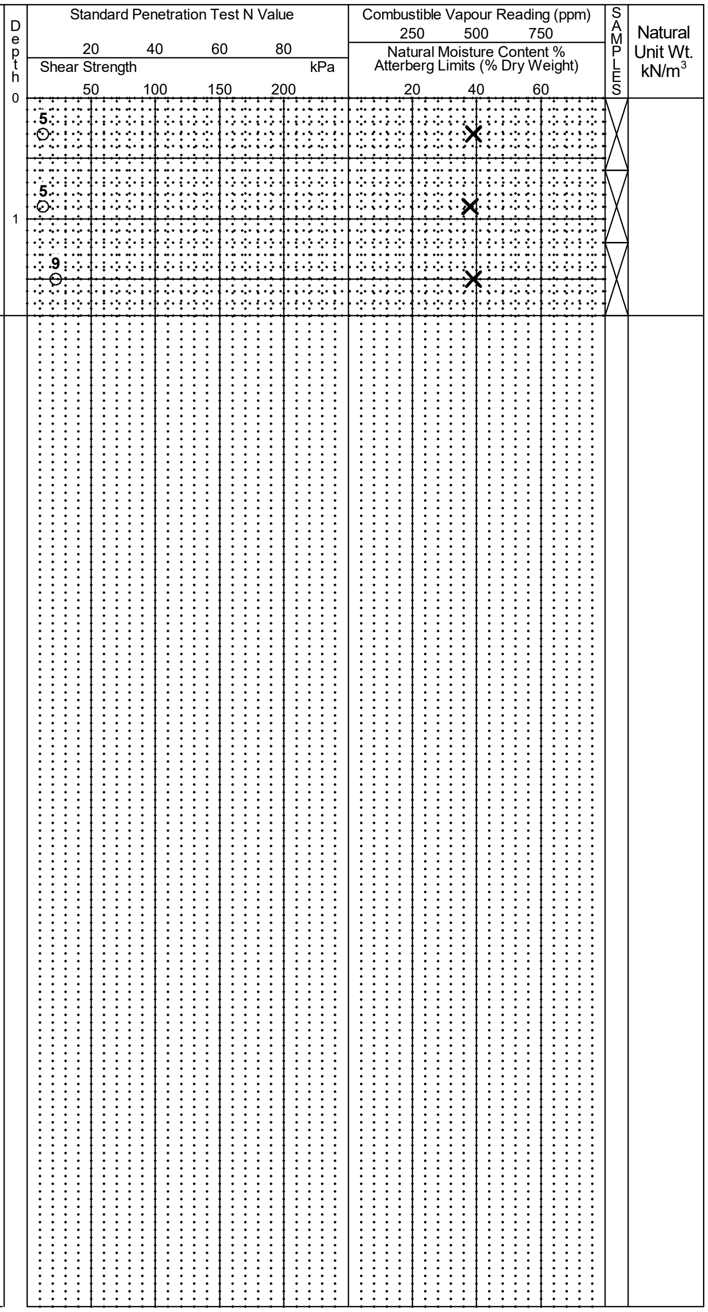
Drill Type: CME-75 Track Mount Drill Rig

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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m
			92.48
		TOPSOIL ~400mm	92.1
		FILL Mixture of sandy clay and topsoil, decayed grass shoots, reddish brown to brown to dark grey to black, wet, (firm to stiff)	90.7
		Borehole Terminated at 1.8 m Depth	



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	dry	

CORE DRILLING RECORD

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

Log of Borehole BH-12

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 15

Page. 1 of 1

Date Drilled: 'April 4, 2018

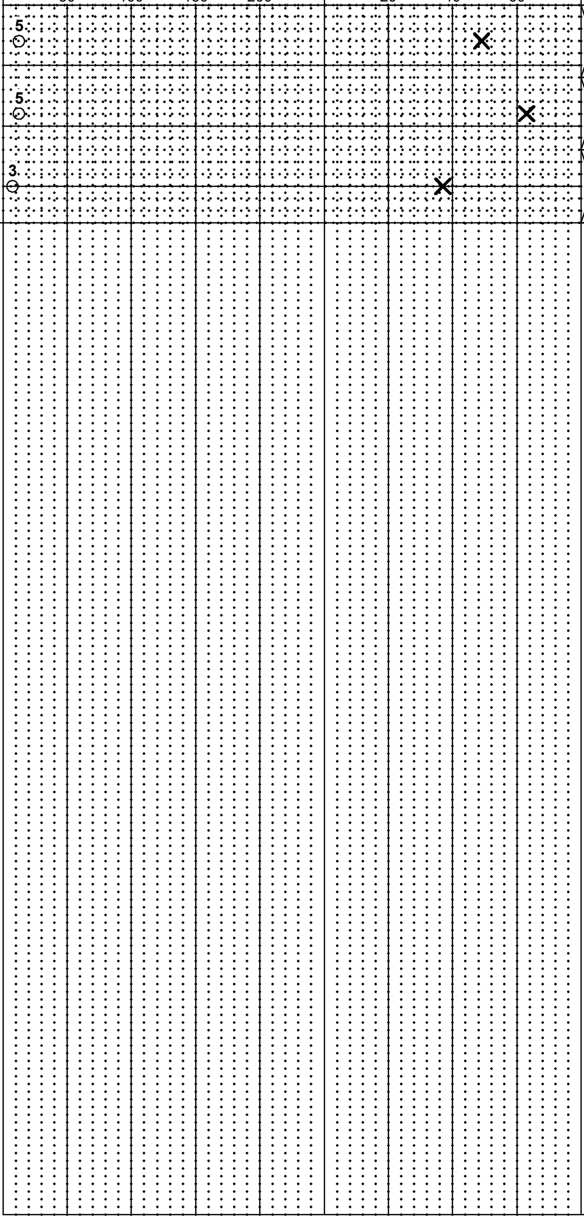
Drill Type: CME-75 Track Mount Drill Rig

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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

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
			92.53	0				
		TOPSOIL ~300mm	92.2				☒	17.7
		SANDY CLAY Brown, wet, (soft to firm) roots and rootlets from 0.3 m to 1.2 m depths		1			☒	17.8
			90.7				☒	
		Borehole Terminated at 1.8 m Depth						



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	dry	

CORE DRILLING RECORD

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

Log of Borehole BH-13

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 16

Page. 1 of 1

Date Drilled: 'April 5, 2018

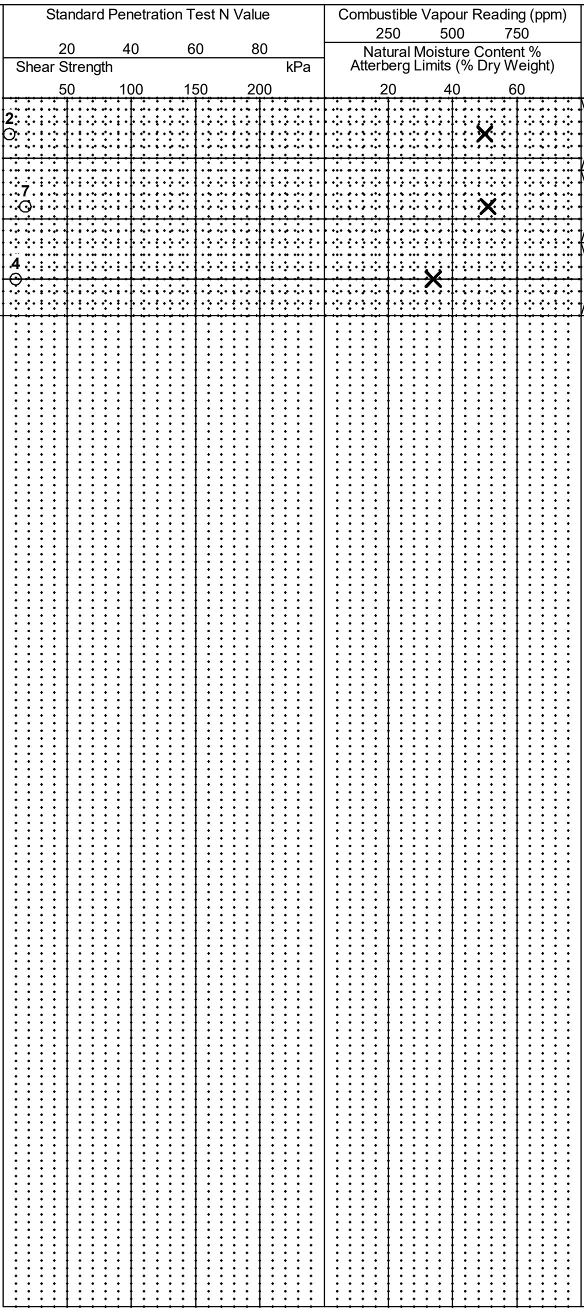
Drill Type: CME-75 Track Mount Drill Rig

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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	SAMPLES	Natural Unit Wt. kN/m^3
	▩	**FILL** Mixture of sand, gravel and clay, brown, wet, (very loose)	92.77 92.2	0	☒	
		TOPSOIL ~250mm	91.9		☒	
	▨	**SILTY CLAYEY SAND** Brown, wet, (loose/firm)	91.0	1	☒	
		Borehole Terminated at 1.8 m Depth				



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	dry	

CORE DRILLING RECORD

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

LOG OF BOREHOLE BH LOGS - 245869 - RIVERSIDE SOUTH SCHOOL.GPJ TROW OTTAWA.GDT 7/24/18

Log of Borehole BH-14

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 17

Page. 1 of 1

Date Drilled: 'July 6, 2018

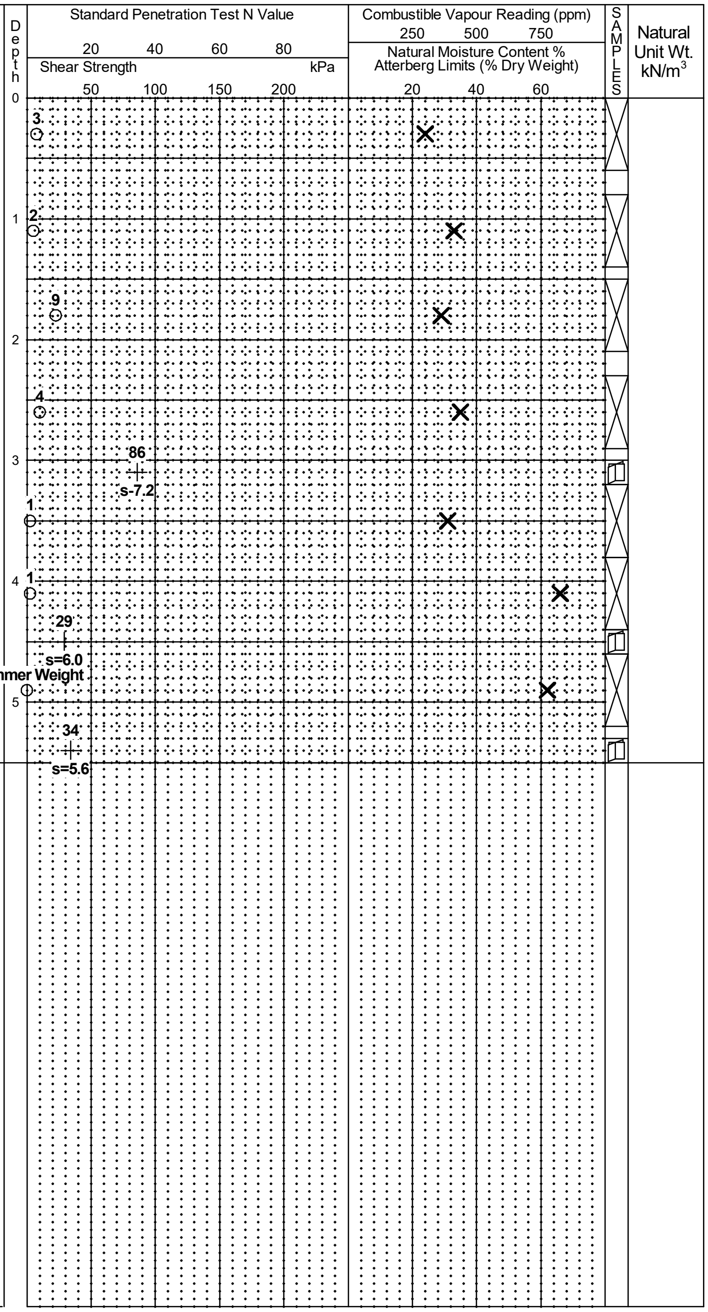
Drill Type: CME-75 Track Mount Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
		FILL Mixture of silty sand and silty clay, gravel, brown and grey, moist, (very loose to loose) dark brown to black organic type stains below 1.5 m depth	93.35	0
		SANDY CLAY Grey sand seams, sensitive, brown, moist, (stiff)	91.2	
▼			90.35	3
		CLAY Sensitive, grey, wet, (firm)	89.7	
		Borehole Terminated at 5.5 m Depth	87.9	



LOG OF BOREHOLE BH LOGS - 245869 - RIVERSIDE SOUTH SCHOOL.GPJ TROW OTTAWA.GDT 7/24/18

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.0	4.6

CORE DRILLING RECORD

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

Log of Borehole BH-15

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 18

Page. 1 of 1

Date Drilled: 'July 6, 2018

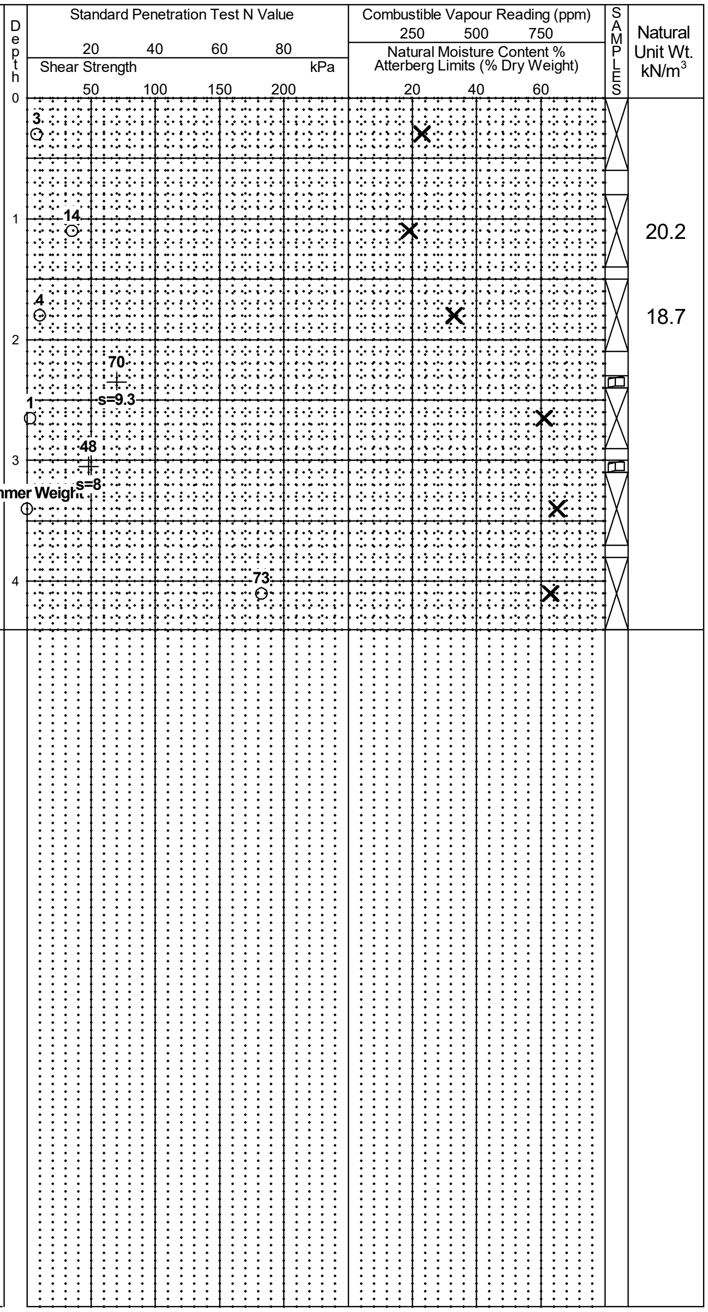
Drill Type: CME-75 Track Mount Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

Geodetic Elevation m	SOIL DESCRIPTION
92.73	**FILL** Mixture of silty sand, silty clay and topsoil, wood debris, brown and dark brown, moist (very loose to compact)
91.2	**SANDY CLAY** Grey sand seams, brown, moist, (soft)
90.4	**CLAY** Extra-sensitive, grey, wet, (firm to stiff)
89.0	**GLACIAL TILL** Silty sand with gravel, cobbles, boulders, grey, wet, (very dense)
88.3	**Borehole Terminated at 4.4 m Depth**



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.2	3.6

CORE DRILLING RECORD

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

Log of Borehole BH-16

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 19

Page. 1 of 1

Date Drilled: 'July 6, 2018

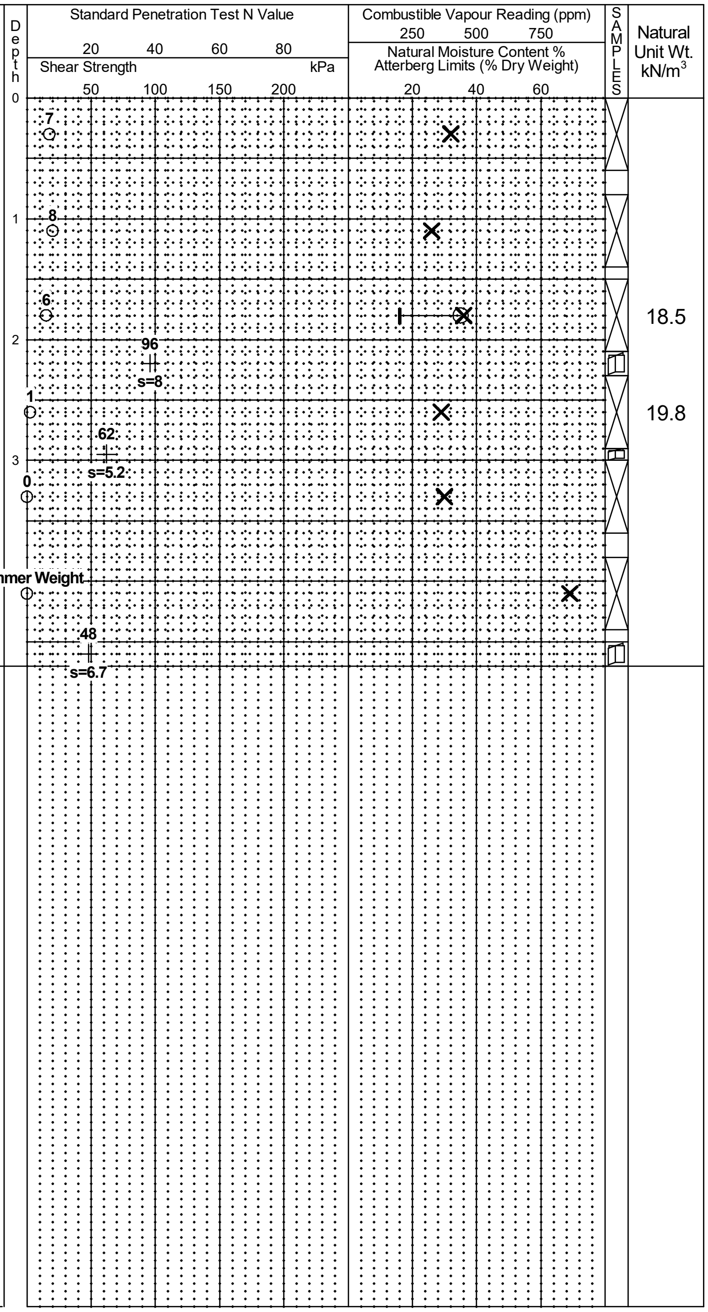
Drill Type: CME-75 Track Mount Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SOIL DESCRIPTION	Geodetic Elevation m
		92.31
	TOPSOIL ~ 150 mm	92.1
	FILL Mixture of silty sand and silty clay, rootlets, decayed grass shoots, brown and grey, moist, (loose)	91.6
	SANDY CLAY Grey sand seams, low to medium plasticity, extra-sensitive, brown, wet, (stiff)	89.7
▼	**CLAY** Sensitive, grey, wet, (firm to stiff)	89.41
		87.6
	Borehole Terminated at 4.7 m Depth	



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	2.9	3.6

CORE DRILLING RECORD

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

LOG OF BOREHOLE BH LOGS - 245869 - RIVERSIDE SOUTH SCHOOL.GPJ TROW OTTAWA.GDT 7/24/18

Log of Borehole BH-17

Project No: OTT-00245869-A0

Figure No. 20

Page. 1 of 1

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Date Drilled: 'July 6, 2018

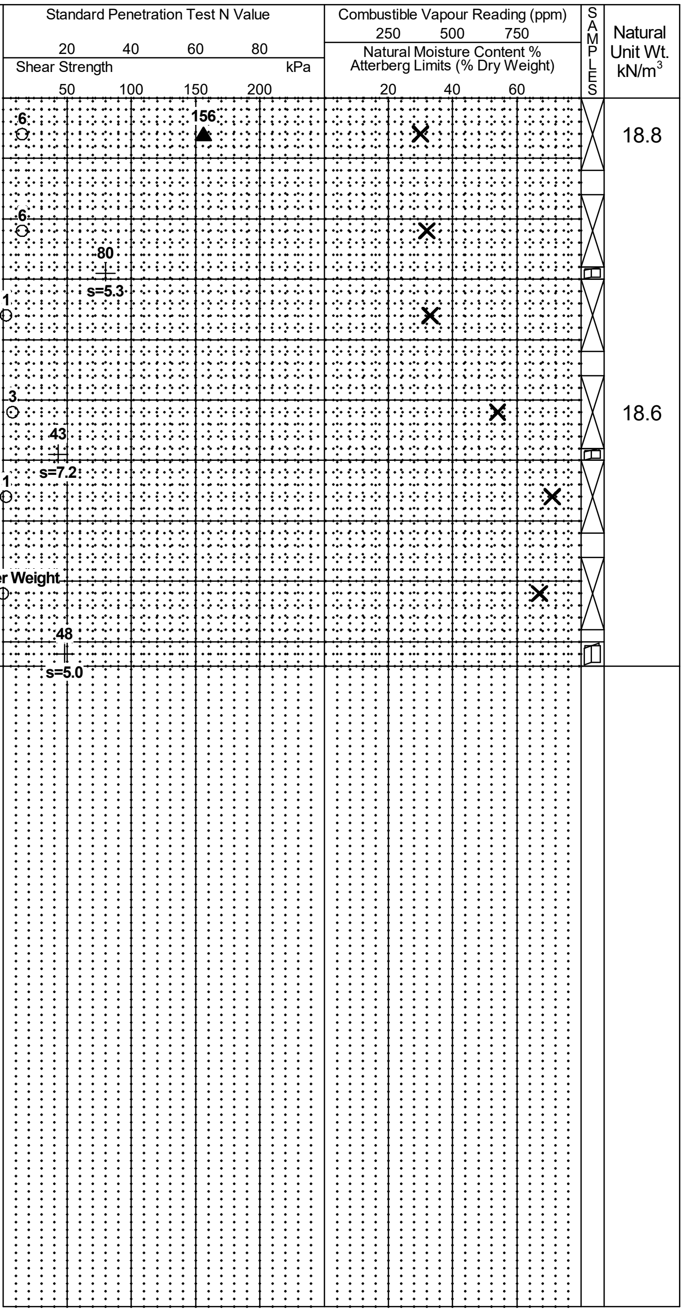
Drill Type: CME-75 Track Mount Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	Natural Unit Wt. kN/m³
	▨	**SANDY CLAY** Grey sand seams, sensitive, brown, moist, (very stiff) rootlets in the upper 600 mm	91.98	0	18.8
				1	
				2	
	▧	**CLAY** Sensitive, grey, wet, (firm)	89.8		18.6
▼			88.98	3	
			87.3		
		Borehole Terminated at 4.7 m Depth			



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.0	3.6

CORE DRILLING RECORD

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

LOG OF BOREHOLE BH LOGS - 245869 - RIVERSIDE SOUTH SCHOOL.GPJ TROW OTTAWA.GDT 7/24/18

Log of Borehole **BH-18**

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 21

Page. 1 of 1

Date Drilled: 'July 6, 2018

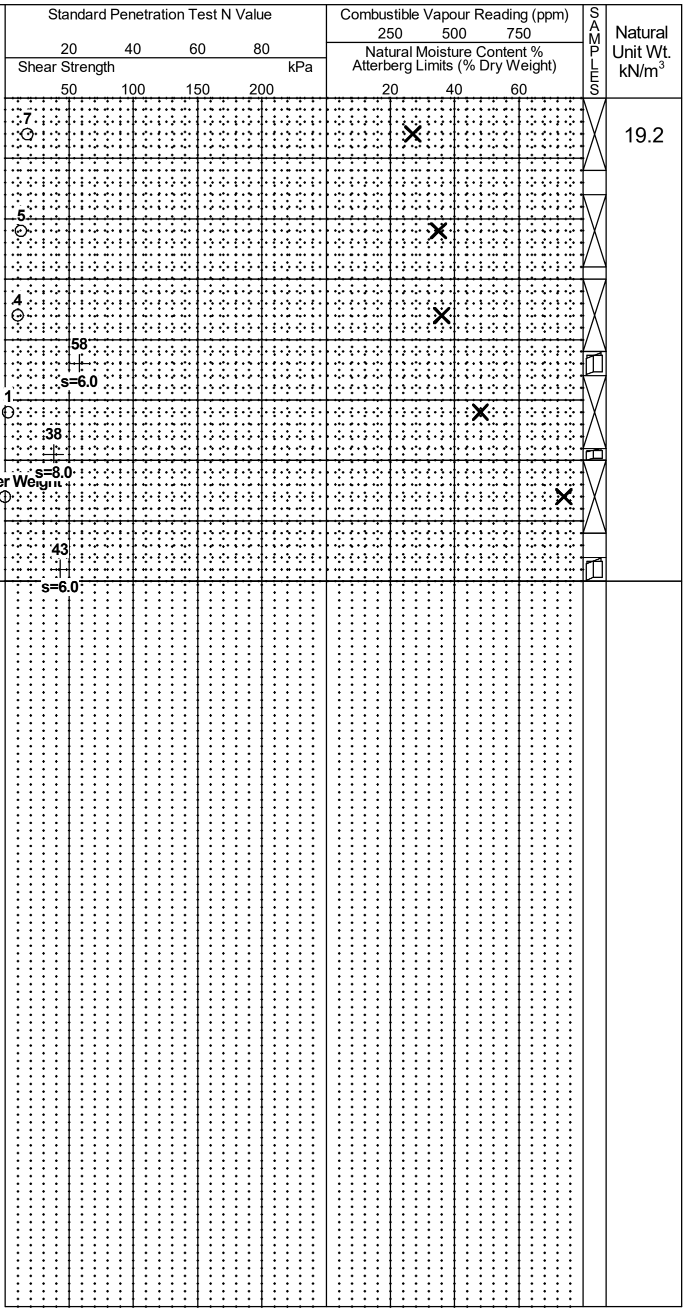
Drill Type: CME-75 Track Mount Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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	SOIL DESCRIPTION	Geodetic Elevation m	Depth
		92.21	0
	SANDY CLAY (possible fill) Grey sand seams, brown, moist, (firm)	91.5	
	SANDY SILT Grey and seams, brown, moist, (loose)	90.0	1 2
▼	**CLAY** Sensitive to extra-sensitive, grey, wet, (firm to stiff)	89.41 88.2	3 4
	Borehole Terminated at 4.0 m Depth		



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	3.0	3.6

CORE DRILLING RECORD

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

Log of Borehole BH-19

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 22

Page. 1 of 1

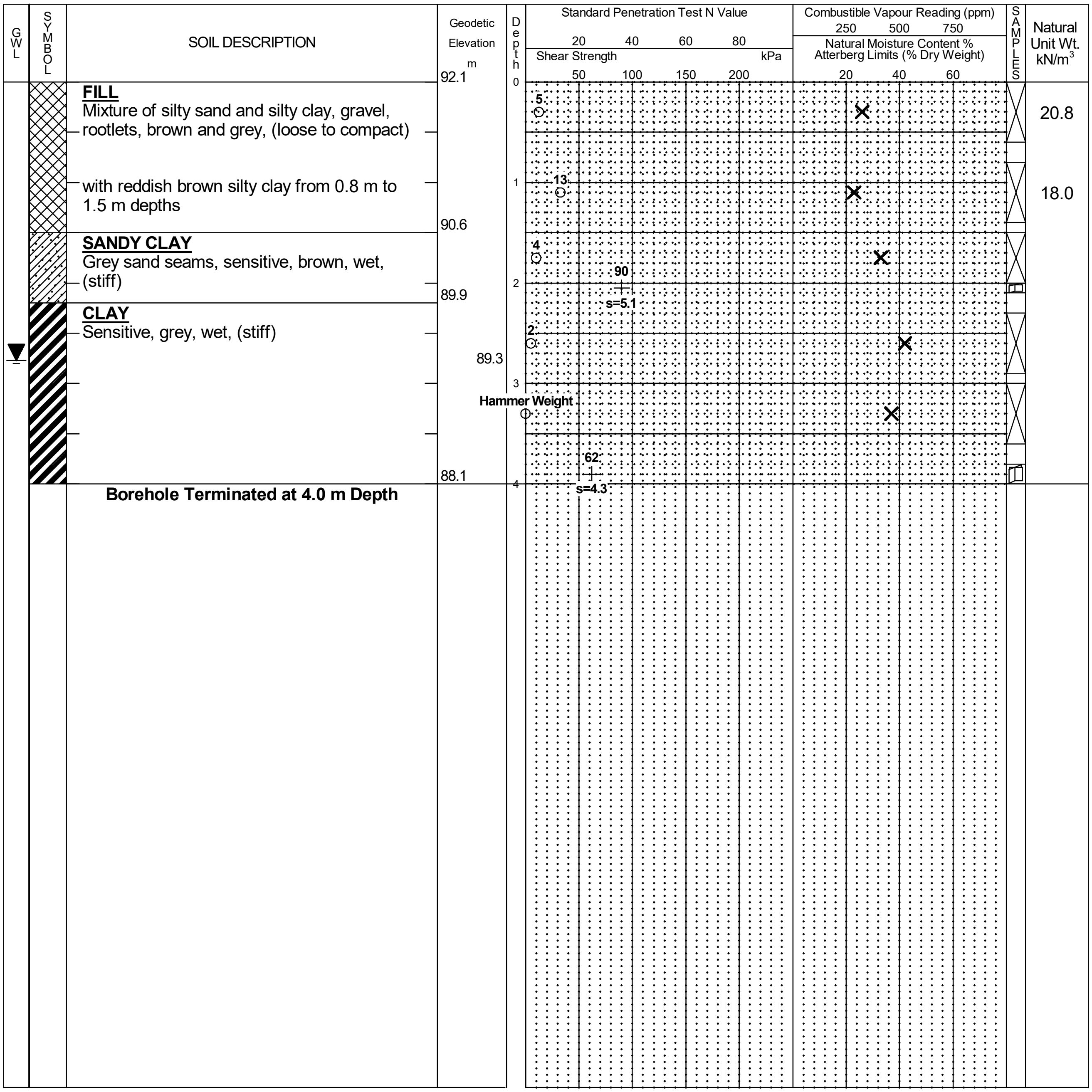
Date Drilled: 'July 6, 2018

Drill Type: CME-75 Track Mount Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲



FILL

Mixture of silty sand and silty clay, gravel, rootlets, brown and grey, (loose to compact)

with reddish brown silty clay from 0.8 m to 1.5 m depths

90.6

SANDY CLAY

Grey sand seams, sensitive, brown, wet, (stiff)

89.9

CLAY

Sensitive, grey, wet, (stiff)

88.1

Borehole Terminated at 4.0 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	2.8	3.0

CORE DRILLING RECORD

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

Log of Borehole BH-20

Project No: OTT-00245869-A0

Project: Geotechnical Investigation - Proposed New Riverside South Catholic Elementary School

Location: Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.

Figure No. 23

Page. 1 of 1

Date Drilled: 'July 6, 2018

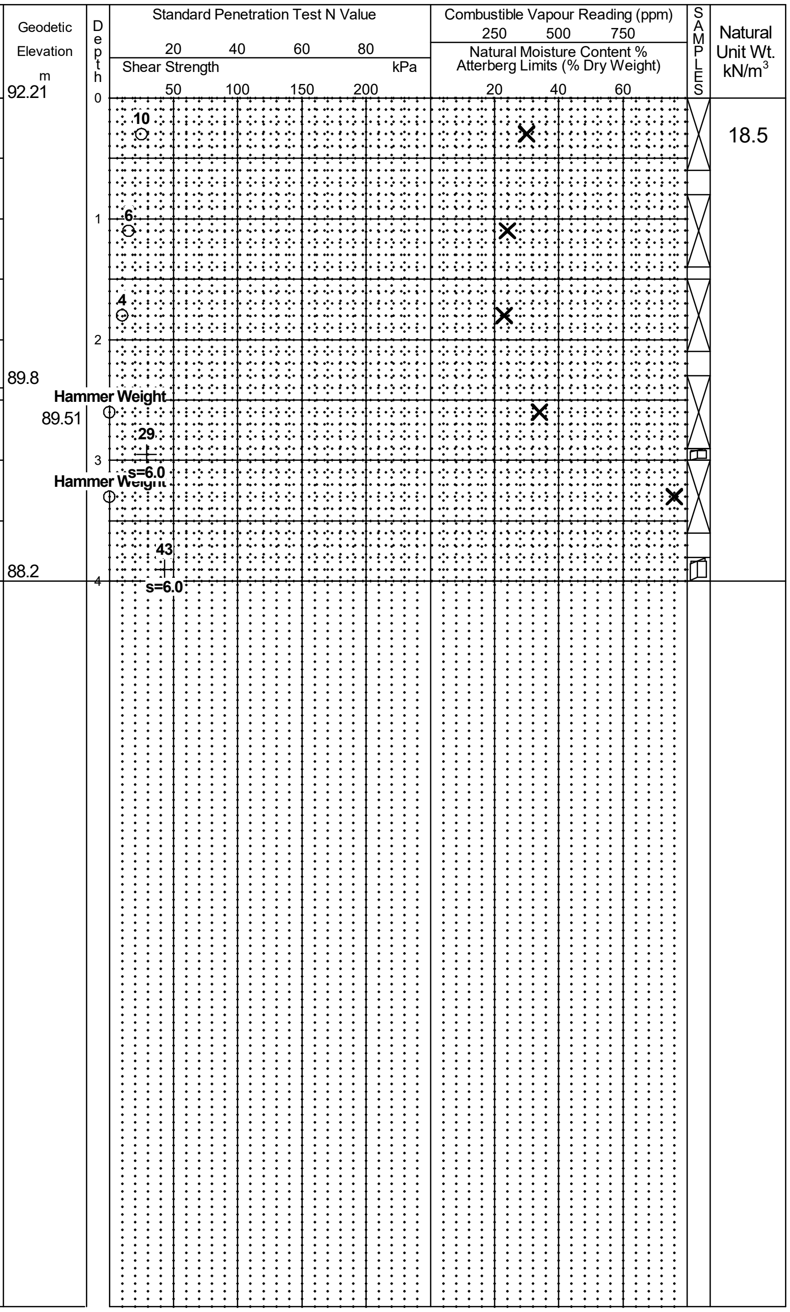
Drill Type: CME-75 Track Mount Drill Rig

Datum: Geodetic Elevation

Logged by: M.L. 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 Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth
	▩	**FILL** Mixture of silty sand, silty clay and topsoil, roots and rootlets, brown to dark brown and grey, moist (loose)	92.21	0
▼	▨	**CLAY** Sensitive, grey, wet, (firm)	89.8	
		Borehole Terminated at 4.0 m Depth	88.2	4



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled with cuttings upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00245869-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	2.7	3.0

CORE DRILLING RECORD

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

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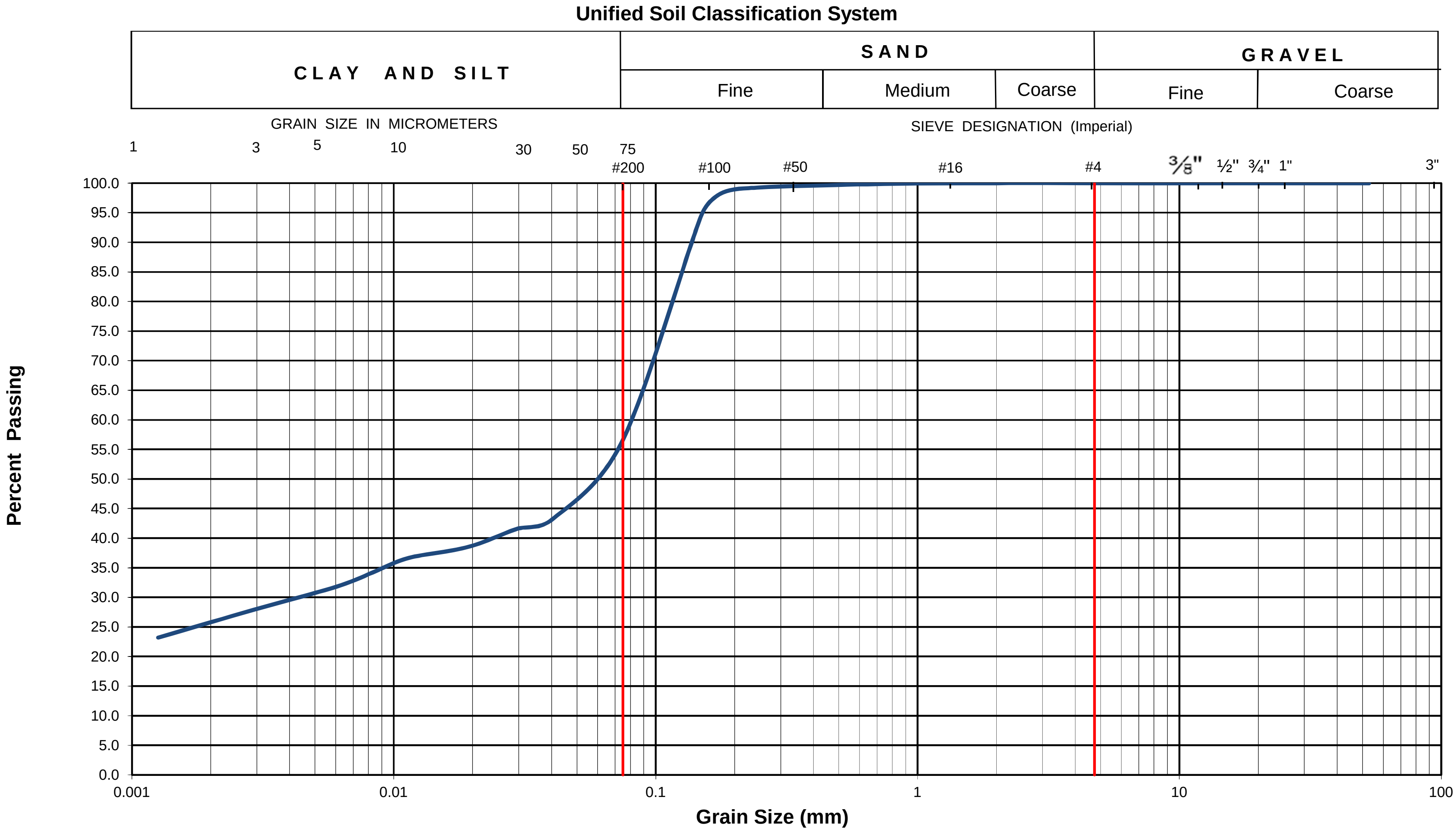
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-00245869-A0	**Project Name :**	Proposed New Riverside South Catholic Elementary School				
Client :	CECCE	**Project Location :**	Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.				
Date Sampled :	April 3, 2018	**Borehole No:**	BH1	**Sample No.:**	SS3	**Depth (m) :**	1.5-2.1
Sample Description :		**% Silt and Clay**	57	**% Sand**	43	**% Gravel**	0
Sample Description :	Sandy Clay (CL)					**Figure :**	24

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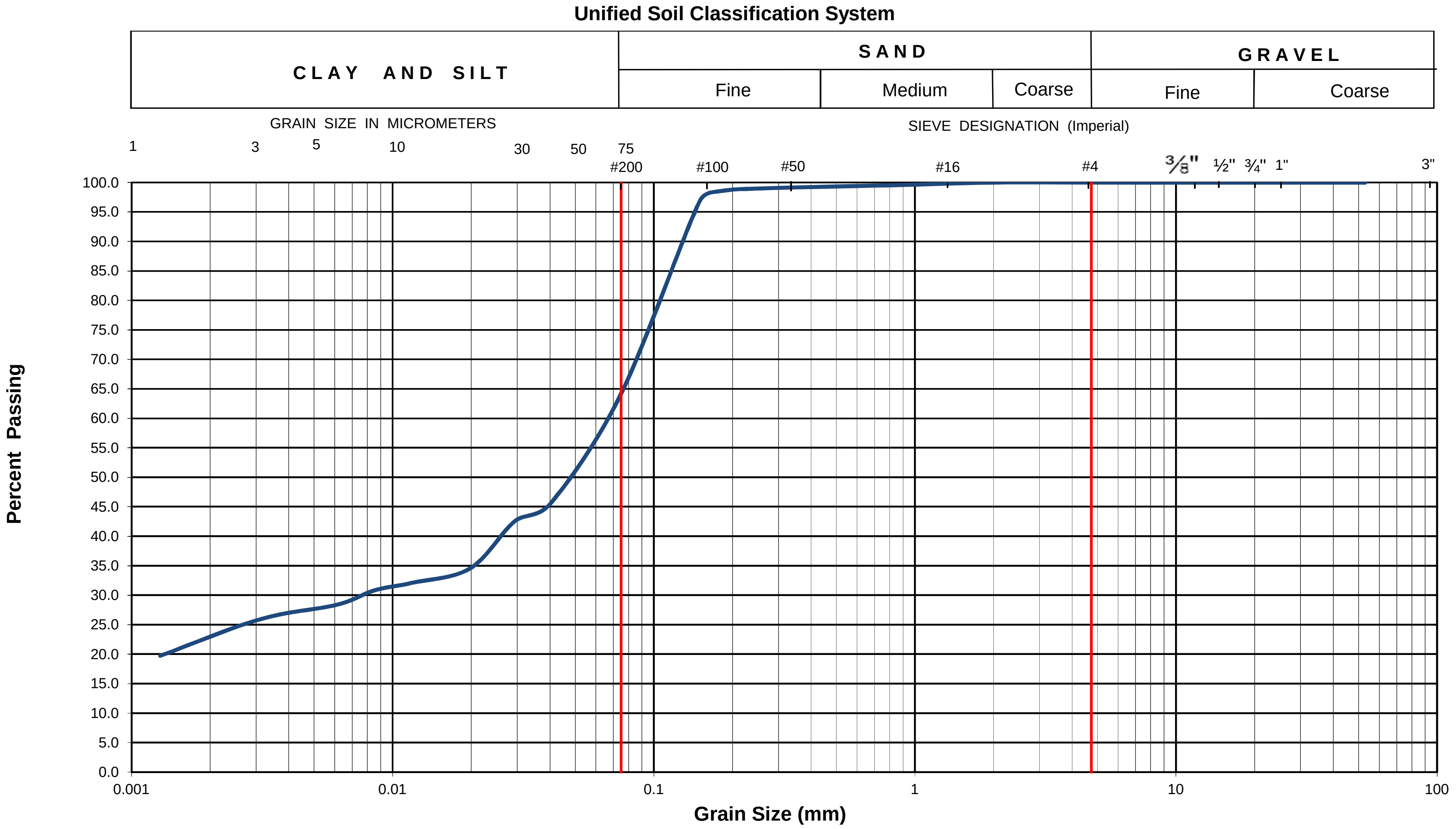
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 3, 2018**	**Borehole No:**		**BH2**	**Sample No.:**		**SS4**	**Depth (m) :**	**2.3-2.9**
Sample Description :		**% Silt and Clay**	**64**	**% Sand**	**36**	**% Gravel**	**0**	**Figure :**	**25**
Sample Description :	**Sandy Clay (CL)**								

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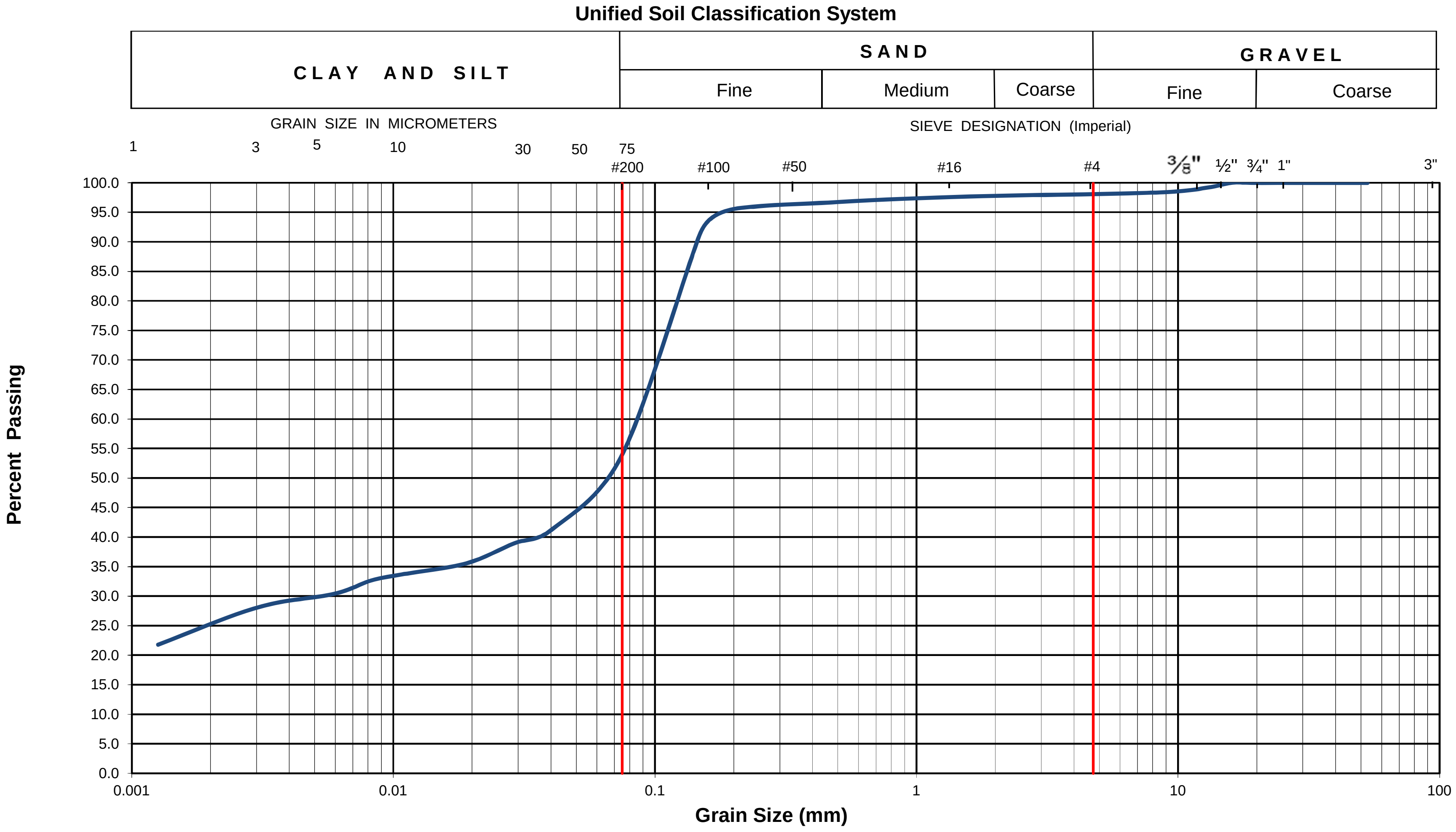
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 3, 2018**	**Borehole No:**	**BH3**	**Sample No.:**	**SS2**			**Depth (m) :**	**0.8-1.4**
Sample Description :		**% Silt and Clay**	**54**	**% Sand**	**44**	**% Gravel**	**2**	**Figure :**	**26**
Sample Description :	**Sandy Clay (CL)**								

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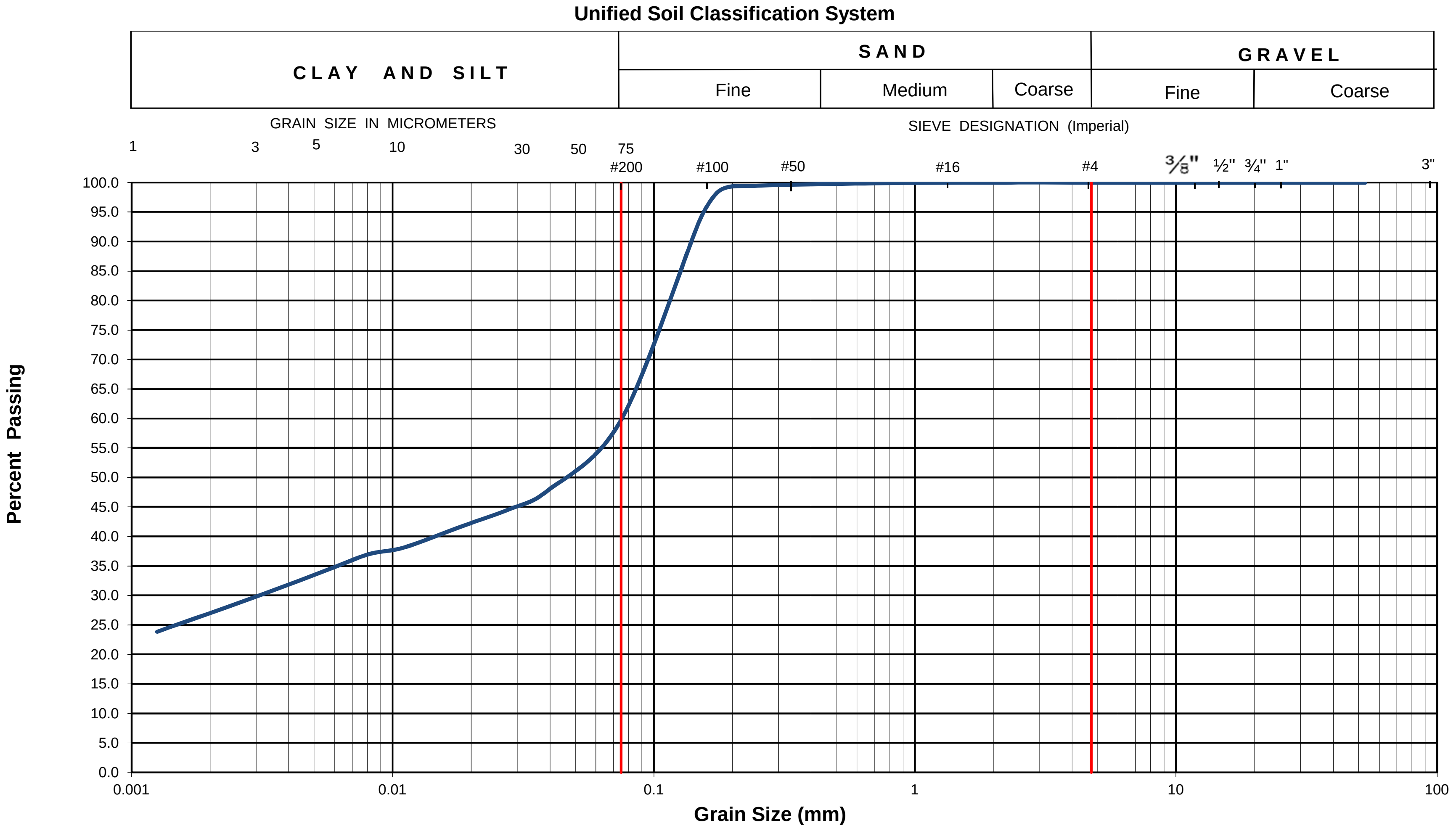
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 3, 2018**	**Borehole No:**	**BH4**	**Sample No.:**	**SS3**			**Depth (m) :**	**1.5-2.1**
Sample Description :		**% Silt and Clay**	**60**	**% Sand**	**40**	**% Gravel**	**0**	**Figure :**	**27**
Sample Description :	**Sandy Clay (CL)**								

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

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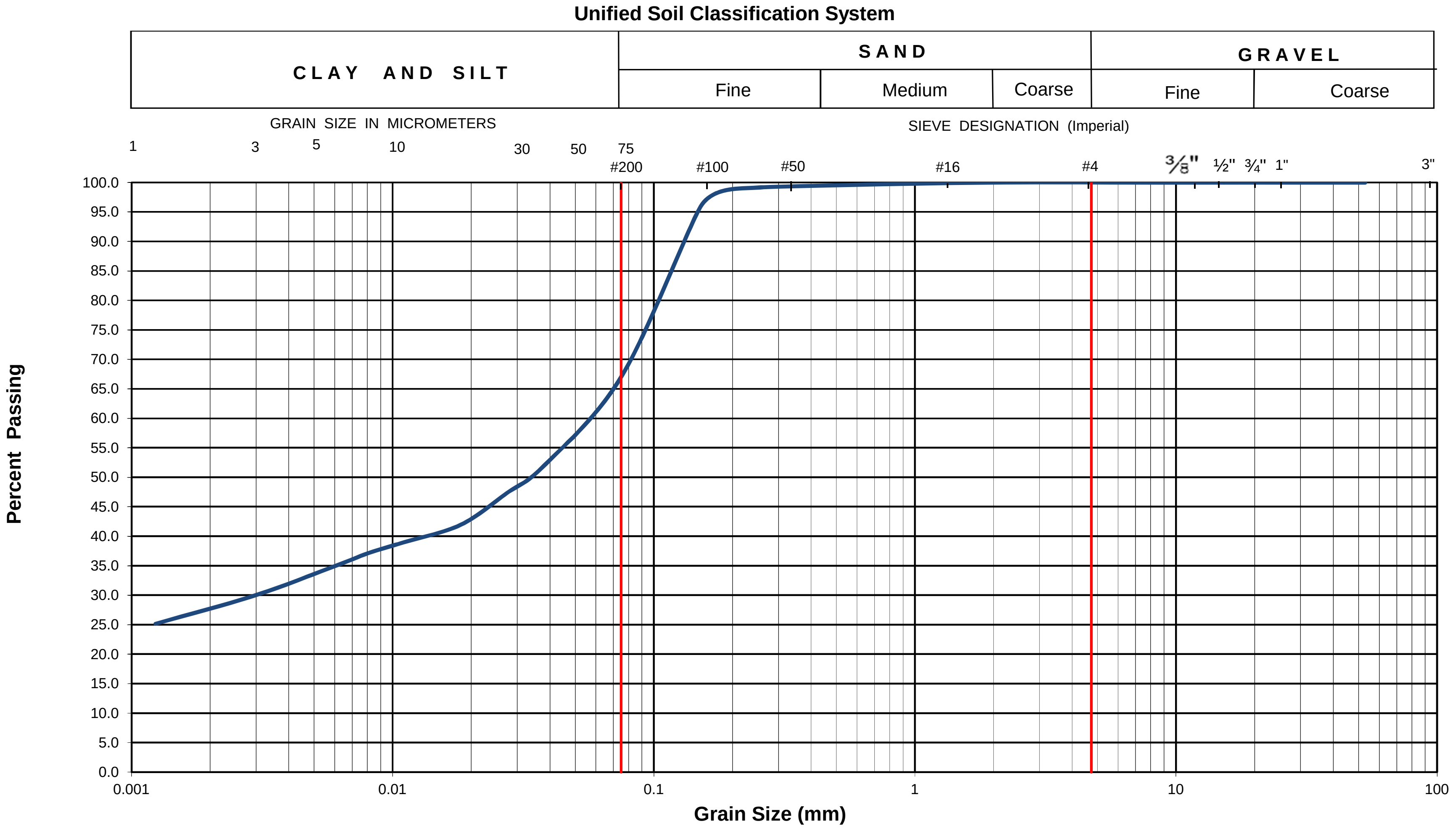

EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 3, 2018**	**Borehole No:**		**BH5**	**Sample No.:**	**SS4**		**Depth (m) :**	**2.3-2.9**
Sample Description :		**% Silt and Clay**	**67**	**% Sand**	**33**	**% Gravel**	**0**	**Figure :**	**28**
Sample Description :	**Sandy Clay (CL)**								

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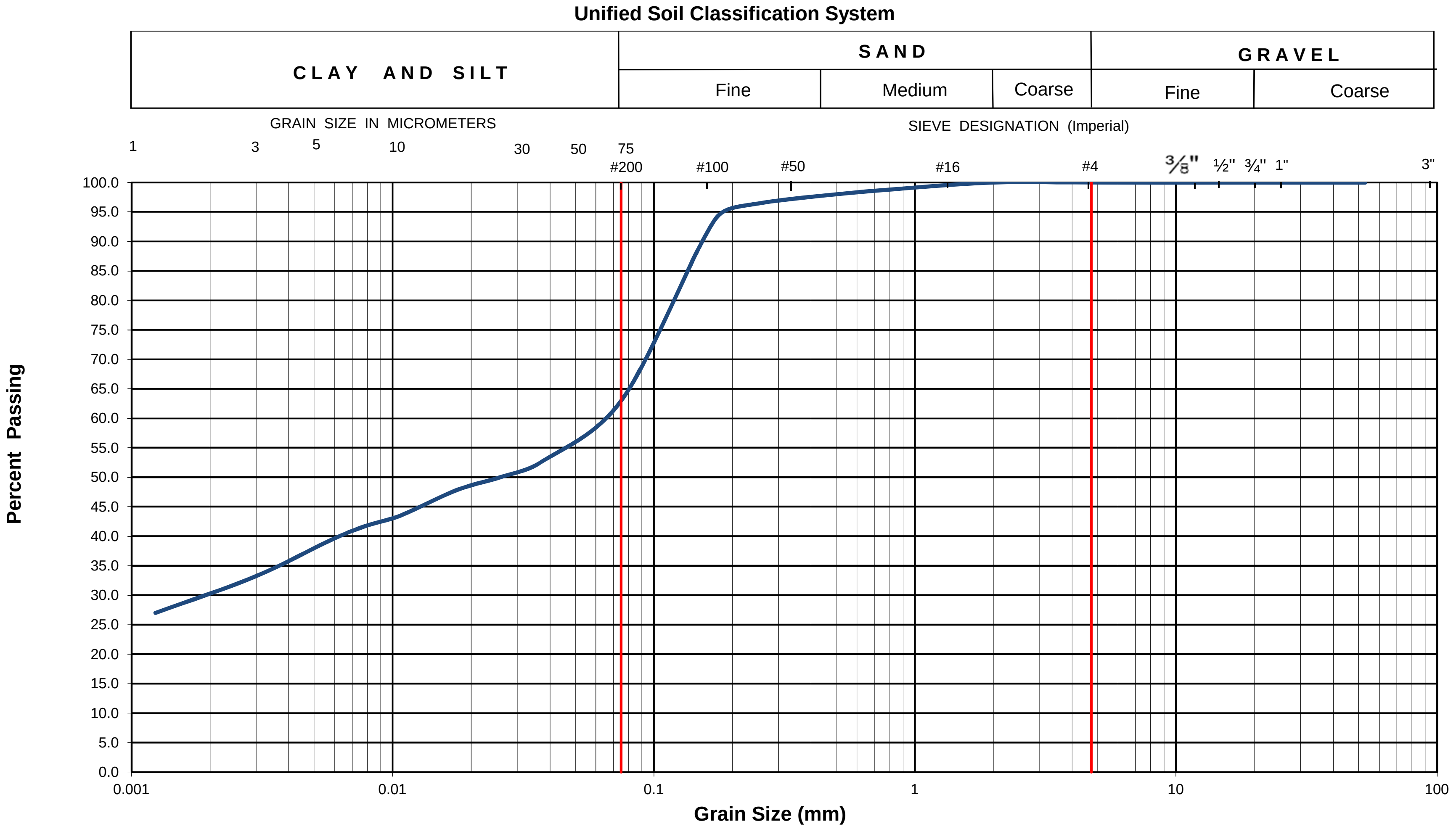
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Elementay School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 2, 2018**	**Borehole No:**		**BH6**	**Sample No.:**	**SS2**		**Depth (m) :**	**0.8-1.4**
Sample Description :		**% Silt and Clay**	**63**	**% Sand**	**37**	**% Gravel**	**0**	**Figure :**	**29**
Sample Description :	**Sandy Clay (CL)**								

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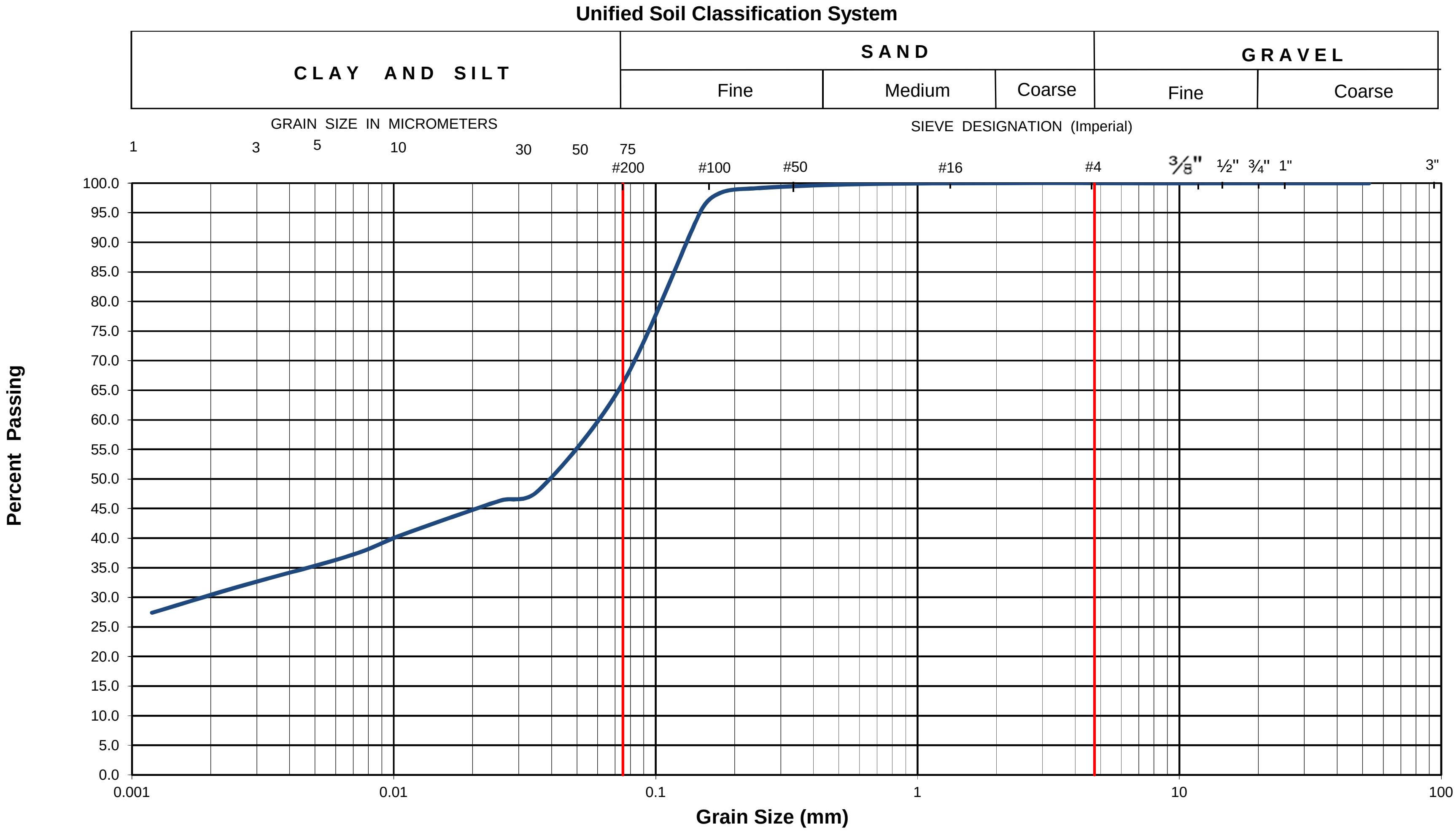
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-00245869	**Project Name :**	Geotechnical Investigation Riverside South Catholic Elementary School				
Client :	CECCE	**Project Location :**	South of Earl Armstrong Between Spratt and Limebank Rd, Ottawa, ON				
Date Sampled :	July 9, 2018	**Borehole No:**	BH16	**Sample No.:**	SS3	**Depth (m) :**	1.5-2.1
Sample Description :		**% Silt and Clay**	66	**% Sand**	34	**% Gravel**	0
Sample Description :	Sandy Clay (CL)					**Figure :**	30

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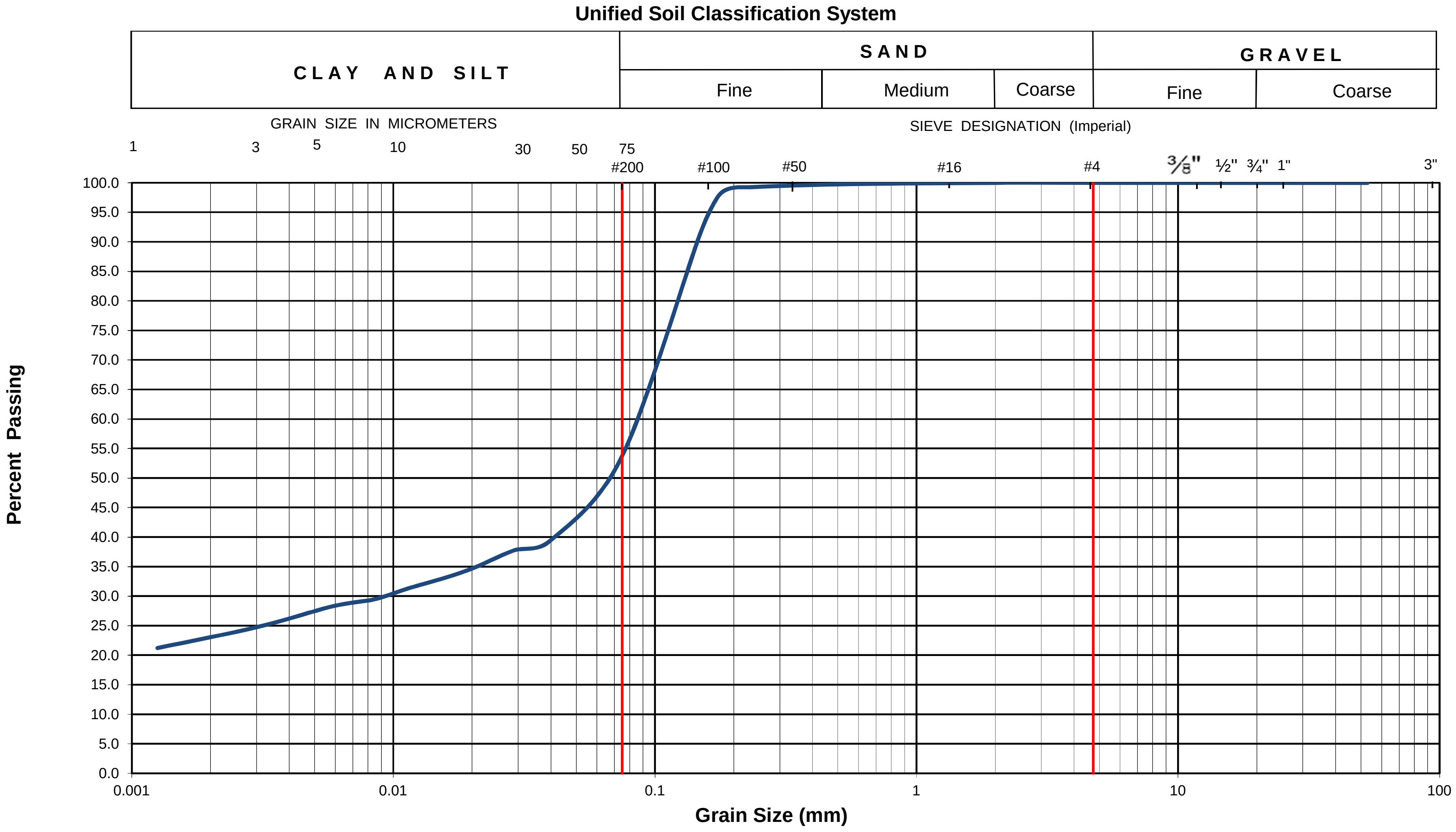
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869**	**Project Name :**	**Geotechnical Investigation Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**South of Earl Armstrong Between Spratt and Limebank Rd, Ottawa, ON**						
Date Sampled :	**July 9, 2018**	**Borehole No:**	**BH18**	**Sample No.:**	**SS2**			**Depth (m) :**	**0.8-1.4**
Sample Description :		**% Silt and Clay**	**54**	**% Sand**	**46**	**% Gravel**	**0**	**Figure :**	**31**
Sample Description :	**Sandy Silt (ML)**								

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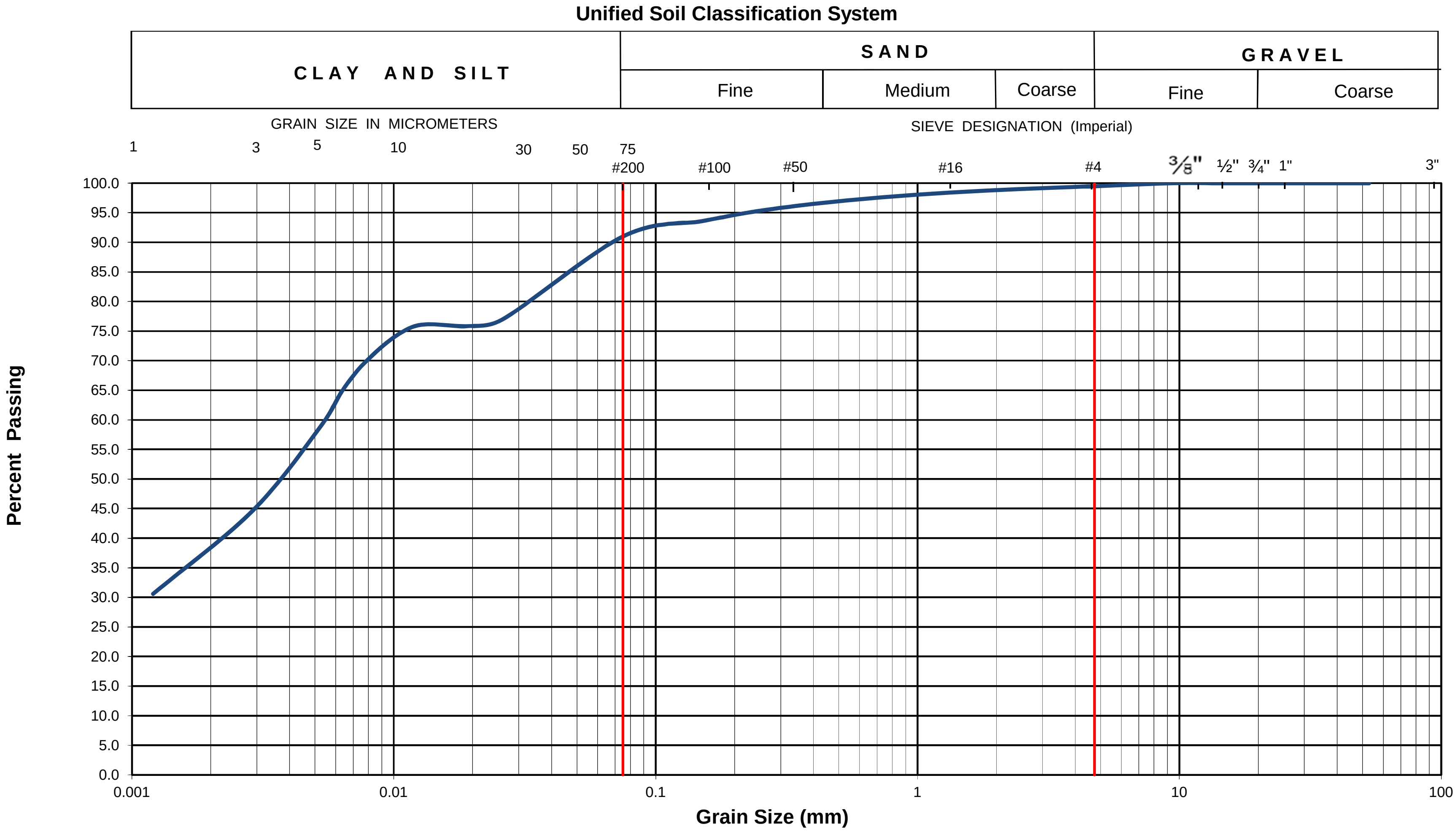
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Catholic Elementary School**					
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**					
Date Sampled :	**April 2, 2018**	**Borehole No:**	**BH3**	**Sample No.:**	**SS8**		**Depth (m) :**	**6.1-6.7**
Sample Description :		**% Silt and Clay**	**91**	**% Sand**	**8**	**% Gravel** **1**	**Figure :**	**32**
Sample Description :	**Clay (CL)**							

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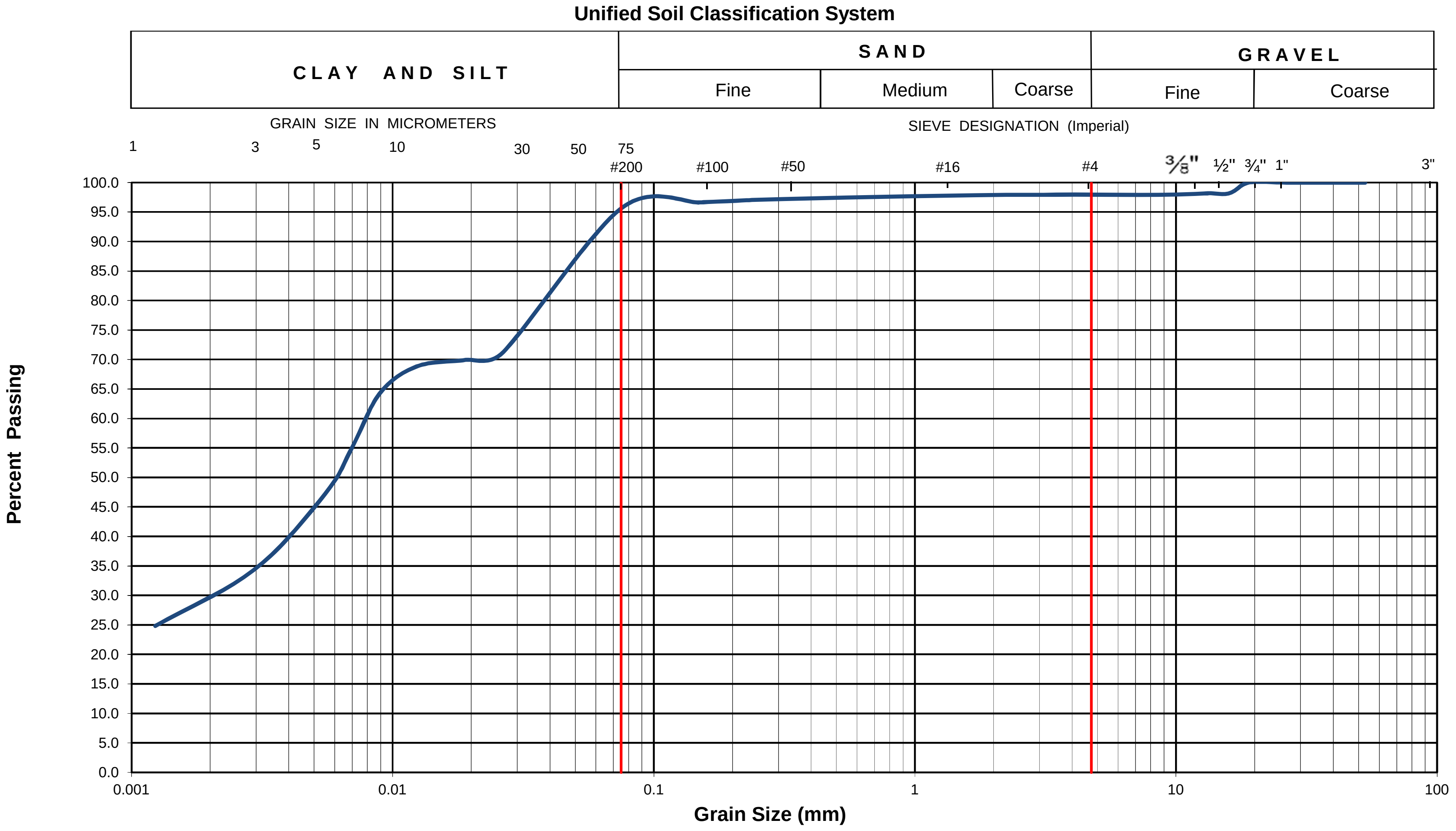
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-00245869-A0	**Project Name :**	Proposed New Riverside South Catholic Elementary School				
Client :	CECCE	**Project Location :**	Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.				
Date Sampled :	April 3, 2018	**Borehole No:**	BH 4	**Sample No.:**	SS9	**Depth (m) :**	7.6-8.2
Sample Description :		**% Silt and Clay**	96	**% Sand**	2	**% Gravel**	2
Sample Description :	Silty Clay (CL)					**Figure :**	33

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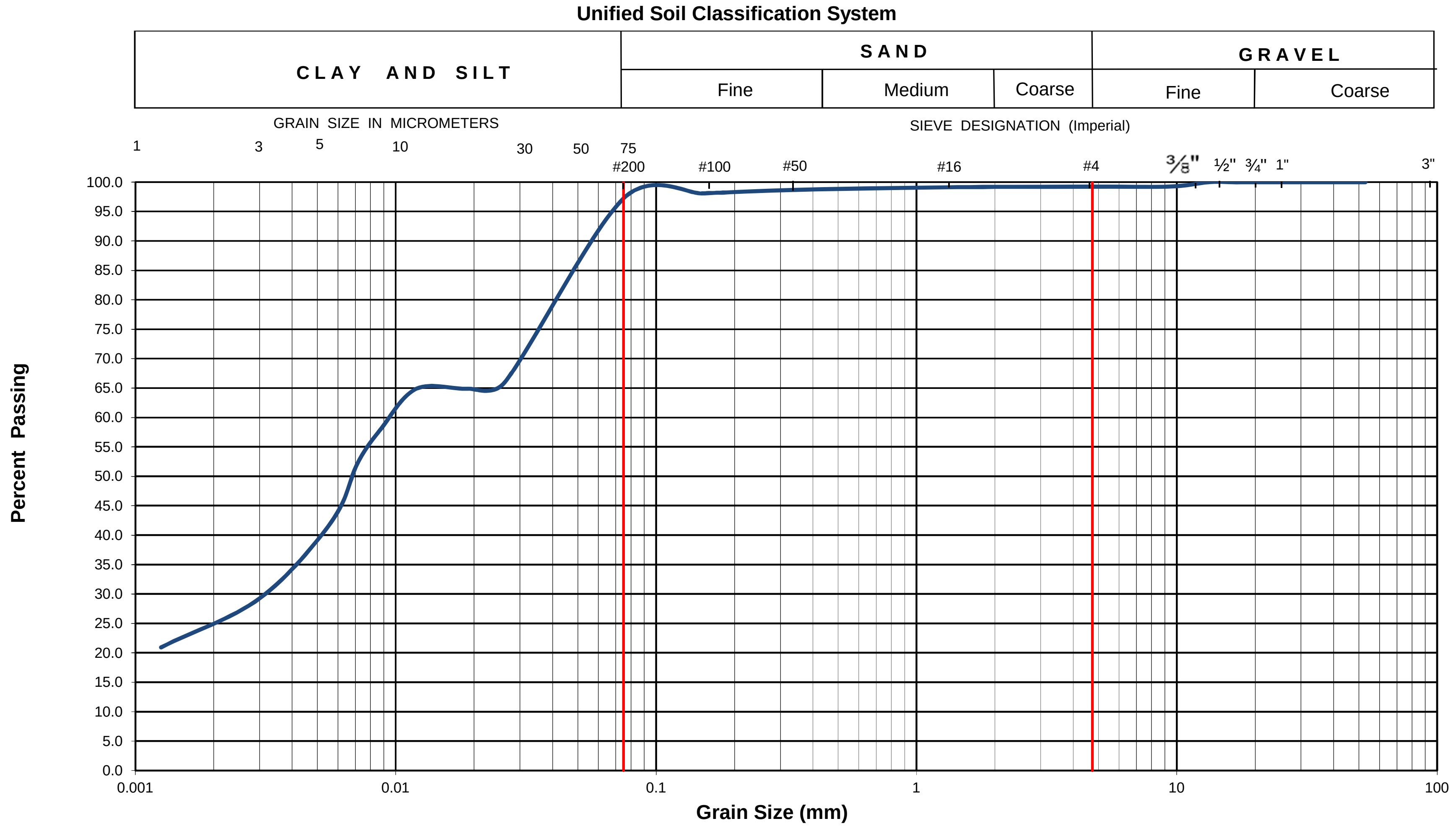
Ottawa, ON K2B 8H6

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



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 2, 2018**	**Borehole No:**		**BH6**	**Sample No.:**	**SS8**		**Depth (m) :**	**6.1-6.7**
Sample Description :		**% Silt and Clay**	**97**	**% Sand**	**2**	**% Gravel**	**1**	**Figure :**	**34**
Sample Description :	**Silty Clay (CL)**								

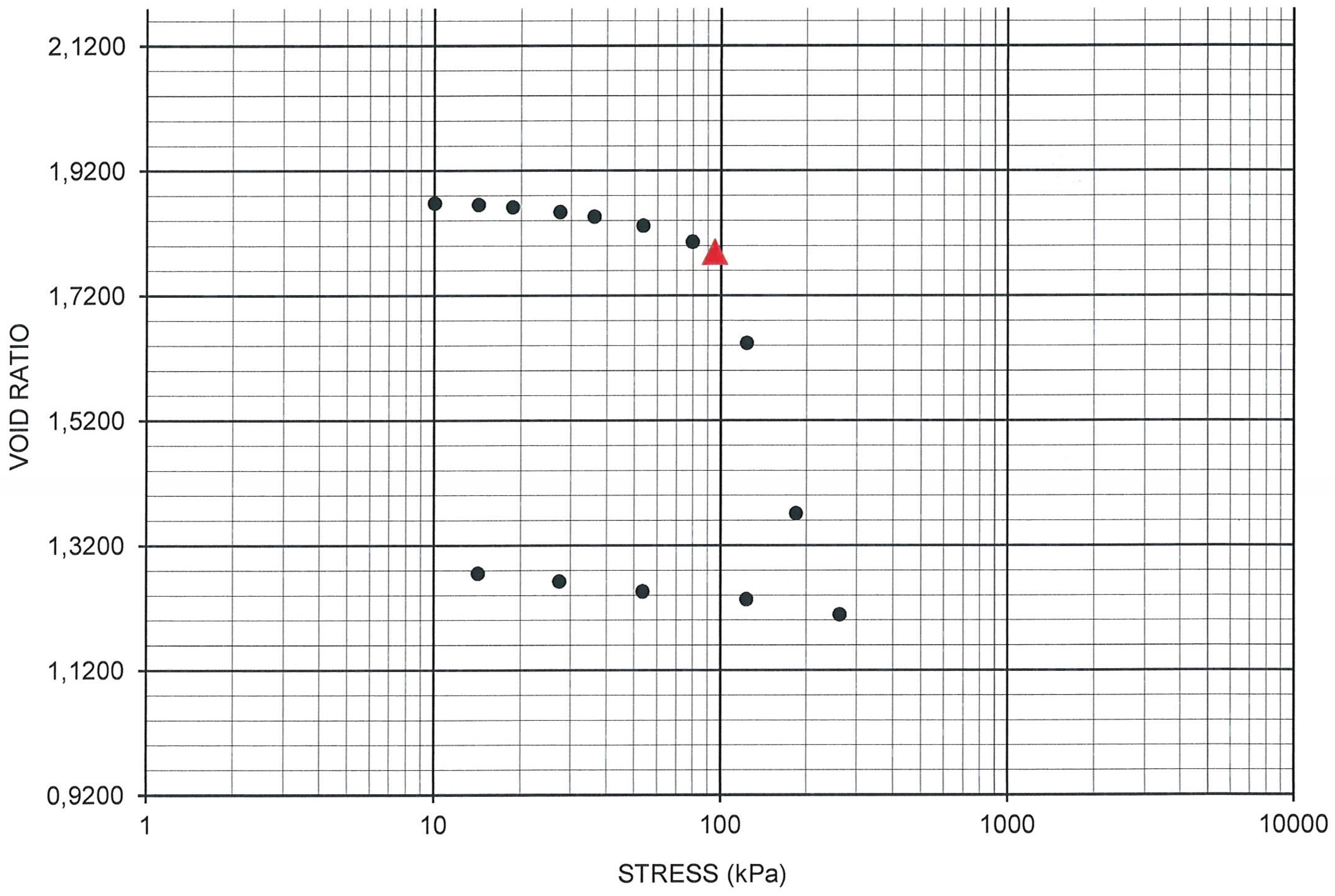
Englobe

One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D 2435 - Taylor Method

Client: Y/Project: OTT-00245869-A0
Date : 2018-04-10
Project : EXP Ontario
Our file No. : P-0011703-6-01
Boring No. : BH-1, ST-6
Sample No. : 10
Depth (m) : 4,10 to 4,20m
Hydrostatic stress at the test (date) :
Provided by ☐ the client ☐ Englobe

STRESS vs VOID RATIO CURVE



Geotechnical Characteristics of Soils :

Initial void ratio (e_o) :	1,873	Recompression index (C_r) :	0,042
Initial water content (w) :	67,6%	Virgin compression index (C_c) :	
Initial humid unit weight (γ_h) :	15,7 kN/m³	Initial effective stress (σ'_v) :	58 KPa
Initial saturation degree (S_r) :	99,2%	Preconsolidation pressure (σ'_p):	95 KPa
		Overconsolidation deviation ($\Delta\sigma$) :	34 kPa

Remarks : The sampling and transportation of the sample were carried out by a client's representative.
The initial effective stress has been provided by the client.

Prepared by :

A. Bouadma

Adlane Bouadma, Jr Eng

Verified by :

Famakhan Fainke, Eng

Englobe

One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D 2435 - Taylor Method

Client: Y-Project: OTT-00245869-A0

Date : 2018-04-10

Project : EXP Ontario

Our file No. : P-0011703-6-01

Boring No. : BH-6, ST-5

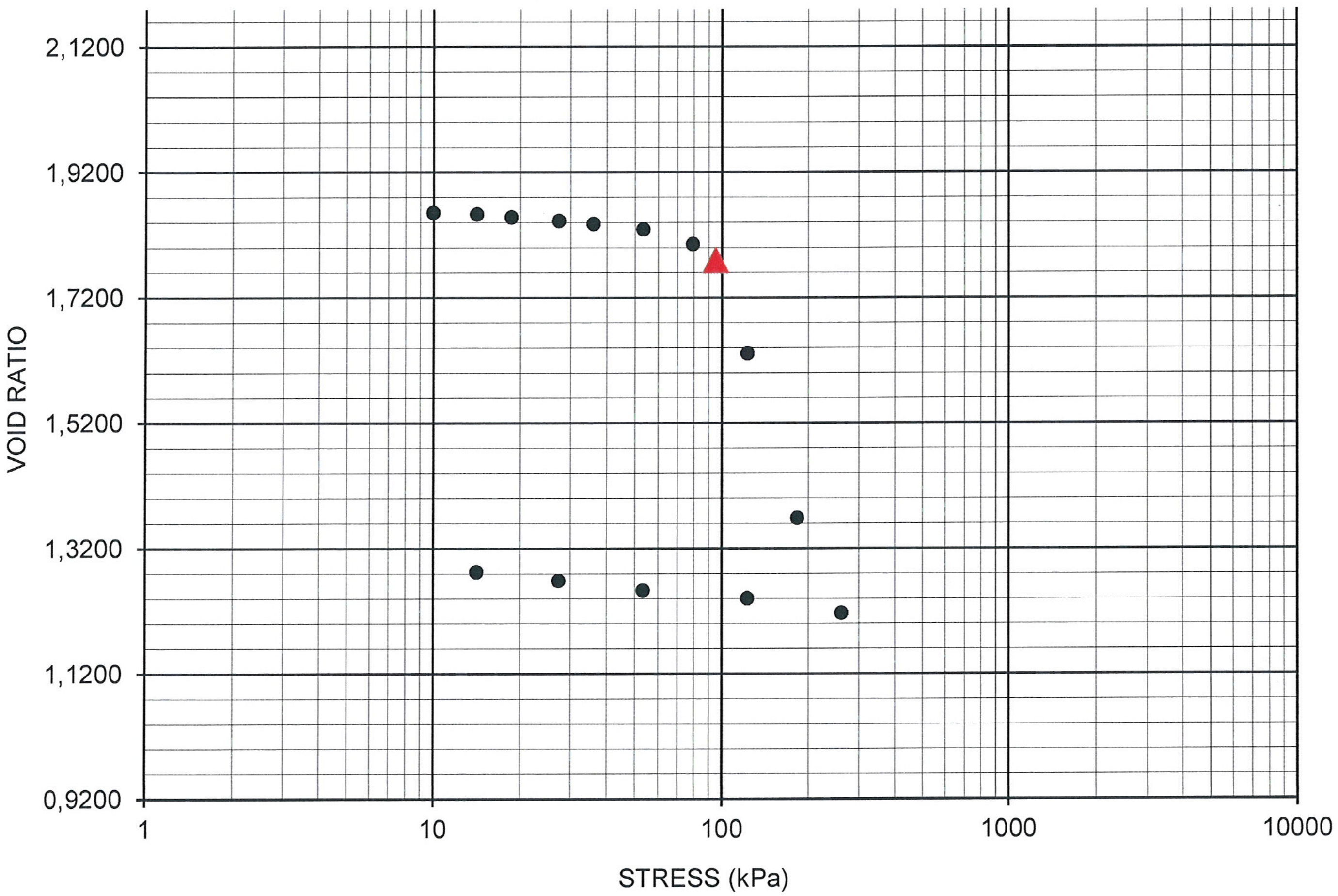
Sample No. : 11

Depth (m) : 3,3 to 3,4m

Hydrostatic stress at the test (date) :

Provided by ☐ the client ☐ Englobe

STRESS vs VOID RATIO CURVE



Geotechnical Characteristics of Soils :

Initial void ratio (e_o) :	1,856	Recompression index (C_r) :	0,038
Initial water content (w) :	67,0%	Virgin compression index (C_c) :	1,49
Initial humid unit weight (γ_h) :	15,8 kN/m³	Initial effective stress (σ'_v) :	43 KPa
Initial saturation degree (S_r) :	99,3%	Preconsolidation pressure (σ'_p):	95 KPa
		Overconsolidation deviation ($\Delta\sigma$) :	41 kPa

Remarks : The sampling and transportation of the sample were carried out by a client's representative.
The initial effective stress has been provided by the client.

Prepared by :

A. Bouadma

Adlane Bouadma, Jr Eng

Verified by :

Famakhan Fainke, Eng

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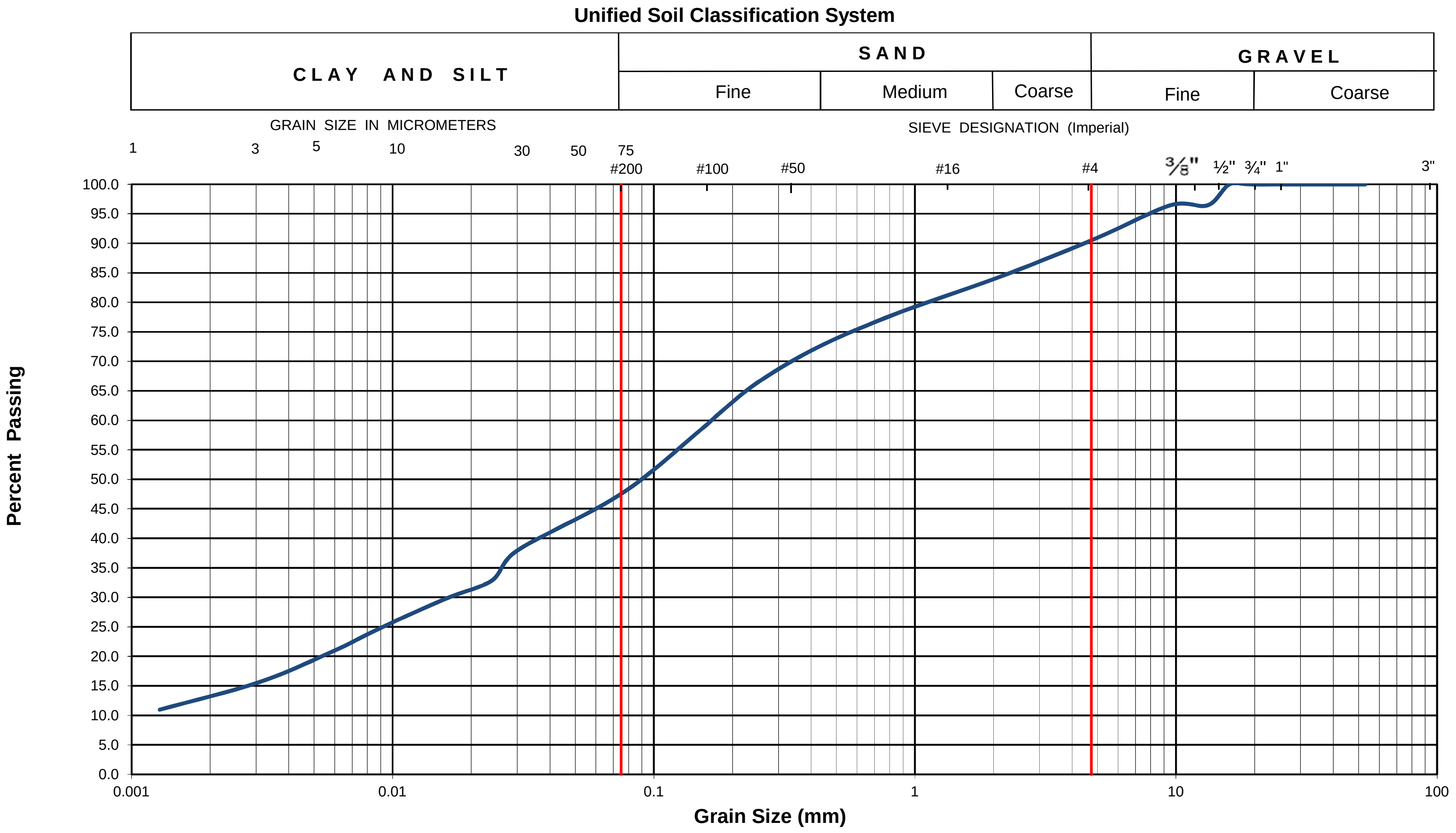
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Geotechnical Investigation Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 3, 2018**	**Borehole No:**	**BH 4**		**Sample No.:**	**SS10**		**Depth (m) :**	**9.9-10.5**
Sample Description :		**% Silt and Clay**	**48**	**% Sand**	**43**	**% Gravel**	**9**	**Figure :**	**37**
Sample Description :	**Sandy Clay (CL)**								

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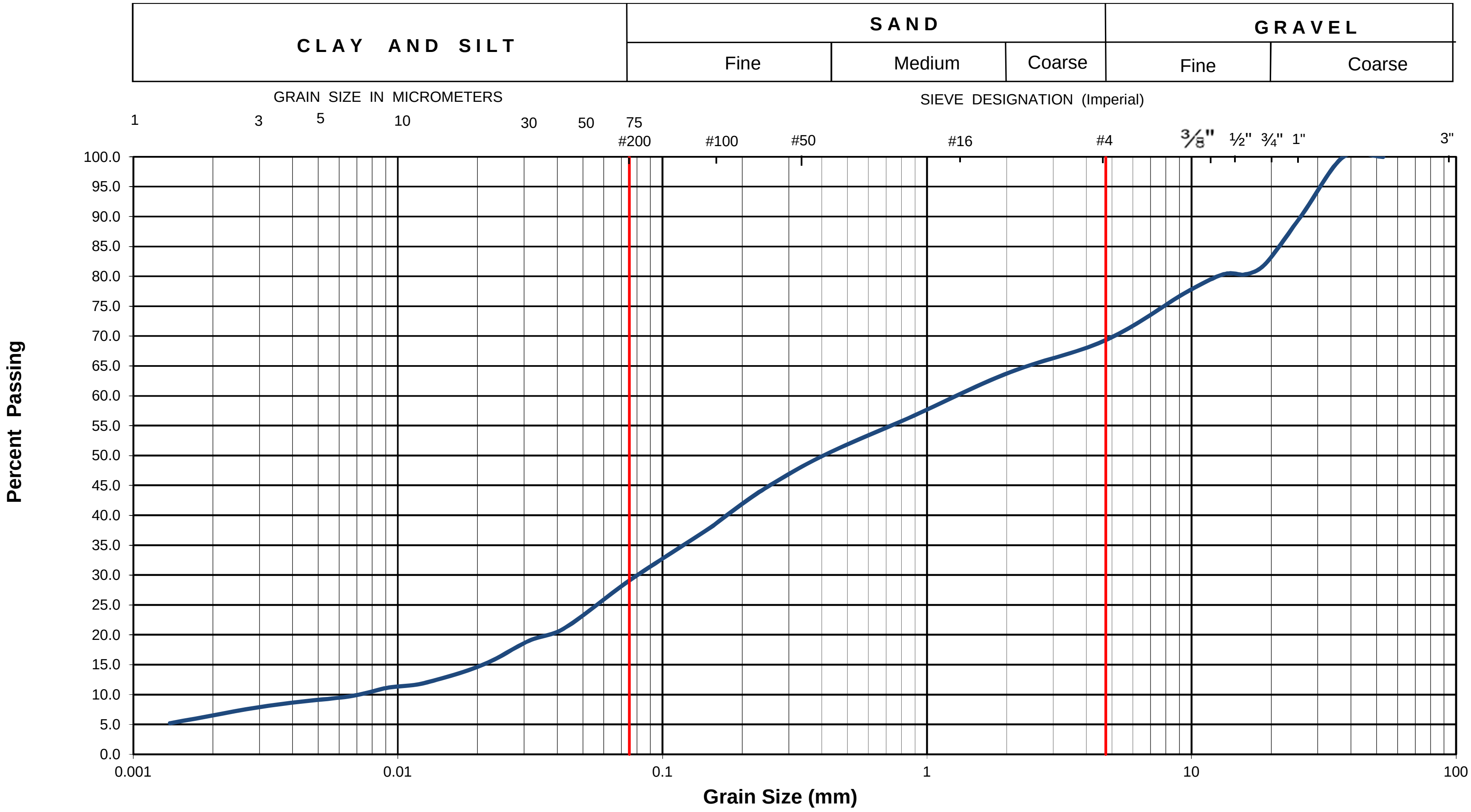
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422

Unified Soil Classification System



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Moutn Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 3, 2018**	**Borehole No:**	**BH2**	**Sample No.:**	**SS9**	**Depth (m) :**	**7.6-8.2**		
Sample Description :		**% Silt and Clay**	**29**	**% Sand**	**40**	**% Gravel**	**31**	**Figure :**	**38**
Sample Description :	**Silty Sand with Gravel (SM)**								

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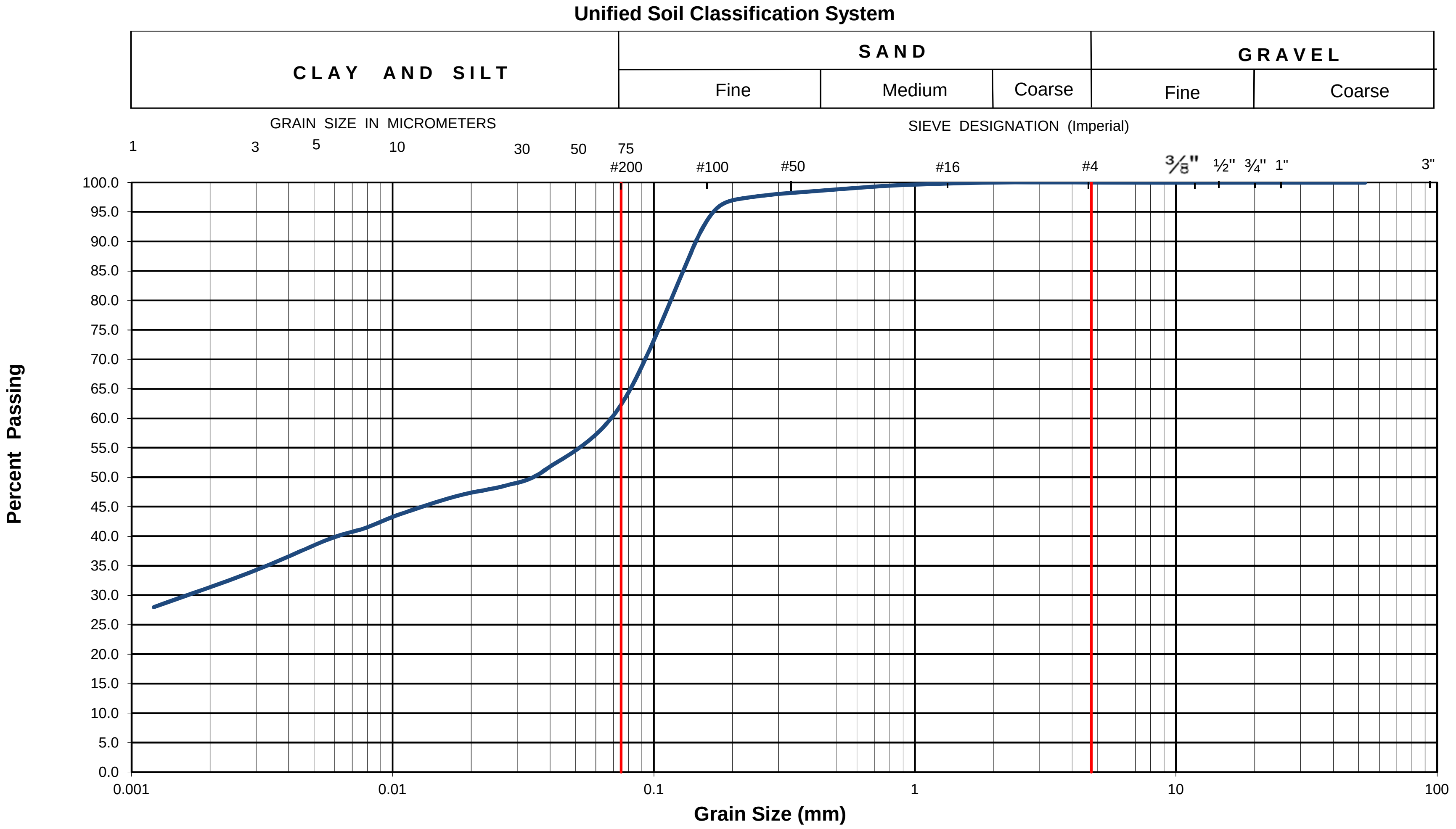
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside Catholic Elementary School**				
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**				
Date Sampled :	**April 3, 2018**	**Borehole No:**	**BH11**	**Sample No.:**	**SS2**	**Depth (m) :**	**0.6-1.2**
Sample Description :		**% Silt and Clay**	**62**	**% Sand**	**38**	**% Gravel**	**0**
Sample Description :	**Fill - Sandy Clay (CL)**					**Figure :**	**39**

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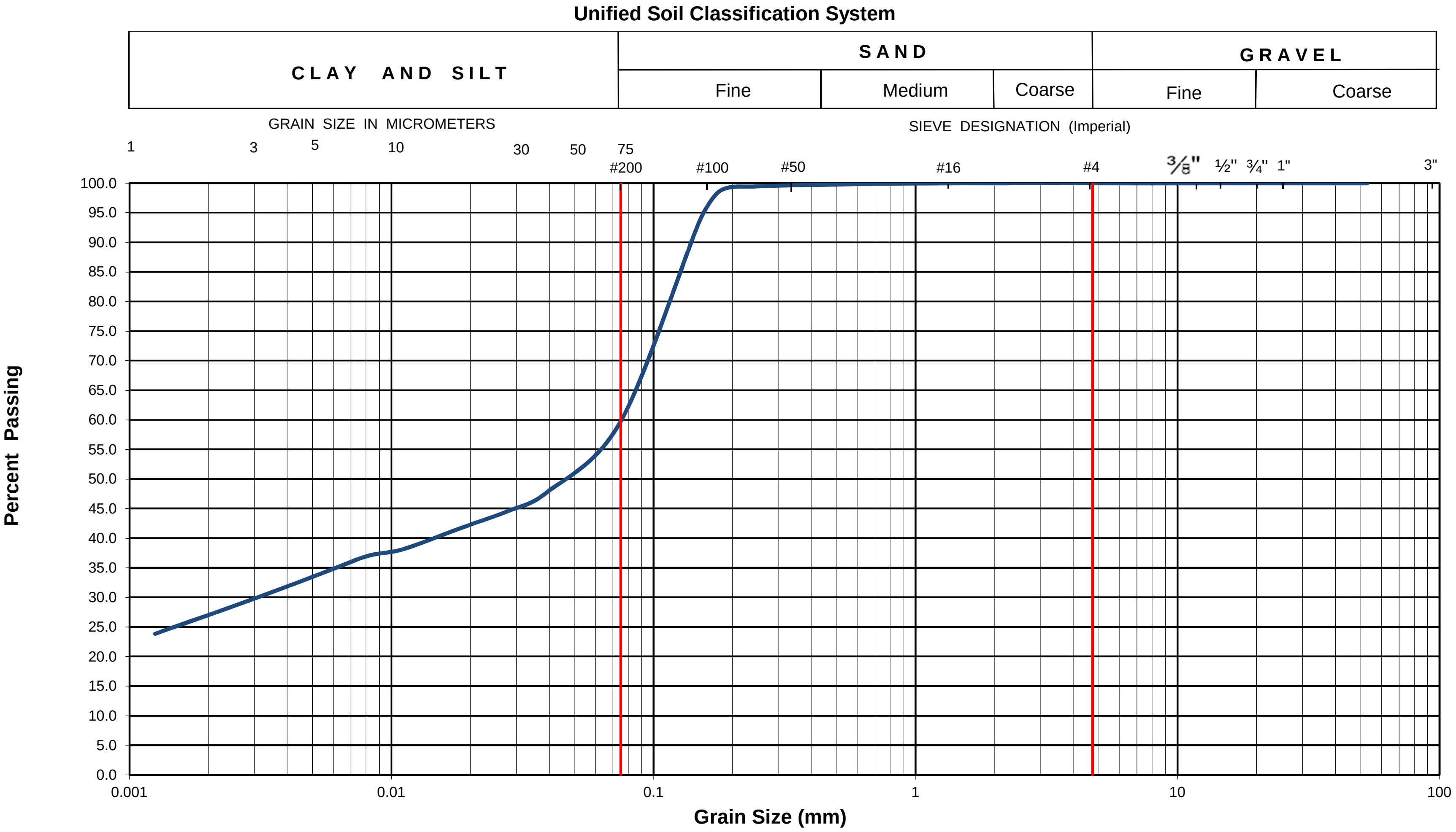
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 3, 2018**	**Borehole No:**	**BH7**	**Sample No.:**	**SS3**			**Depth (m) :**	**1.5-2.1**
Sample Description :		**% Silt and Clay**	**65**	**% Sand**	**35**	**% Gravel**	**0**	**Figure :**	**40**
Sample Description :	**Sandy Clay (CL)**								

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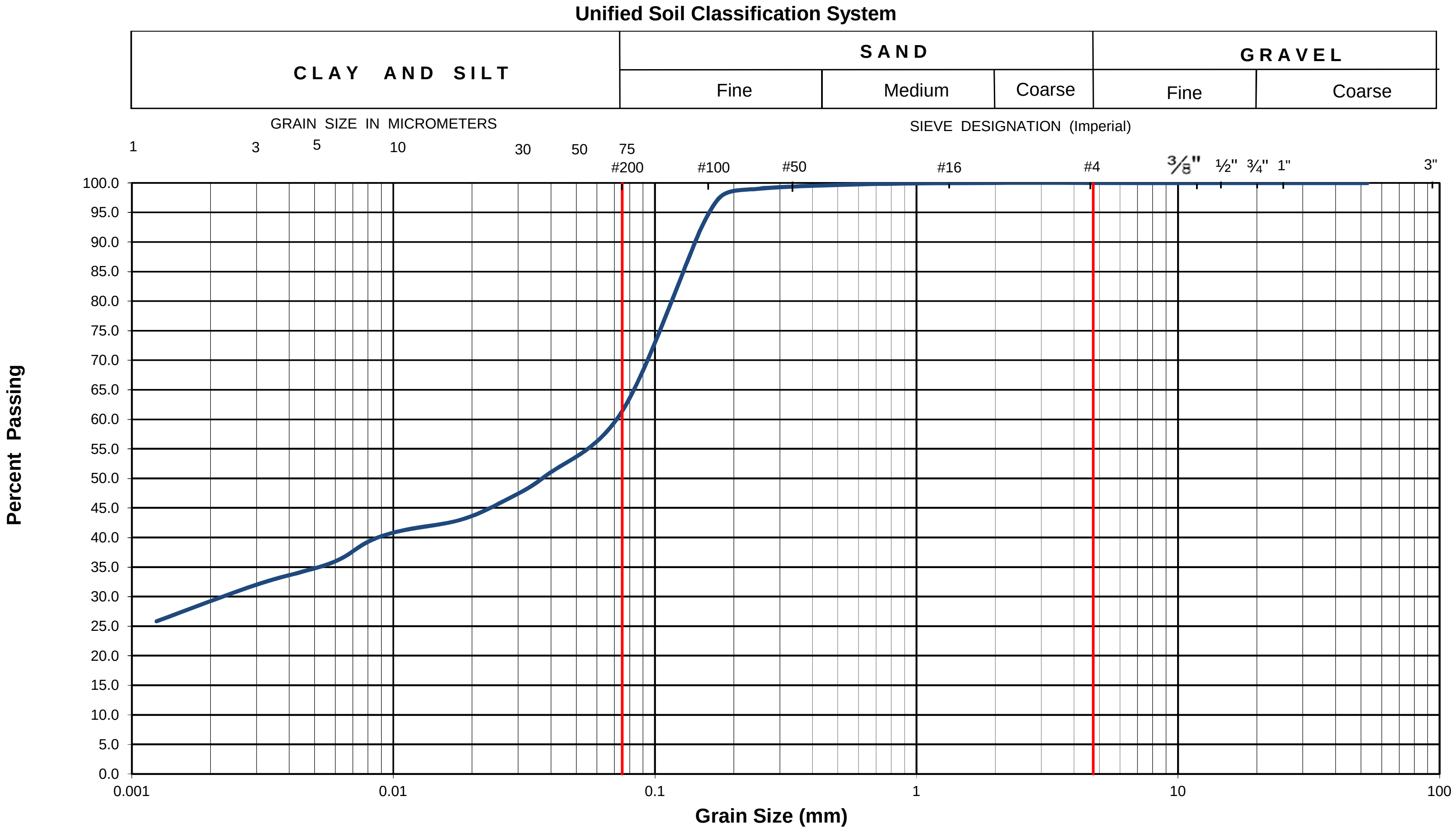
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-00245869-A0**	**Project Name :**	**Proposed New Riverside South Catholic Elementary School**						
Client :	**CECCE**	**Project Location :**	**Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.**						
Date Sampled :	**April 4, 2018**	**Borehole No:**	**BH8**	**Sample No.:**	**SS2**			**Depth (m) :**	**0.6-1.2**
Sample Description :		**% Silt and Clay**	**61**	**% Sand**	**39**	**% Gravel**	**0**	**Figure :**	**41**
Sample Description :		**Sandy Clay (CL)**							

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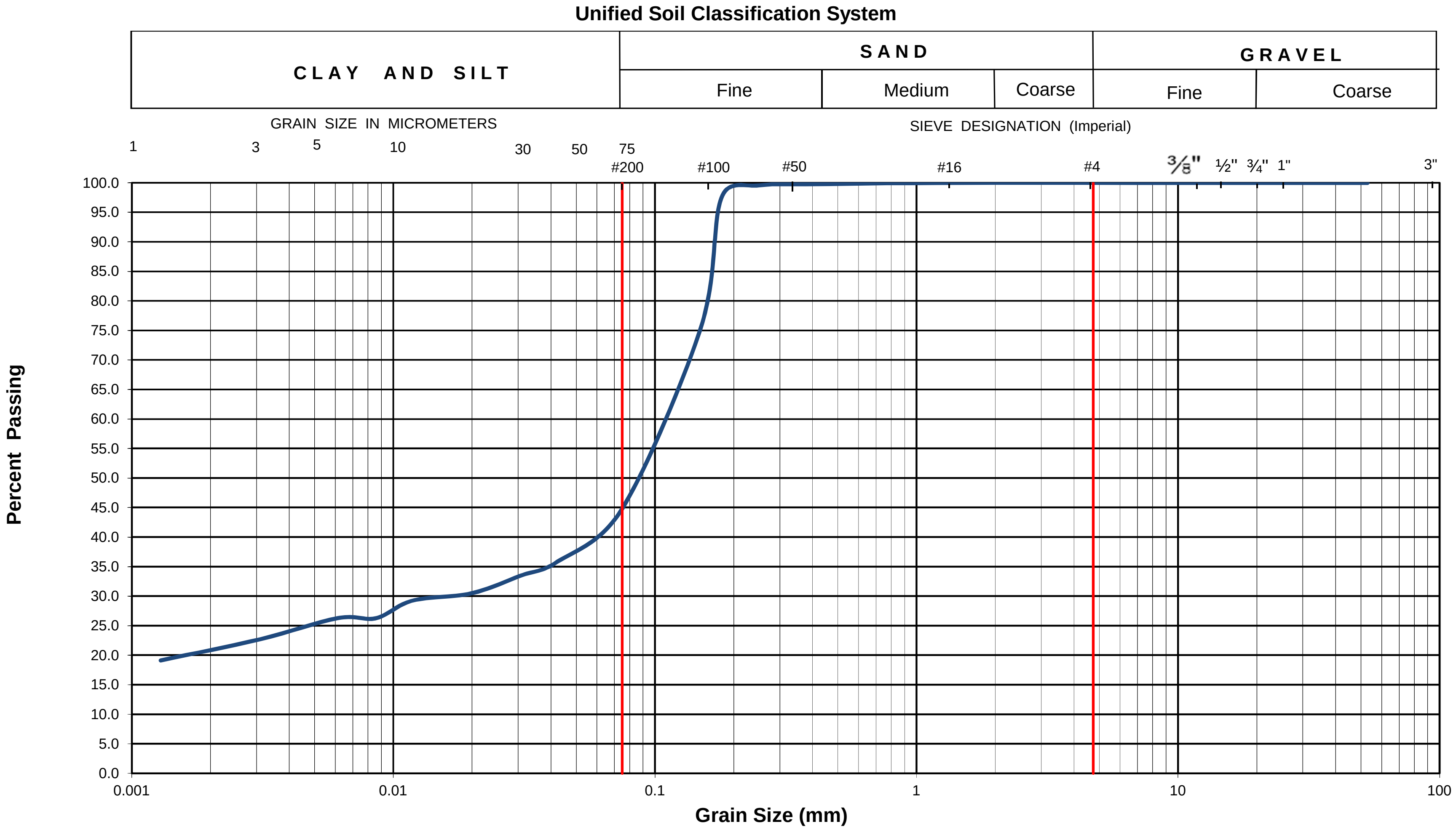
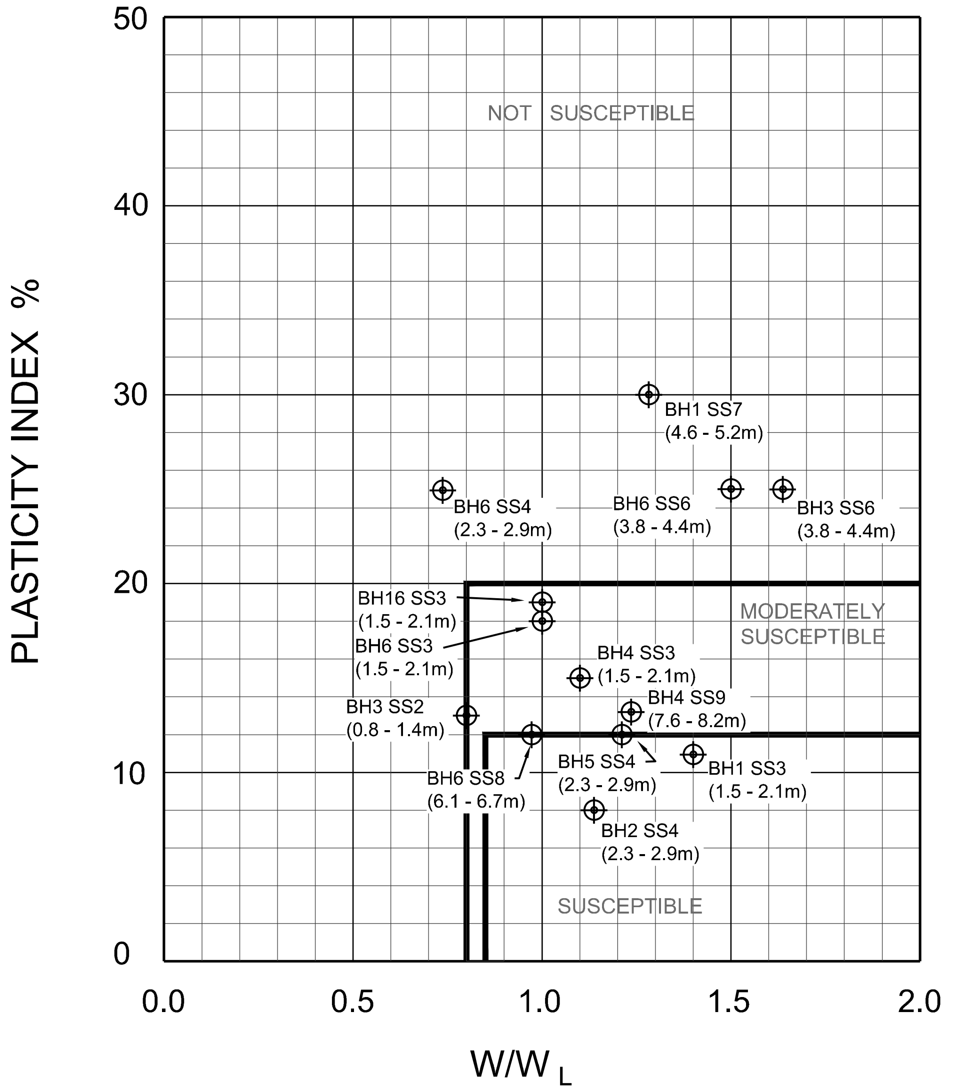
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	OTT-00245869-A0	Project Name :	Proposed New Riverside South Catholic Elementary School						
Client :	CECCE	Project Location :	Ralph Hennessy Avenue and Mount Nebo Way, Ottawa, ON.						
Date Sampled :	April 4, 2018	Borehole No:	BH13		Sample No.:	SS3		Depth (m) :	1.2-1.8
Sample Description :		% Silt and Clay	45	% Sand	55	% Gravel	0	Figure :	42
Sample Description :	Silty Clayey Sand (SC-SM)								



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

LEGEND

⊕	BH6 SS3 (1.5 - 2.1m)	BOREHOLE AND SAMPLE NO. SAMPLE DEPTH (m)

exp Services Inc.
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2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6
Canada

www.exp.com

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• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

scale N.T.S.
date JUNE 2018
drawn by M.N.

CLIENT: CONSEIL DES ECOLES CATHOLIQUES DU CENTRE-EST □CECCE□

TITLE: PROPOSED NEW RIVERSIDE SOUTH CATHOLIC ELEMENTARY SCHOOL

project no. OTT-00245869-A0

Figure 43

Filename: r:\240000\245000\245869-a0\245869-ge fig 34 li□uefaction assessment.dwg
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Appendix A – Site Photographs

Photograph No. 1

View of the site from the northwest corner facing east

Photograph No. 2

View of the site from the central part of the site facing southeast

Photograph No. 3

View of the site from the central part of the site facing west

Photograph No. 4

View of the site from the northwest corner of the site facing southeast

Photograph No. 5

View of the site from the northeast corner of the site facing northwest

Photograph No. 6

View of fill piles on site

Photograph No. 7

View of some debris located along the property line in the central west side of the site

Appendix B – Seismic Shear-wave Survey

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 1st, 2018

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

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**
Ralph Hennessy Avenue, Riverside South, Ottawa (ON)

[Project: OTT-00245869-A0]

Dear Madam,

Geophysics GPR International Inc. has been requested by **exp** Services Inc. to carry out seismic shear wave surveys on a property, located South-East of the intersection of Ralph Hennessy Avenue and Cambie Road, Riverside South, 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 24th, 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 a vacant field, at the South-East of the intersection of Ralph Hennessy Avenue and Cambie Road. The geophone spacing for the main spread was of 3 metres, using 24 geophones. A shorter seismic spread, with geophone spacing of 1 metre, was 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 approximately 9 to 10 metres deep (± 1 metre). Its seismic velocity was calculated between 2315 and 2435 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.

The calculated $\overline{V}_{S30}$ value is 478.8 m/s (cf. Table 1), corresponding to the Site Class “C”. However, very low to low seismic velocities were calculated for the clayey materials, from the surface to approximately 5 metres deep.

CONCLUSION

Geophysical surveys were carried out on a vacant field, located South-East of the intersection of Ralph Hennessy Avenue and Cambie Road, 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 479 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. It must be noted that very low seismic velocities were calculated for the unconsolidated materials between the surface and approximately 5 metres deep. A geotechnical assessment related to these materials should be realised, at least to verify if the $\overline{V}_{S30}$ Site Class can be considered.

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

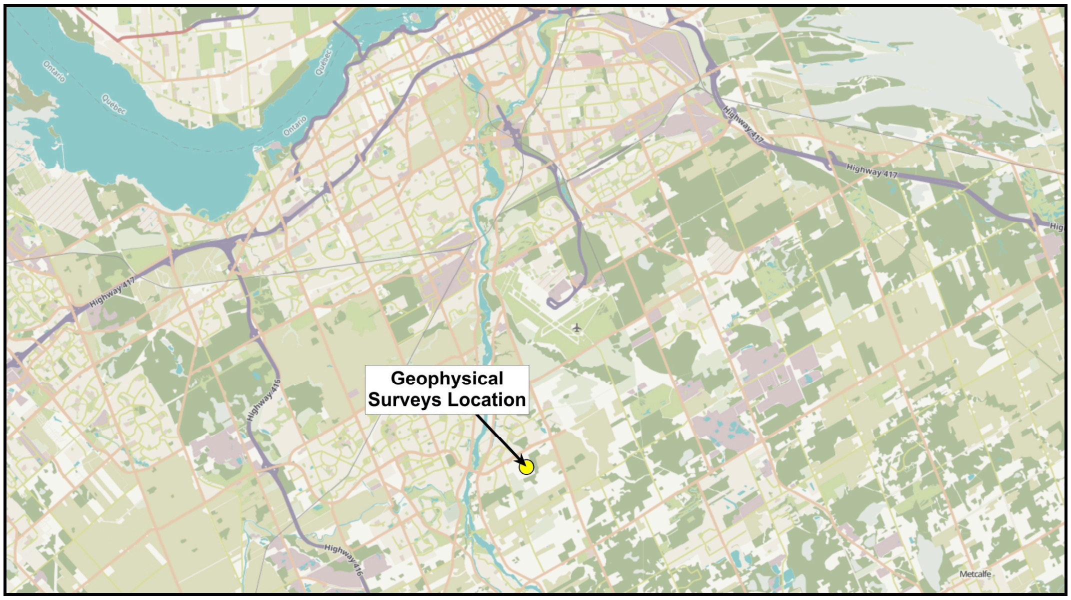


Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

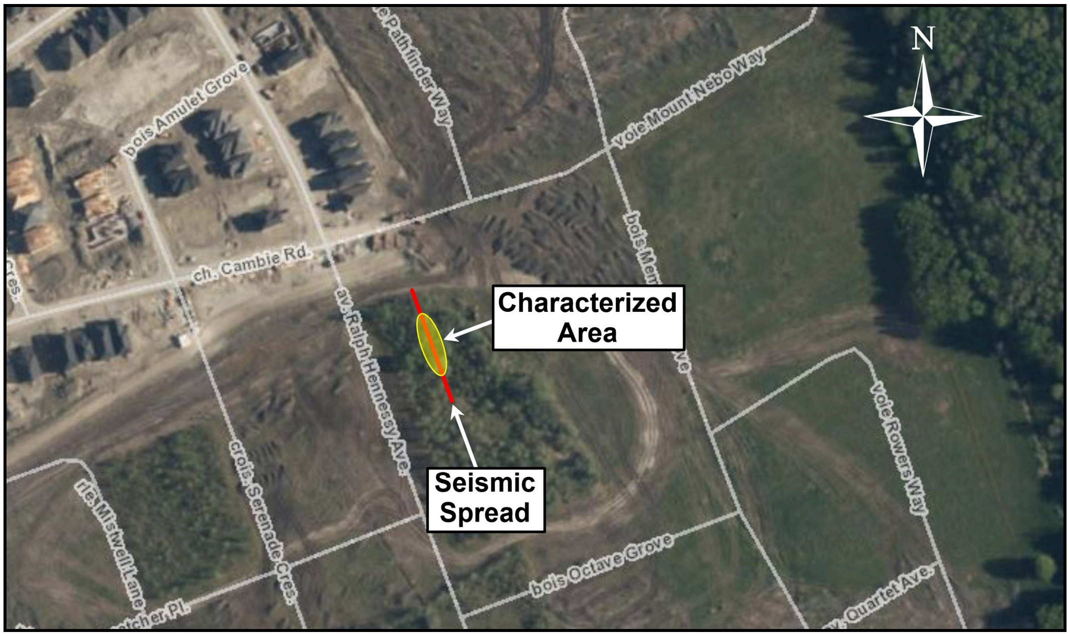


Figure 2: Location of the seismic spreads
(source: *geoOttawa*)

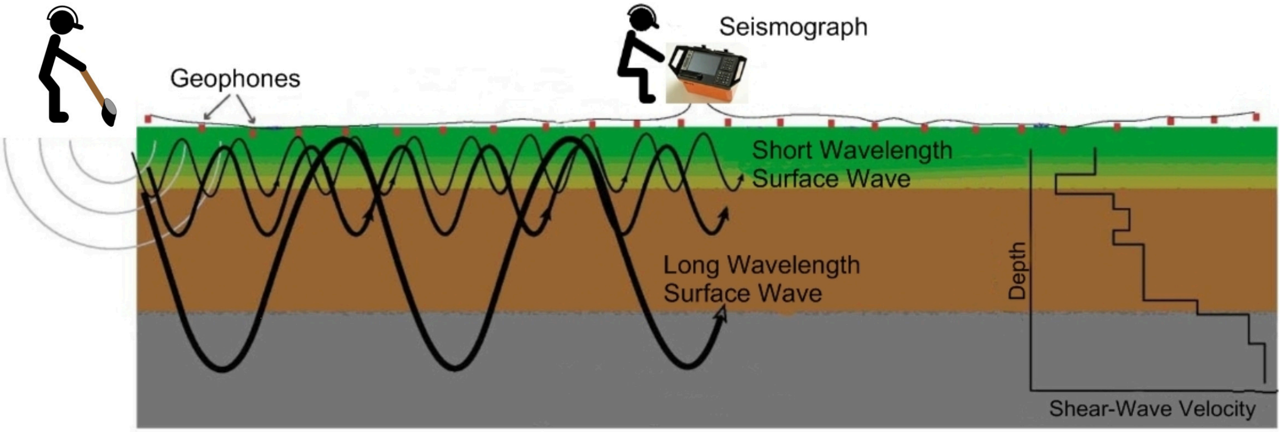
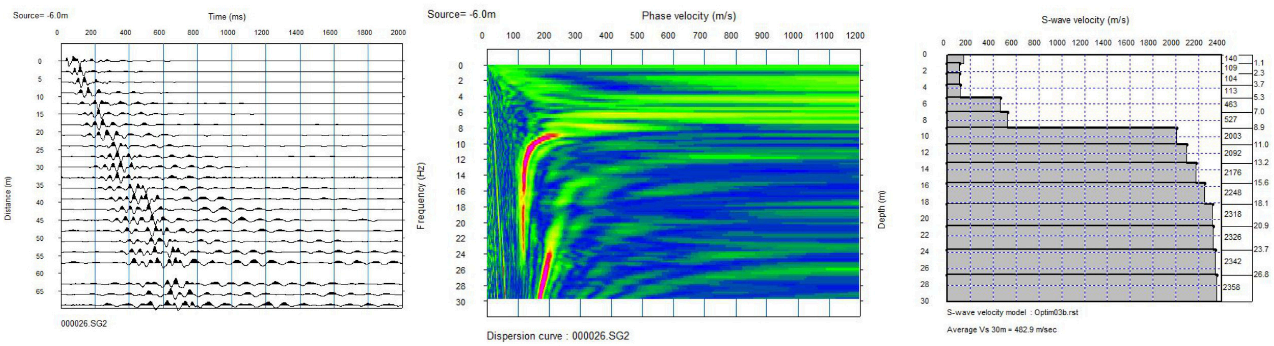


Figure 3: MASW Operating Principle

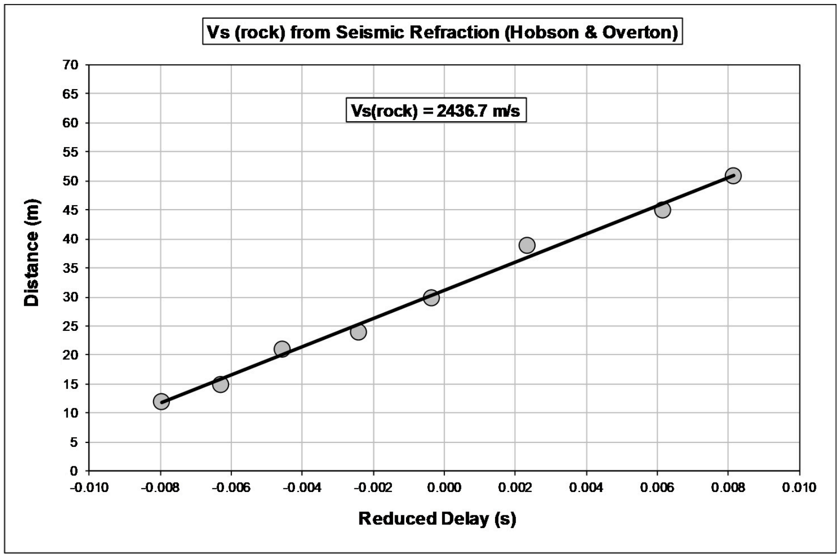


Raw Seismic Record

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



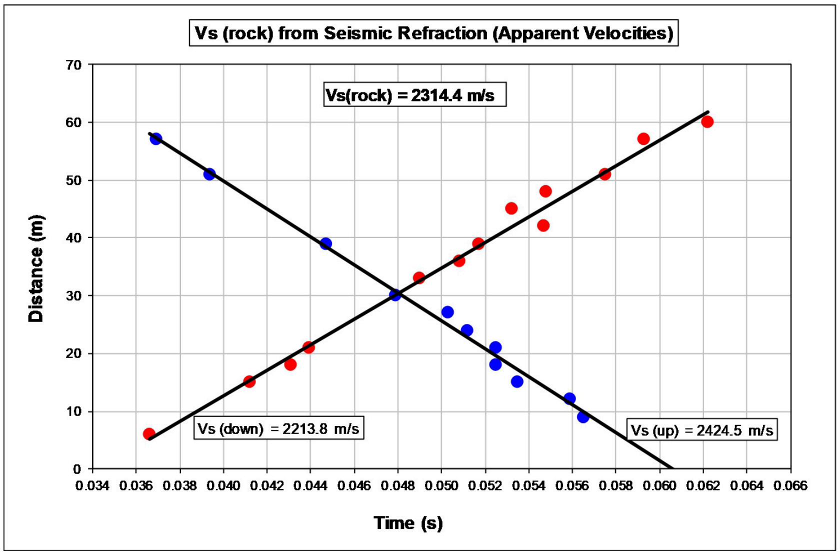


Figure 5: Rock V_S from Seismic Refraction

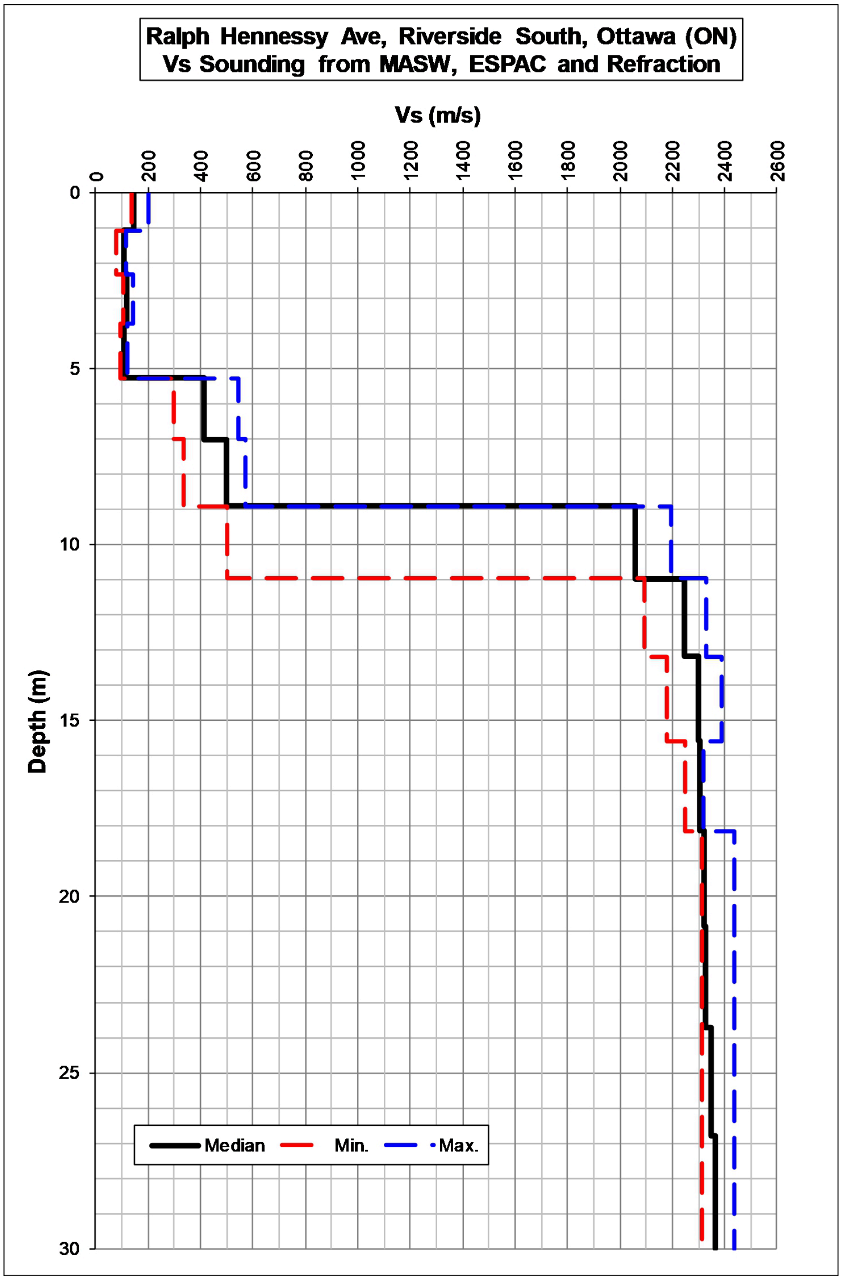


Figure 6: MASW Shear-Wave Velocities Sounding

TABLE 1
V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	**135.1**	**144.8**	202.7					
1.07	**78.4**	**108.0**	**112.6**	1.07	1.07	0.007401	0.007401	144.8
2.31	**104.7**	**119.6**	**140.3**	1.24	2.31	0.011446	0.018846	122.4
3.71	**93.6**	**105.3**	**120.8**	1.40	3.71	0.011713	0.030559	121.4
5.27	297.1	412.0	544.5	1.57	5.27	0.014875	0.045434	116.1
7.01	333.8	497.5	568.8	1.73	7.01	0.004201	0.049635	141.1
8.90	500.7	2057.7	2193.9	1.90	8.90	0.003810	0.053446	166.5
10.96	2092.8	2244.7	2328.0	2.06	10.96	0.001001	0.054447	201.3
13.19	2176.7	2301.1	2386.1	2.23	13.19	0.000991	0.055438	237.9
15.58	2248.4	2303.9	2316.7	2.39	15.58	0.001039	0.056477	275.8
18.13	2311.7	2319.7	2436.7	2.55	18.13	0.001109	0.057586	314.9
20.85	2314.4	2329.4	2436.7	2.72	20.85	0.001172	0.058758	354.9
23.74	2314.4	2345.8	2436.7	2.88	23.74	0.001238	0.059997	395.6
26.79	2314.4	2362.0	2436.7	3.05	26.79	0.001300	0.061297	437.0
30				3.21	30.00	0.001361	0.062658	478.8

V_{S30} (m/s)	**478.8**
Class	**C** (1)

(1): Geotechnical assessment should be undertaken for the low seismic velocities materials located from the surface to approximately 5 metres deep, at least, to verify if the Site Class from V_{S30} can be considered.

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

June 04, 2018

Site: 45.2716 N, 75.6801 W User File Reference: Ralph Hennessy Avenue and Mount Nebo Way, Ottaw

Requested by: , EXP Servics 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)**
0.454	0.530	**0.443**	0.336	**0.238**	**0.118**	**0.056**	**0.015**	**0.0054**	**0.283**	**0.197**

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)	0.042	0.146	0.247
Sa(0.1)	0.059	0.184	0.300
Sa(0.2)	0.053	0.159	0.255
Sa(0.3)	0.043	0.123	0.195
Sa(0.5)	0.030	0.087	0.138
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	0.031	0.100	0.163
PGV	0.021	0.067	0.110

References

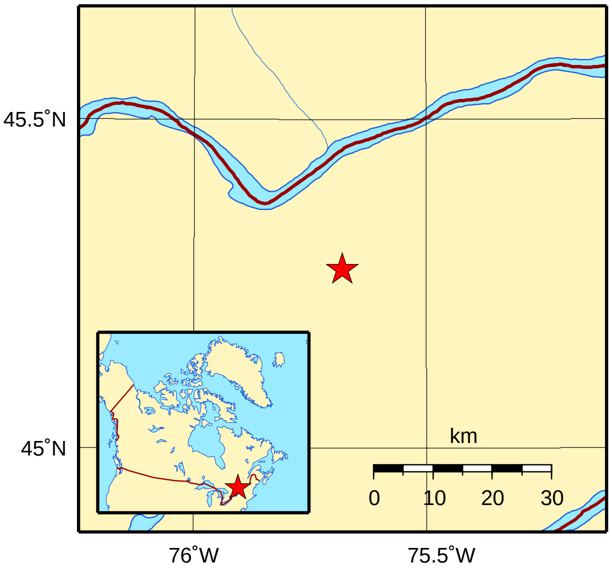
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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

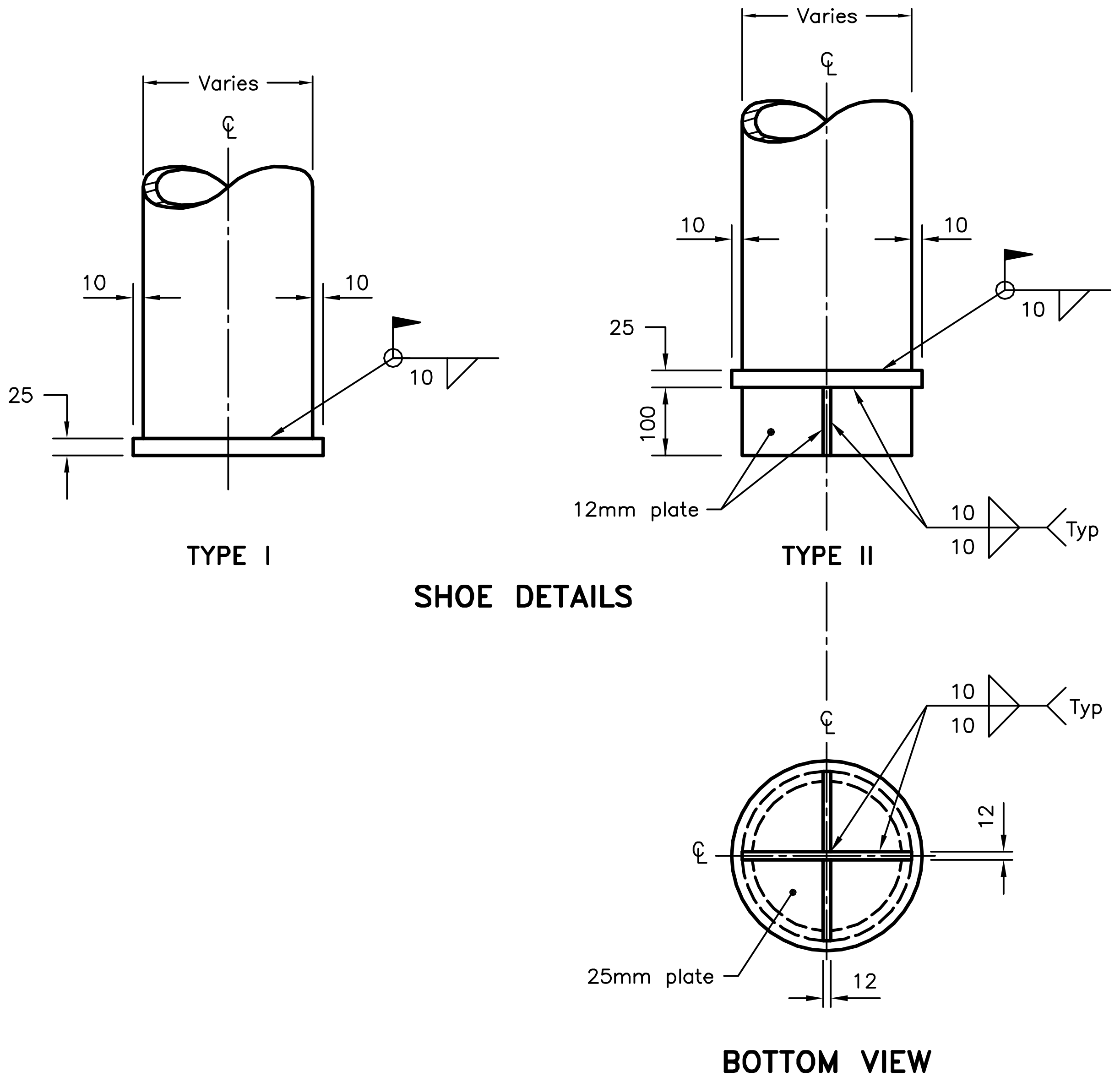
Aussi disponible en français



Natural Resources Canada Ressources naturelles Canada

Canada

Appendix D - Pile Driving Shoe



NOTES:

A Driving shoe Type I or II as specified.
B Welding shall be according to CSA W59.
C Steel plates shall be according to CSA G40.20/G40.21, Grade 300W.
D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
FOUNDATION PILES STEEL TUBE PILE DRIVING SHOE	OPSD 3001.100	

Appendix E - Certificate of Laboratory 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-00245869-AO**

AGAT WORK ORDER: **18Z327739**

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

DATE REPORTED: **Apr 16, 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.

Certificate of Analysis

AGAT WORK ORDER: 18Z327739

PROJECT: OTT-00245869-AO

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

CLIENT NAME: EXP SERVICES INC

ATTENTION TO: Susan Potyondy

SAMPLING SITE:Riverside South Elementary School

SAMPLED BY:exp

Inorganic Chemistry (Soil)

DATE RECEIVED: 2018-04-10

DATE REPORTED: 2018-04-16

			SAMPLE DESCRIPTION:	BH#2 SS3 5'-7'	BH#3 SS5 10'-12'	BH#6 SS3 5'-7'
			SAMPLE TYPE:	Soil	Soil	Soil
			DATE SAMPLED:	2018-04-03	2018-04-02	2018-04-02
Parameter	**Unit**	**G / S**	**RDL**	**9174559**	**9174560**	**9174561**
Chloride (2:1)	µg/g		2	4	3	3
Sulphate (2:1)	µg/g		2	42	104	84
pH (2:1)	pH Units		N/A	7.47	8.01	7.91

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

9174559-9174561 Chloride, Sulphate and pH 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

Quality Assurance

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-00245869-AO
SAMPLING SITE:Riverside South Elementary School

AGAT WORK ORDER: 18Z327739
ATTENTION TO: Susan Potyondy
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		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Inorganic Chemistry (Soil)															
Chloride (2:1)	9174559	9174559	4	4	NA	< 2	101%	70%	130%	101%	70%	130%	104%	70%	130%
Sulphate (2:1)	9174559	9174559	42	37	12.7%	< 2	95%	70%	130%	105%	70%	130%	112%	70%	130%
pH (2:1)	9174559	9174559	7.47	7.51	0.5%	N/A	101%	90%	110%	NA			NA		

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

Method Summary

CLIENT NAME: EXP SERVICES INC

PROJECT: OTT-00245869-AO

SAMPLING SITE:Riverside South Elementary School

AGAT WORK ORDER: 18Z327739

ATTENTION TO: Susan Potyondy

SAMPLED BY:exp

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

AGAT Laboratories

1 large

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

Laboratory Use Only

Work Order #: 18Z327739

Cooler Quantity: one

Arrival Temperatures: 23.1 | 23.1 | 23.0

LT- 3.1 | 5.5 | 6.5

Custody Seal Intact: ☐Yes ☐No ☐N/A

Notes: no ice ON ICE

Chain of Custody Record

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

Report Information:

Company: exp Services Inc. Ottawa
Contact: Susan Potyondy
Address: 2650 Queensview Dr. Unit 100
Ottawa K2B 8H6
Phone: 613-688-1899 Fax:
Reports to be sent to:
1. Email: susan.potyondy@exp.com
2. Email:

Regulatory Requirements: ☐ No Regulatory Requirement

(Please check all applicable boxes)

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

☐ Sewer Use
☐ Sanitary
☐ Storm
Region ____ Indicate One
☐ MISA

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

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

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

Turnaround Time (TAT) Required:

Regular TAT ☑ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

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

OR Date Required (Rush Surcharges May Apply):

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

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

Project Information:

Project: OTT-00245869-A0
Site Location: Riverside South Elementary School
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	☐ All Metals ☐ 153 Metals (excl. Hydrides) ☐ Hydride Metals ☐ 153 Metals (Incl. Hydrides)	ORPs: ☐ B-HWS ☐ Cl⁻ ☐ CN ☐ Cr⁶⁺ ☐ EC ☐ FOC ☐ Hg ☐ pH ☐ SAR	Full Metals Scan	Regulation/Custom Metals	Nutrients: ☐ TP ☐ NH₃ ☐ TKN ☐ NO₃ ☐ NO₂ ☐ NO₃+NO₂	Volatiles: ☐ VOC ☐ BTEX ☐ THM	PHCs F1 - F4	ABNs	PAHs	PCBs: ☐ Total ☐ Aroclors	Organochlorine Pesticides	TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	Sewer Use	pH	sulphate	chlorides
BH#2 SS3 5'-7'	Apr. 3		1	S																	✓	✓	✓
BH#3 SS5 10'-12'	Apr. 2		1	S																	✓	✓	✓
BH#6 SS3 5'-7'	Apr. 2		1	S																	✓	✓	✓

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Jeff MacMillan	Apr. 10/18		Ben Thelett	10-Apr-18	12h44
(BT) to FedEx	10-Apr-18	16h00	Avineet Minhas AM	Apr 11/18	8:51

Page 1 of 1

Nº: T 063377

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