16 EDGEWATER STREET, KANATA, ON
PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT

Project No.: CCO-22-0244

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October 2021

McINTOSH PERRY

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16 Edgewater Street, Kanata, ON

1.0 INTRODUCTION

McIntosh Perry (MP) was retained by McCluskey Group (the Client) to carry out foundation investigation for the proposed multi-storey buildings and stacked townhouses located at 16 Edgewater Street in Kanata, Ontario. Information regarding the revised basement floor elevations of the proposed underground parking levels were communicated by the client on October 4th, 2021 and the revised drawing A.100.Rev 1. Based on this information, it is indicated that this multi-storey building will consist of ten (10) storey (mid-rise) building over part of the P1 level basement, and two (2) storey townhouse blocks founded overlying part of the P1 level and P2 level basements (see Figs. 1b and 1c). The purpose of the investigation was to understand the subsurface conditions at the site by means of undertaking boreholes, field and laboratory testing, supplemented by perusing related background technical information in the public domain.The fieldwork was carried out between June 9 and 11, 2021, and comprised of eleven (11) boreholes advanced to a maximum depth of 12 m. Based on the geotechnical investigation, this report presents the factual findings and preliminary engineering recommendations.

This report provides anticipated geotechnical conditions influencing the design and construction of the proposed multi-storey residential buildings and townhouses, as well as recommendations for foundation design. Preliminary recommendations are offered based on the authors' interpretation of the subsurface investigation and test results. The readers are referred to Appendix A, Limitations of Report, which is an integral part of this document.

2.0 PROJECT UNDERSTANDING

It is understood that the Client intends to construct a multi-storey apartment building, 10 storeys above ground, with two underground parking levels. The first underground parking level will be constructed over the entire footprint of the property and a lower underground parking area is proposed at the east half of the property. There are also blocks of stacked townhouses proposed at the north and east boundaries. The block on the east side is nine (9) units and the block on the north side is thirteen (13) units. Proposed townhouses are to be supported on the parking structure.

A preliminary investigation by McIntosh Perry was completed for the site dated December 2018. Borehole locations were selected at the time of preliminary investigation based on the available project concepts. Additional boreholes were drilled in 2021 based on the new design. The geotechnical investigation dated in December 2018 is presented in Appendix H.

3.0 SITE DESCRIPTION

3.1 Site Geology

Based on the published physiography maps of the area (Ontario Geological Survey), the site is located within the Ottawa Valley Clay Plains. Surficial geology maps of southern Ontario indicate the proposed development is located within fine-textured glaciomarine deposits composed mainly of silt and clay, minor sand and gravel, surrounded by areas of coarse-textured glaciomarine deposits to the north and stone-poor, sandy silt to silty sand-textured till on Paleozoic terrain to the east.

Bedrock geology maps of southern Ontario indicate the bedrock formation at which the site is located is Limestone, dolostone, shale, arkose, and sandstone.

The Ottawa Valley between Pembroke and Hawkesbury, Ontario, consists of clay plains interrupted by ridges of rock or sand. It is naturally divided into two parts, above and below Ottawa, Ontario. Within the valley, the bedrock is further faulted causing some of the uplifted blocks to appear above the clay beds.

3.2 Existing Site Conditions

The property is located in a mixed commercial and industrial area with a residential subdivision at its east, separated by a chain-link fence and a line of mature trees. Currently the site is accessible from Edgewater Street through the parking lot of the adjacent commercial property that accommodates Tim Hortons and Wendy's. Boulders were used on the property line between both properties to prevent vehicular traffic on the property. A commercial property separated with a chain-link fence and a line of mature trees and bushes at the northwest. A ditch separates the site from Edgewater Street at its west.

The site topography is generally flat with no major changes in grade. Currently, the site covered with grass that is appeared to be regularly mowed.

The property limits are shown in Figure 2a and 2b, in Appendix B.

4.0 FIELD AND LABORATORY INVESTIGATIONS

4.1 Field Investigation

McIntosh Perry staff visited the site before the site investigation to mark out the proposed borehole locations. Utility clearance was carried out by Underground Service Locators (USL-1) on behalf of McIntosh Perry. Public and private utility authorities were informed, and all utility clearance documents were obtained before the commencement of drilling work.

At the time of investigation, no information was available about the location, size, and structural loads of the proposed development. The field investigation program aimed at investigating the subsurface stratigraphy across the site and to model the entire site stratigraphy. Field investigation included drilling eleven (11) new boreholes in addition to the five boreholes which were drilled in 2018.

The hydraulic powered rig used for drilling was owned and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario. Boreholes were advanced using hollow stem augers aided by a track-mounted CME 850 drill rig. Boreholes were advanced to a maximum drilling and coring depth of 12 m (El. 89.7 m) below the ground level in BH21-01. Soil samples were obtained at 0.75 m intervals in boreholes using a 51 mm outside diameter split spoon sampler following the Standard Penetration Test (SPT) procedure. The reported SPT testing conforms to ASTM D1586 – Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils. SPT testing alternated with in-situ measurement of the undrained shear strength (Su) of cohesive soils using the filed Vane Shear Test. The reported field vane shear testing conforms to ASTM D2573 – Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.

Dynamic cone penetration testing (DCPT) was undertaken with the standard SPT test hammer (63.5 kg) in BH21-11 to estimate the extent of soil layers and to infer the depth of bedrock or DCPT refusal.

The bedrock was cored and sampled to approximately 3.0 m in six (6) boreholes. Rock cores were obtained by diamond drilling and wireline tooling. Rock cores were retrieved in dual-wall NQ core barrel samplers to reduce the risk of mechanical breaks.

A monitoring well was installed in BH21-07. Details and location information of the well are provided in Section "4.3" and summarized in Tables 4-2.

The bedrock cores were sealed with holeplug and the boreholes were backfilled with auger cuttings and restored to the original surface level. A summary of borehole designations, location and depths is shown in Table 4.1. Borehole locations are shown in Figure 2a and 2b, included in Appendix B.

Table 4-1: Borehole Designations, Locations, and Depth

BH No.: Drilling Date	Coordinates (Geodetic) Lat (Long): degrees	Surface El. (m)	Drilling		Coring			Termination depth
			Depth (m)	El. (m)	Run (m)	Depth (m)	El. (m)	
21-01 June/09/2021	45.295308 / -75.894941	101.7	9.0	92.7	3.0	12	89.7	Hollow Stem Auger Bedrock cored (NQ)
21-02 June/09/2021	45.295376 / -75.895153	101.7	6.1	95.6	2.9	9.0	92.7	Hollow Stem Auger Bedrock cored (NQ)
21-03 June/09/2021	45.295566 / -75.895236	102.1	2.7	99.4	3.3	6.0	96.1	Hollow Stem Auger Bedrock cored (NQ)

21-04 June/10/2021	45.295546 / -75.895063	102.0	3.1	98.9	--	--	--	Hollow Stem Auger Inferred bedrock/boulders
21-05 June/9/2021	45.295453 / -75.894851	101.9	6.3	95.6	--	--	--	Hollow Stem Auger Inferred bedrock/boulders
21-06 June/10/2021	45.295491 / -75.894595	101.9	5.2	96.7	--	--	--	Hollow Stem Auger Inferred bedrock/boulders
21-07 June/10/2021	45.295679 / -75.894747	102.1	4.9	97.2	3.3	8.2	93.9	Hollow Stem Auger Bedrock cored (NQ)
21-08 June/10/2021	45.295849 / -75.894846	102.5	4.6	97.9	3.1	7.7	94.8	Hollow Stem Auger Bedrock cored (NQ)
21-09 June/11/2021	45.295653 / -75.894478	102.1	4.0	98.1	3.4	7.4	94.7	Hollow Stem Auger Bedrock cored (NQ)
21-10 June/11/2021	45.295841 / -75.894420	102.4	5.8	96.6	--	--	--	Hollow Stem Auger Inferred bedrock/boulders
21-11 June/11/2021	45.296023 / -75.894679	102.8	7.5	95.3	DCPT	11.1	91.7	Hollow Stem Auger DCPT refusal.

Field investigation, including drilling and sampling, were supervised on a full-time basis by McIntosh Perry technical staff. All boreholes were logged during the drilling progress. All samples were labeled by waterproof paper one by one as they were retrieved. All soil samples were preserved in double plastic bags to mitigate the risk of moisture loss during transportation to the geotechnical laboratory. Rock cores were laid and labeled in specialty boxes made for rock core transportation. The Rock Quality Designation (RQD) was measured for the first time in the field immediately after drilling to reduce the measurement errors caused by transportation induced damages to the rock cores.

4.2 Laboratory Investigations

Laboratory testing on representative SPT samples was performed at McIntosh Perry geotechnical lab and included grain size distribution analysis (ASTM D422/LS-702). The laboratory tests to determine index properties performed based on the American Society for Testing Materials (ASTM) test procedures.

Six (6) representative rock cores underwent uniaxial compressive strength (UCS) test to determine the UCS of the intact rock cores. The tests were performed at McIntosh Perry geotechnical lab, located in Ottawa, in accordance with the American Society for Testing Materials (ASTM D7012) test procedures.

The rest of the recovered soil samples and rock cores are stored in McIntosh Perry storage facility for one month after submission of the final report. Samples and rock cores will be disposed of after this time unless otherwise requested in writing by the Client.

Paracel Laboratories Ltd., in Ottawa, carried out chemical tests on two representative soil samples to determine the soil corrosivity characteristics. The chemical tests included corrosivity chemical tests (pH, Chloride, Sulphate, and Resistivity). Laboratory test results are included in Appendix E.

5.0 SUBSURFACE CONDITIONS

5.1 Subsurface Conditions

5.1.1 Overview

In general, the site stratigraphy within the property limits based on the borehole findings consists of topsoil underlain by a fill layer followed by native overburden overlying bedrock. The native overburden consists of four stratified layers of variable thickness across the site.

Below the fill, a layer silty clay/clayey silt was encountered and was found to be of firm to stiff consistency. This layer was observed to vary in thickness across the site from 0.4 m to 3.8 m (BH21-1).

This was followed by a variable deposit of clayey sand/silty sand of very loose to loose compactness, found in the majority, of the boreholes. This layer was observed to vary in thickness across the site from 0.0 m to 1.4 m (BH21-10 and BH21-11).

The cohesionless deposit was followed by a silty clay deposit of soft to firm consistency, in general.

This cohesive deposit was underlain by a variable till deposit. Bedrock underlies the till deposit which was cored and sampled in six (6) out of the eleven (11) boreholes.

The soils encountered during the investigation, together with the field and laboratory test results are shown on the Record of Borehole sheets included in Appendix C. Rock cores and uniaxial compressive strength of intact rock results are presented in Appendix D. Description of the strata encountered are given below. Subsurface characterization generally follows the Canadian Foundation Engineering Manual (CFEM) (2006). Characterization of the consistency of cohesive deposits follow the observations derived from the more reliable field shear vane testing in preference to the less reliable SPT ‘N’ blow counts for purposes of characterizing the consistency of cohesive deposits.

Section 5.1 discusses the 2021 borehole findings only. The findings of the 2018 borehole series will be incorporated in Section 6 (Discussion part of the report) onwards.

5.1.2 Topsoil

A thin layer of topsoil was encountered at the surface in most of the boreholes. The topsoil extends to an approximate depth of 25 mm to 75 mm below the ground surface.

5.1.3 Fill

Topsoil was underlain by fill encountered in all the boreholes. Fill thickness ranged between 0.6 m and 1 m with an average thickness of 0.8 m. The fill is mainly composed of sand and gravel, some silt to sandy and gravely silt with traces of clay, with presence of organic matter identified in four out of eleven boreholes. The fill was observed to be brown and dry.

SPT ‘N’ blow counts within the fill ranged between 13 and 49 indicative of a compact to dense nature. The average SPT ‘N’ blow count was 30 blows/300 mm.

5.1.4 Silty Clay/Clayey Silt

A silty clay/clayey silt deposit was observed in all the boreholes. This layer was observed to vary in thickness across the site from 0.4 m to 3.8 m (BH21-1).

In general, the layer is composed of different portions of silt and clay and minor portions of sand and gravel. It was observed to be brown with moisture content ranging from damp to moist. This cohesive deposit is underlain by a cohesionless deposit.

A representative sample, SS-04 from BH21-07, underwent Atterberg’s Limits and water content testing. The test results showed that the water content (wc) for the tested sample was 67% and the Atterberg limits were 51% for Liquid Limit, 30.3% for Plastic Limit and 21% for Plasticity Index. The deposit can be classified as silt of high plasticity (MH) as per USCS. A summary of Atterberg Limits for the tested sample from this layer is shown in Table 5-1.

Table 5-1: Atterberg’s Limits Summary – Silty Clay

Sample Tested	Atterberg Limits (%)			wc (%)	Remarks
	LL	PL	PI		
BH21-07 - SS-04	51	30	21	67	USCS Classification: MH Fig. 3

The SPT ‘N’ blow counts within the deposit range from 1 to 11 blows/300mm and based on the SPT results, the consistency of the deposit can be described as typically firm to stiff.

5.1.5 Clayey Sand/Silty Sand

A variable deposit of clayey sand/silty sand was observed, in the majority, of the boreholes. The layer thickness varies across the site and was observed to range between 0.5 m to 1.4 m in BH21-10 and BH21-11 (towards the east boundary). It was observed to be brown to grey with moisture content ranging from damp to wet. This deposit is underlain by a cohesive deposit.

Two representative samples, SS-03 and SS-04 from BH21-11, underwent grain size analysis testing and classified as silty sands. Two representative samples, SS-03 from BH21-05 and SS-03 from BH21-10, underwent Atterberg's Limits and water content testing. The test results showed that the water content for the tested samples was 30% on average. Based on the Atterberg results of 26% for Liquid Limit, 14% for Plastic Limit and 12% for Plasticity Index on average, the two specimens tested reflect a clayey sand. A summary of the grain size distribution and Atterberg's Limits for the tested samples from this layer is shown in Table 5-2.

Table 5-2: Grain Size Distribution and Atterberg's Limits Summary – Sandy Silt/Silty Sand

Sample Tested	Size Fraction (%)				Atterberg Limits (%)			wc (%)	Remarks
	Gravel	Sand	Silt	Clay	LL	PL	PI		
BH21-11/SS-03	10	65	18	7					Fig. 4
BH21-11/SS-04	1	65	25	9					
BH21-05/SS-03	--	--	--	--	27	14	13	28	Fig. 5
BH21-10/SS-03	--	--	--	--	24	14	10	32	Fig. 5

This deposit, in general, was observed to be of very loose to loose compactness, with SPT 'N' blow counts ranging between 2 and 7 blows/300mm.

5.1.6 Silty Clay

A silty clay layer was observed below the cohesionless deposit in most boreholes across the site. The layer thickness ranges between 0.0 m in BH21-07 to 3.0 m in BH21-11. This deposit was not observed in BH21-07 (see Fig. 2a). Along the east side of the property, the layer was observed to be thickest, 2.9 m (BH21-10) to 3.0 m (BH21-11). It was observed to be brown to grey with moisture content ranging from moist to wet. The silty clay was underlain by a till deposit.

Five representative samples underwent grain size analysis testing, and the grain size distribution results are shown in Table 5.3 and Fig. 6. Three representative samples, listed in Table 5-3, underwent Atterberg's Limits and water content testing. The test results showed that the water content for the tested samples was 57% on average, and the limits for silty clay was 44% Liquid Limit, 19% Plastic Limit and 23% Plasticity Index on average. In general, this deposit can be classified as silty clay of intermediate plasticity (CI) based on the plasticity chart (see Fig. 7). However, BH21-08/SS-05 is classified as a sandy clay.

Table 5-3: Grain Size Distribution and Atterberg Limits Summary – Silty Clay

Sample Tested	Size Fraction (%)				Atterberg Limits (%)			wc (%)	Remarks
	Gravel	Sand	Silt	Clay	LL	PL	PI		
BH21-01/SS-04	0	15	54	31	--	--	--	--	Fig. 6
BH21-04/SS-04	0	11	50	39	--	--	--	--	
BH21-11/SS-06	1	3	45	52	--	--	--	--	
BH21-08/SS-05	3	36	34	24					
BH21-06/SS-04	0	13	47	40	47	20	26	59	Fig. 6 & Fig. 7
BH21-01/SS-06	--	--	--	--	39	19	20	54	Fig. 7
BH21-02/SS-05	--	--	--	--	45	18	27	58	

The SPT 'N' values recorded within the deposit range from 0 to 3 blows/300mm. The SPT test was alternated with field vane shear tests to measure the undrained shear strength of the deposit. The undrained shear strength within the layer ranges from 25 kPa to 85 kPa, based on the FSV. Accordingly, the consistency of the silty clay deposit can be described as soft to stiff. The layer with sensitivity values ranging between 5 and 10, can be classified as low sensitivity according to CFEM.

5.1.7 Till

A till layer was observed in all the boreholes. The layer thickness ranges between 0.3 m in BH21-04, to 2.9 m in BH21-01.

The till layer is composed of different portions of sand and gravel with silt and clay with presence of boulders and cobbles over 50% of the boreholes. It was observed to be brown to grey and wet. The till is underlain by bedrock or inferred bedrock/boulders, within the depths investigated.

Two representative samples, SS-07 from BH21-02 and SS-06 from BH21-06, underwent grain size analysis testing. A summary of the grain size distribution for the tested samples from this layer is shown in Table 5-4.

Table 5-4: Grain Size Distribution Limits Summary – Till

Sample Tested	Size Fraction (%)				Remarks
	Gravel	Sand	Silt	Clay	
BH21-02/SS-07	15	43	33	9	Fig. 8
BH21-05/SS-06	14	38	33	15	

The till in general was observed to be loose to compact with SPT 'N' blow counts range between 3 and 22 blows/300mm. Higher SPT 'N' blow counts and sometimes spoon refusal were encountered in the till layer which is possibly due to presence of cobbles and boulders.

5.1.8 Bedrock

Below the till, bedrock was cored in six boreholes with cored depths ranging from 2.9 m to 3.4 m, with the elevation of top of the cored bedrock ranging between El. 99.4 m to El. 92.7 m, almost a 7.0 m variation (see Fig. 2b). The bedrock and inferred bedrock/boulders elevations were observed to vary across the site. The DCPT refusal in BH 21-11 was met at an elevation of 91.7 m. Hence, the top of bedrock is at least 11.1 m deep at this location.

As mentioned, bedrock was cored and confirmed in six boreholes during the current investigation, and further, in two boreholes in the 2018 preliminary investigation. Based on the retrieved samples and rock core quality designation (RQD), the rock was identified as highly weathered Limestone and the rock mass can be classified as fair to excellent quality as per CFEM Table 3.10. Six (6) selected rock cores were tested for uniaxial compressive strength (UCS), and the results are shown in Table 5-5. Based on the intact rock strengths, the intact rock can be classified as strong to very strong as per CFEM Table 3.5. Based on the Total Core Recoveries reported (TCR%), with an average of 92% and a minimum of 72% based on 22 TCR measurements, there is hardly any evidence for the presence of solution cavities within the explored bedrock in six boreholes.

One rock core from BH21-09 exhibited failure pattern Type 1 "Reasonably well-formed cones on both ends". Microcracks were observed throughout the rock core from BH21-09. Two rock cores exhibited failure pattern Type 2 "Well-formed cone on one end, vertical cracks running through", and three rock cores exhibited failure pattern Type 4 "Diagonal fracture with no to slight cracking through ends" per ASTM D7012 Method C. Rock core photo logs are shown in Appendix D.

Table 5-5: Uniaxial Compressive Strengths - Rock Cores

Borehole	Rock Core	Core Depth (m)	Core El. (m)	TCR (%)	RQD (%)	UCS (MPa)
BH21-01	RC-12	10.9 to 11.0	90.8	100,100	65	197.3
BH21-02	RC-9	6.8 to 6.9	94.9	100,100	67	112.0
BH21-03	RC-5	3.8 to 3.9	98.3	100,100	70	109.4
BH21-07	RC-9	5.8 to 5.9	96.3	4 Nos; each 100	90	118.5
BH21-08	RC-7	5.2 to 5.3	97.3	95,72,100	77	72.8
BH21-09	RC-8	4.9 to 5.0	97.2	100,100,100	59	88.1

5.2 Groundwater

Groundwater levels were observed during and upon completion of drilling in the open boreholes, except for BH21-03, and 21-08 due to presence of excess water during rock coring. Also, in BH21-10, groundwater level in open borehole was not measured since the drillers added water to facilitate drilling. The groundwater level was observed in open boreholes in BH21-04, 21-05, and 21-11. In BH21-01, 21-04, 21-05, 21-06, and 21-09, signs of groundwater were observed on the retrieved SPT spoon samplers and in the soil samples.

One monitoring well was installed in BH21-07 in the current investigation and its assembly is shown on the borehole log records. In addition, the groundwater was observed in open boreholes or in auger, in eight (8) boreholes out of eleven (11).

Groundwater levels were measured in all monitoring wells (4 Nos) on August 6, 2021, one from the current investigation and three other wells that were installed in the 2018 preliminary site investigation. The groundwater levels measured during the current investigation are presented in Table 5-6.

The groundwater level may be expected to fluctuate due to extreme weather events and seasonal changes.

Table 5-6: Groundwater Levels in Boreholes and Monitoring Wells

BH No.: Drilling Date	Ground Surface El. (m)	GW in Open BH		Borehole Bottom		Monitoring Date	GW in Well		Remarks
		Depth (m)	El. (m)	Depth (m)	El. (m)	From	Depth (m)	El. (m)	
BH21-01 June/09/2021	101.7	3.0	98.7	12	89.7	--	--	--	
BH21-02 June/09/2021	101.7	1.9	99.8	9.0	92.7	--	--	--	
BH21-04 June/10/2021	102.0	2.3 3.0	99.7 99.0	3.1	98.9	--	--	--	
BH21-05 June/09/2021	101.9	1.1 3.0	100.8 98.9	6.3	95.6	--	--	--	
BH21-06 June/11/2021	101.9	3.7	98.2	5.2	96.7	--	--	--	
BH21-07 June/10/2021	102.1	2.6	99.5	7.0	75.1	Aug/06/2021	1.7	100.4	Sub-Artesian, i.e., above the filter pack zone
BH21-09 June/11/2021	102.1	3.8	98.3	7.4	94.7	--	--	--	
BH21-11 June/11/2021	102.8	2.6	100.2	7.5	95.3	--	--	--	

BH18-01 Dec./13/2018	102.3	--	--	4.6	97.7	June/11/2021	1.7	100.6	
						Aug/06/2021	1.7	100.6	
BH18-02 Dec./13/2018	102.4	--	--	6.1	96.3	June/11/2021	1.2	101.2	Sub-Artesian, i.e., above the filter pack zone
						Aug/06/2021	1.3	101.1	
MW18-03 Dec./13/2018	102.9	--	--	9.0	93.9	June/11/2021	1.3	101.6	
						Aug/06/2021	1.3	101.6	

5.3 Chemical Analysis

The chemical tests were conducted by Paracel Laboratories in Ottawa, ON, to determine the resistivity, pH, sulphate and chloride content of two representative soil samples collected from two different boreholes. The samples were chosen from within the estimated foundation depths and are shown in Table 5-7:

Table 5-7: Soil Chemical Analysis Results

Road Name	Sample	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (Ohm-cm)
BH21-02	SS-05	3.0 – 3.6	7.7	0.0074	0.0141	2380
BH21-10	SS-05	3.0 – 3.6	7.92	0.0471	0.0012	1520

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

This section of the report provides general and project-specific geotechnical guidelines for the design and construction of the proposed development. It is recalled as discussed under Section 1.0, that the Client intends to construct a multi-storey building that will consist of ten (10) storey (mid-rise) building over part of the P1 level basement, and two (2) storey townhouse blocks founded overlying part of the P1 level and P2 level basements (see Figs. 1b and 1c). Information regarding the revised basement floor elevations of the proposed underground parking levels were communicated by the client on October 4th, 2021. However, information on the elevations of elevator pit levels, etc., were unavailable at the time of compiling this report. The P1 underground parking level will be constructed over the entire construction footprint of the property and the lower P2 underground parking area is proposed over approximately the east half of the property. The current geotechnical investigation, discussed herein, has revealed the crucial role of geological and hydrogeological factors that have a significant influence on the geotechnical aspects of the design and construction of the building complex. It has brought to focus the geotechnical areas where the current understanding has important gaps and recommendations will be given of measures to address such issues. Consequently, the provided recommendations in this report are preliminary, and are based on the interpretation of the subsurface conditions and our understanding of project details current at this stage.

The foundation details of the existing townhouses to the east are unknown but they are expected to be on shallow foundations as typical for such buildings as founded in Ottawa. It is understood that only limited grade raise will be required outside the proposed building footprint.

The provided recommendations are only intended for the defined limits and the scope of the project. These recommendations shall not be used without the consent of the authors for engineering applications other than those specified in this scope of work. The readers are referred to Limitations of Report included in Appendix A.

The design team should satisfy themselves of the adequacy of the information presented in this report for their intended design. The constructors must make their own interpretation of the provided factual information as it affects their construction equipment, efficiency, schedule, and the safety of their staff.

The preliminary foundation recommendations presented in this section have been developed following Part 4 of the 2012 Ontario Building Code (OBC).

The discussion in Section 6 of the report will draw upon the findings from the current 2021 borehole investigation (BH 21-01 to BH21-11, 11 boreholes) as well as from the 2018 borehole investigation (BH 18-1 to BH18-5, 5 boreholes). Regarding the previous 2018 borehole investigation, in the absence of any laboratory testing, reliance will be made only of the field test results, as for quantitative geotechnical information.

6.2 Geotechnical Characterization

6.2.1 Overview

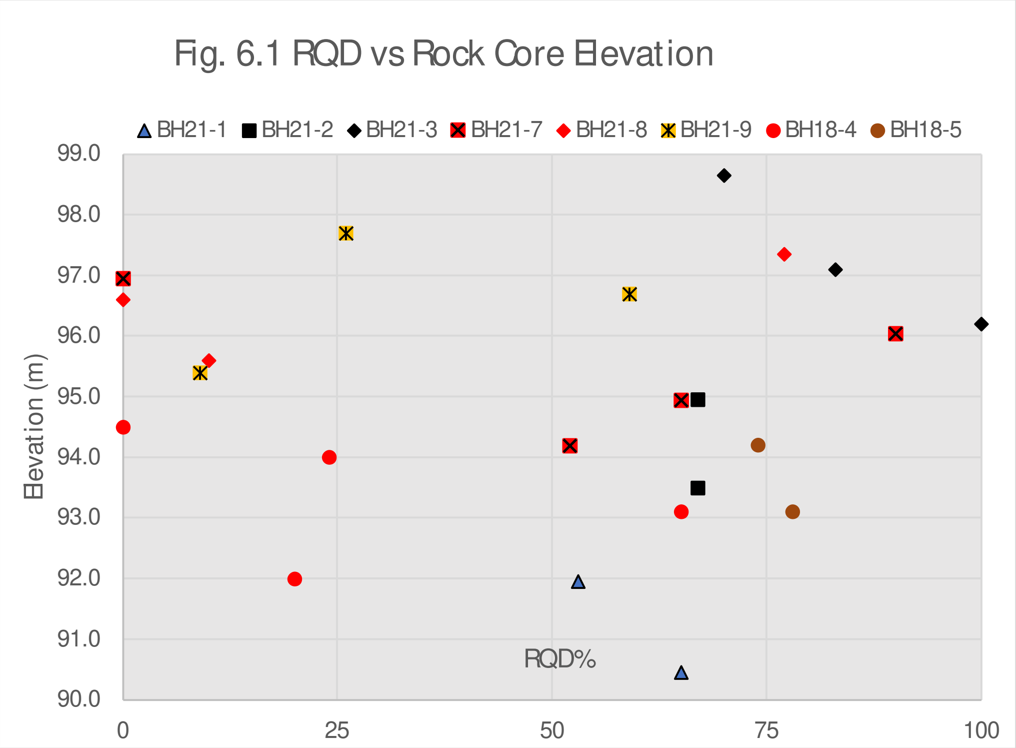
In general, the site stratigraphy within the property consists of topsoil underlain by a fill layer followed by native overburden overlying bedrock. The native overburden consists of four stratified layers of variable thickness across the site. Below the fill, a layer silty clay/clayey silt deposit was encountered followed by a variable cohesionless deposit of clayey sand/silty sand. The cohesionless deposit is underlain by a silty clay and was followed by a till deposit. Bedrock was confirmed underlying the till at the borehole locations, where bedrock was cored.

The ground surface elevation of the site at the borehole locations varied from El. 102.8 m to El. 101.7 m, i.e., relatively flat. The confirmed thickness of the intercepted overburden, however, ranged from 2.7 m to 9.0 m based on six boreholes where bedrock was cored. In BH 18-3 and BH21-11 (these two boreholes are in very close proximity to each other), the overburden was advanced to 13.2 m (El. 89.8 m) and 11.1 m (El. 91.7 m), respectively, before reaching DCPT refusal.

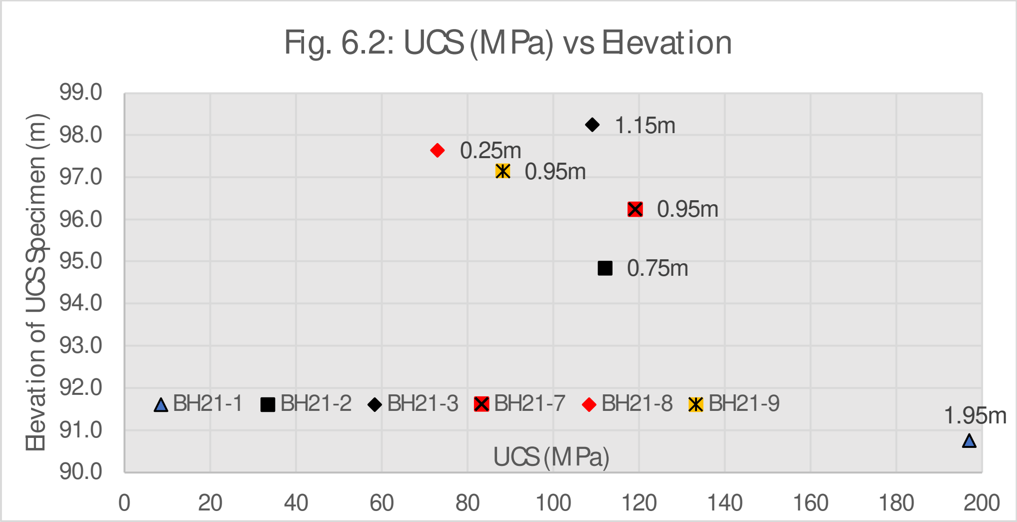
In general, the intercepted native overburden comprised of an upper cohesive deposit of typically firm to stiff consistency. A very loose to loose, variable cohesionless deposit was intercepted underlying the upper cohesive deposit, in the majority, of the boreholes. The cohesionless deposit was followed by a lower cohesive deposit comprising primarily, silty clay, typically of soft to stiff consistency. The consistency of the cohesive deposits is based upon observations from the field shear vane testing reported in Section 5. The lower cohesive deposit was underlain by a till deposit with the underlying bedrock either cored or inferred. Out of the total of 16 boreholes, in seven of the boreholes, SPT blow counts of 0 or "WOH" (weight of hammer) recordings were observed, predominantly in the lower cohesive deposit and a few such observation within the till deposit.

As structure foundations and temporary shoring embedment will be into bedrock, understanding of the rock mass properties is important for addressing both design and construction issues. Cored top of bedrock levels across the site were found to be very variable, i.e., ranging in depth from 2.7 m to 9.0 m below existing ground surface based on eight boreholes where bedrock was cored. In terms of elevations, the elevation of top of the cored bedrock ranged between El. 99.4 m to El. 92.7 m, almost a 7.0 m variation, in eight cored boreholes between the two series of investigations. It is important to note that at the northeast corner of the site (based on BH21-11 and BH18-03 locations) bedrock will be deeper than 13.2 m (El. 89.8 m) (See Fig. 2).

The measured RQD (a measure of the degree of fracturing of the rock mass) values of the subject rock core specimens did not show a trend for less jointed rock, i.e., higher RQD values either with decrease of elevation, or with depth below top of bedrock for a given borehole, as seen in Figure 6.1.



The UCS of the bedrock specimens tested, however, show an increasing trend of intact strength with elevation (also with depth), as seen in Fig. 6.2. Rock coring was undertaken up to 3.2 m depth below top of bedrock but as seen in Fig. 6.2, the UCS testing has covered only up to about 2 m depth. Another important caveat to note is the average RQD of the entire drilled bedrock was 52%, i.e., more fractured (based on 17 specimens) but the average RQD of the tested UCS samples was 71%, i.e., less fractured (based on 6 specimens). The minimum geometry requirements of the UCS specimens dictate the testing to be confined to less fractured bedrock and hence this explains the apparent anomaly. In view of this, the reported UCS strengths are likely to be an upper bound on the intact strengths.



While the trend of increasing UCS with elevation/depth is a positive outcome, the effective deformation modulus (stiffness) of a rock mass depends besides the intact rock modulus (which is related to the intact rock UCS) on the RQD, which is related to the fracture deformation modulus, i.e., low RQD implies more fractured rock mass and hence the rock mass is of lower stiffness. In this context, we have seen from Fig. 6.1 that a trend of decreasing fracture, i.e., lower RQD, with elevation is not apparent.

The measured stabilized groundwater levels indicate shallow groundwater (between 1.0 m to 2.0 m below the existing ground surface) and indicative of sub-artesian conditions. It is seen that low RQD values, i.e., indicative of high degree of fracturing, persists with depth. However, what is not known for sure is if the overburden groundwater regime is hydraulically connected with the fractured rock within the depths of foundation interest. Any hydraulic connection can have a significant impact on construction dewatering, design of foundations and slabs-on-grade.

6.2.2 Bedrock Topography

To better understand the bedrock and inferred bedrock/boulder topography, Figure 6-3 was established based on data from borehole records. The site was divided into three sections: A-A on the north side, B-B in the middle of the site from west to east, and C-C at the south. The bedrock and inferred bedrock/boulders elevations were plotted against the corresponding boreholes. The nature of the variation of top of bedrock levels between boreholes are unknown, but for simplicity the inter-borehole variation has been shown connected by straight line segments. These top of bedrock elevational variations are shown in Figs. 6.4 to 6.6.

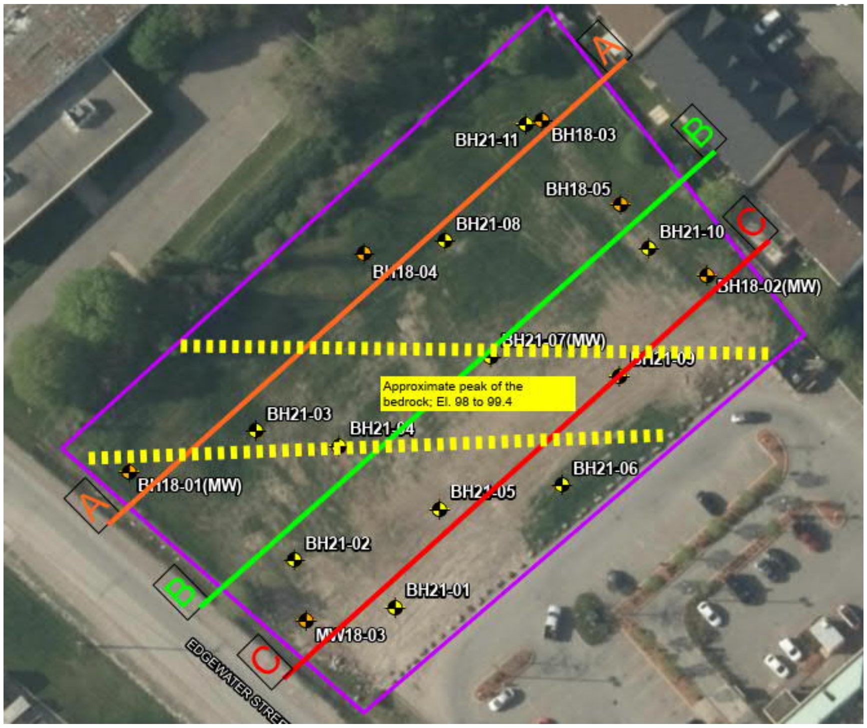


Figure 6-3: Cross -Section locations

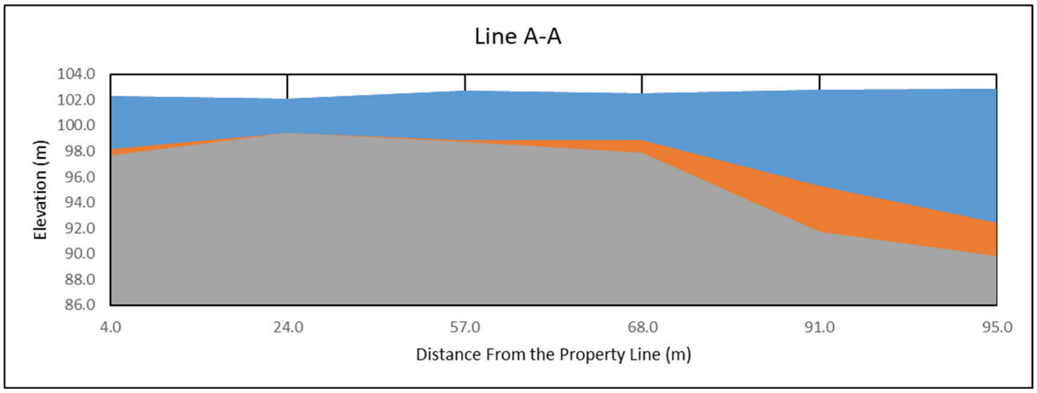


Figure 6-4: Cross-section A-A

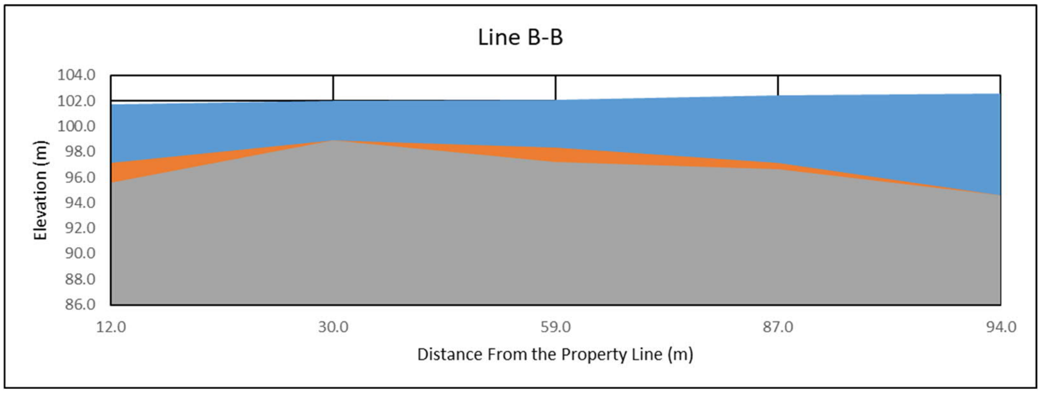


Figure 6-5: Cross-section B-B

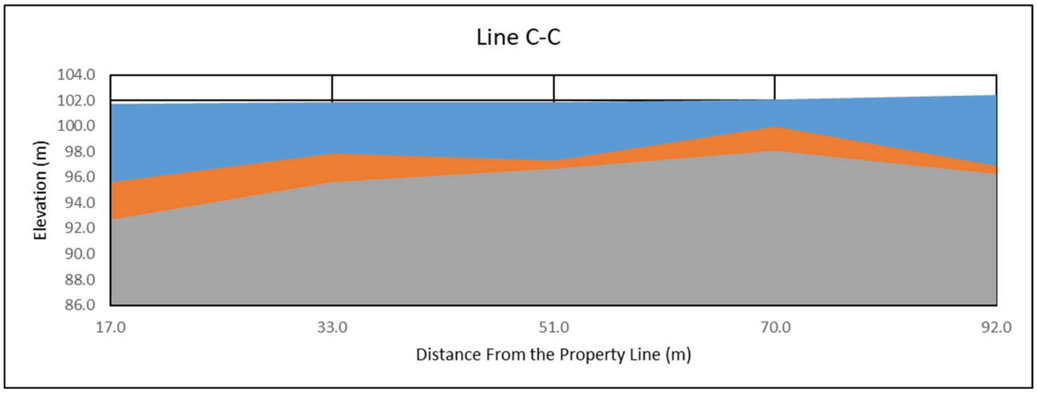


Figure 6-6: Cross-section C-C

6.2.3 Geotechnical Model

Based on the all the available information from the geotechnical investigation and our engineering judgement and experience, the following geotechnical model has been developed to address the various design issues.

Table 6-1: Geotechnical Model

Geotechnical Parameter		Fill	Silty Clay / Clayey Silt	Clayey Sand / Silty Sand	Silty Clay	Till	Bedrock
Unit Weight (γ) kN/m^3	Above groundwater	20.0	18.0	18.0	18.0	20.0	26
	Below groundwater (Submerged)	8.2	8.2	8.2	8.2	10.2	16
Plasticity Index (PI) (average)		N/A	21	13	27	N/A	N/A
Moisture Content %		N/A	67	32	59	N/A	-

Undrained Shear Strength $(S_u)_{FSV}$ kPa	N/A	N/A	N/A	30	N/A	
Clay Sensitivity – based on FSV (average)	N/A	N/A	N/A	10	N/A	
Effective Angle of Internal Friction (ϕ') degrees	28	29	31	28	30	
Effective Cohesion Intercept (kPa)	0	0	0	0	0	
Intact Rock: UCS (MPa); E (GPa); Poisson's Ratio, ν	N/A	N/A	N/A	N/A	N/A	70; 21; 0.25
Estimated Rock Mass Parameters: GSI, RMR; E_m (rock mass stiffness)	N/A	N/A	N/A	N/A	N/A	45, 50; 0.9 GPa

Notes:

1. N/A - Not Applicable
2. Lateral earth pressure coefficients assume horizontal backfill, level ground in front of the wall toe; neglect wall friction
3. For design purposes, in the absence of any groundwater dewatering/depressurization, assume the groundwater level at the existing ground surface. The earth pressure coefficients are only to calculate lateral backfill pressure. Any surcharge effects due to static loads such as stockpiles and moving machinery loading must be accounted for, based on the zone of influence (ZOI). At the minimum, the ZOI should be equal to the depth of excavation for surcharging effects
4. See Section 6.5 for limitations on the reliance on passive earth pressure resistance in bedrock for shoring.

6.2.4 Frost Depth & Frost Classification

The frost depth for the project site as per MTO OPSD 3090.101 is indicated as 1.8 m. Underlying the fill, the silty clay/clayey silt deposit has been identified up to 2.3 m thickness. Based on the PI, the material can be classified as a MH material or a borderline MH/CH material. As a potential MH/CH deposit, based on Table 13.1 of the CFEM, the material classifies as belonging to the F3/F4 frost group for frost design. The implications of this frost-heave potential will be discussed under temporary shoring since the structural foundations and slab on grade will be on a much lower elevation, well below this deposit and founded on bedrock or concrete.

6.2.5 Seismic Site Classification

Seismic site classification is completed based on OBC 2012 Section 4.1.8.4 and Table 4.1.8.4.A. This classification system is based on the average soil properties in the upper 30 m and accounts for site-specific shear wave velocity, standard penetration resistance, and plasticity parameters of cohesive soils.

Selected spectral responses in the general vicinity of the site for 2% chance of exceedance in 50 years (2475-year return period) are as indicated in Table 6-2, and in Appendix F.

Table 6-2: Selected Seismic Spectral Responses (2% in 50 years) – NRCan 2010

$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	PGA
0.619	0.299	0.134	0.045	0.316

Based on the subsurface conditions and field shear vane and SPT values, provided that the boundary zones of the shear walls and all column loads are extended to and supported on bedrock, using caissons where bedrock is deeper or on mass concrete founded on bedrock, then the site can be classified as Seismic Site Class (B) for design of foundations for axial vertical loading. Otherwise, for all lateral earth pressure issues related to basement walls and associated interactions, Site Class D would be applicable.

6.2.6 Overall Implications for Design and Construction based on Ground Characterization

- The overburden has been shown to be of variable thickness, saturated and not competent. It cannot be relied upon to carry any structural loads from the proposed building complex. Hence the building loads need to be carried on/within bedrock. The depth to top of bedrock is variable from 2.7 m to at least 13.2 m (towards the northeast corner of the site) based on boreholes. It is expected that significant bedrock excavation will be required to reach the P1 only and P1 and P2 segments of the building complex
- In view of the high ground water levels, the weak overburden cannot be relied upon to stand unsupported temporarily for shoring options such as soldier pile and timber lagging, unless construction dewatering/depressurization measures are undertaken. However, due recognition of groundwater lowering impacts on the existing townhouses, likely supported on shallow footings on the east, close to the subject property limits, needs to be taken into consideration
- Given the weak overburden, for deeper sections of the foundation excavation, perimeter temporary shoring bracing support will be required, e.g., with tiebacks into bedrock for lateral support or for possible raker pile support
- If the overburden groundwater regime is hydraulically connected with the fractured bedrock (this needs to be established within a sealed bedrock zone), the resulting significant upward hydrostatic pressures on slab-on-grade, either need to be resisted, e.g., using rock anchored structural slab or relieved via explicitly engineered moisture barriers (this latter approach can have significant risk of becoming dysfunctional in the long-term)
- Since the rock is fractured, if the excavated walls within the bedrock is under significant groundwater pressure, then jointed bedrock vertical cut faces will require shotcreting with mesh and may even require rock dowelling for stability.

From the foregoing, it is clear, that the method of groundwater control and the nature of the hydrogeological regime, i.e., if the fractured bedrock is part of the groundwater table or not, will dictate the requirements for excavation support, foundation support for the structural loads and slabs-on-grade.

6.3 Excavation Support and Groundwater Control Considerations

6.3.1 Overview

Excavation support issues will critically depend on the required depth of excavations and the nature of subsurface conditions down to and below the required depths. According to the most recent information, the ground level is at El. 102.0 m, P1 at El. 99.0 m and P2 at El. 96.0 m. These are interpreted as finished levels.

Dwg A2.04 shows the plan area of the Mid-Rise tower block, towards the southwest quarter of the property. This plan area will be within the P1 level only area of this building complex ("the one-level basement area"). Hence, the foundation loads will be transmitted below the P1 level slab-on grade. Comparing Fig. 2b with Dwg A2.04 and matching scales approximately, it is clear, that locations of BH21-01 and BH21-02 will be within the mid-rise tower plan area. it is very likely that BH21-03 will also be within the subject plan area. Based on Fig. 2b, there is an approximately 7 m level difference of the top of bedrock levels between BH21-03 and BH21-01.

For purposes of preliminary discussion, based on the finished P1 elevation at El. 99.0 m, and allowing another 1.5 m for slab-on-grade, moisture barrier and the footing thickness, the underside of the footing elevation is estimated at El. 97.5 m.

Similarly, the underside of the P2 footing elevation is estimated at El. 94.5 m. Hence, the P2 area will be able to exploit, in general, competent foundation support from the already excavated or the underlying, not too deep, bedrock. While it may turn out to be beneficial for the foundations to be founded on excavated bedrock, given the jointed nature of the bedrock, basement excavations with vertical faces into jointed bedrock will pose additional lateral stability issues. These issues are discussed in Section 6.3.2.

At the design stage, in the absence of any reasonable knowledge of the extent of the variable bedrock topography over the site (which is only known from the limited bedrock drilling findings), it will be necessarily prudent to adopt a cautionary approach towards excavation support and foundation design for the building complex. Such an approach will significantly minimize contractually expensive changes to designs during construction.

Detailed design considerations for excavation support are detailed in Section 6.5 and Considerations for Control of Groundwater are detailed in Sections 6.3.2 to 6.3.4.

6.3.2 Excavation Support Considerations for the two-level basements

The nature of temporary excavation support, e.g., contiguous caissons, which are pre-installed prior to excavation, except for bracing support installed with progress of excavation or as with soldier piles, which are pre-installed, and timber lagging and bracing support, progressively made with the excavation, will primarily depend on:

- Whether the overburden can stand unsupported for short periods; the cohesionless deposit was found in fourteen (14) out of the sixteen (16) boreholes, in general, to be of very loose to loose compactness.

The underlying silty clay, recorded 0 or WHO (weight-of-hammer), SPT "N" blow counts, in one or more SPT determinations, in seven (7) out of sixteen (16) boreholes

- Ability to secure permission from adjoining property owners for installation of temporary tiebacks; ensuring tieback trajectories are clear of buried utilities and structures; ability to avoid adverse impacts due to tieback installations on buried utilities and structures, even if tieback trajectories are clear of direct impact on buried elements
- the need to control lateral and vertical ground movements outside the excavation to safeguard adjoining buildings and utilities.

While temporary groundwater lowering during construction can stiffen the weak deposits and enhance their stand-up times, however, this will also have the adverse effect of causing settlements on adjacent buildings and utilities, especially given the townhouses, in close proximity, outside the eastern limits of the subject property.

It is estimated that the farther length side of the townhouses is within about 20 m from the eastern boundary of the subject property. Based on the grain size distribution results, for the predominantly silty sand deposit, i.e., the cohesionless deposit, the hydraulic conductivity is estimated as 2.65e-7 m/s (based on BH21-11 SS3 and SS4). The required plan dimensions for the two-level basement excavation are estimated as a rectangle of 40 m by 55 m. Based on the estimated parameters, and assuming an average drawdown of 6.5 m, the estimated zone of influence exceeds 40 m. Hence, the existing townhouses will be well within the zone of influence of temporary construction dewatering. These are preliminary estimates only and need to be confirmed based on a carefully planned hydrogeological investigation.

As discussed in Section 6.3.1, with the estimated base of footings for the two-level basement at El. 94.5 m, excavations will progress into bedrock. Out of the 16 boreholes (from both investigations), only five boreholes were advanced below El. 93.0 m. Out of the five boreholes, only four boreholes have intercepted proven bedrock. As mentioned in Section 5.1.8, elevation of top of the cored bedrock ranged between El. 99.4 m to El. 92.7 m, almost a 7.0 m variation, in eight cored boreholes between the two series of investigations. It is estimated that vertical bedrock excavations up to 4 m will be required for the two-level basement area.

Given the above challenges of dewatering, from a geotechnical point of view, the adoption of contiguous caissons, is recommended for the two-level basement shoring. Within the two-level excavation, with significant overburden thickness to be supported, i.e., in excess about 8.0 m, even contiguous caisson walls are likely to require lateral bracing in the form of tiebacks etc. With this approach for the two-level basement shoring support, i.e., use of contiguous caissons with tieback support, negative impacts of dewatering, and issues of limited stand-up time caused by fast ravelling and running conditions within the saturated overburden can be largely mitigated under good workmanship.

6.3.3 Excavation Support Considerations for the One-Level Basement Area (P1 only footprint)

The P1 basement footprint will require shallower excavations in general. Even within the P1 only footprint, the top of bedrock variations will also impact on the nature of temporary shoring support. For example, excavations on the north can mobilize passive resistance from bedrock within the excavations. However, excavations towards the south will require deeper shoring support to reach bedrock, having to advance through weak sediments, well below PI finish levels, along the construction boundary (with top of bedrock elevation at 92.7 m in BH 21-01).

Given the shallower excavations required for the one-level basement footprint, in general, the depth of dewatering drawdown will be lower. Further, this construction perimeter being further from the most susceptible townhouses compared to the two-level construction perimeter, lateral excavation support in terms of conventional soldier piles and timber lagging would be feasible for this footprint. However, bedrock conditions at the southern boundary, e.g., BH21-01, will need to be established with further clarity to finalize the excavation support and groundwater control requirements.

6.3.4 Implications of Jointed bedrock in Hydraulic Contact with the Overburden Groundwater Table

This situation is likely given the fractured nature of bedrock with elevation (See Fig. 6.1). This groundwater vulnerability has emerged out of the findings of the current investigation. This needs to be explicitly addressed before the foundation designs are finalized. While this will not have a direct impact on lateral support issues, all structural foundations and slabs-on-grade will be subject to uplift pressures with the impact becoming more significant at the two-level basement grade, since more fracturing is evident, i.e., in terms of lower RQD values, below El. 96.0 m (See Fig. 6.1).

From the visual appearance of rock cores, it does not appear that the rock matrix is porous and water flows are hence expected to convey via the fractures. It is important, to confirm if the expected flows can be pumped from within the excavation at the P2 level from filtered and strategically placed sumps excavated into bedrock.

Only general comments will be made in Section 6.4.7 on mitigation of uplift effects, particularly pertaining to Sections 6.4 on Foundation Options, Floor Slab and Underdrainage and Elevator and Sump Pits, due to suspected artesian groundwater pressures within the bedrock mass, until a better understanding of the bedrock mass hydraulic conductivity regime is established.

6.4 Foundation Options

6.4.1 Overview

Based on the discussions on subsurface conditions in Section 6.2, the saturated weak overburden deposits should be avoided as potential foundation support for the proposed building complex. All structural footings should be supported either by spread/strip footings or caissons, founded on or into bedrock. This site as discussed has significant variation between the top of bedrock levels and based on the ground information at hand could be 10 m plus. The topography of the bedrock surface is unclear, and this will require a cautionary approach in addressing foundation interactions. This will have the greatest impact over the tower footprint, i.e., the most heavily loaded area. (see Edgewater Basement Plan dated October 4, 2021, Drawing A.100 Rev1 (Dwg A100)).

Based on our understanding of the likely construction sequence, temporary shoring will be installed first. This shoring will require excavations to be carried down to the respective P1 and P2 levels in the relevant areas. This will also include excavation of bedrock in some areas. These excavations will require groundwater control and ongoing groundwater management during construction to maintain relatively dry working conditions and subgrades.

It is clear, that at the northern portion of the underside of the P1 footings will be into bedrock and at the southern end will be at least a few metres above the top of bedrock. To economize on foundation construction costs, if the mid-rise tower can be re-located to the north-west quadrant of the construction perimeter, subject to further confirmation of bedrock topography, potential for considerable cost savings is feasible. This will enable the exploitation of shallower bedrock levels, thus making it possible, the use of spread/strip footings founded into bedrock.

As per the discussion in Section 6.3.1, the two-level basement excavations are likely to go down to El. 94.5 m. Hence, the base of the P2 area will be largely within excavated bedrock. Hence, the P2 area will be able to exploit, in general, competent foundation support from the already excavated or the underlying, not too deep, bedrock.

Permanent basement wall foundation and lateral support issues, where the major loading is lateral, are discussed in Section 6.5.3.

6.4.2 Mid-Rise Tower

6.4.2.1 Overview

At this preliminary stage of design, foundation support for the high-rise structure can be provided by two options as follows:

(a) Use of rock-socketed caissons over the entire mid-rise footprint

(b) By a combination of spread/strip footings on/into bedrock and where the bedrock is sloping, use to be made of rock-socketed caissons into bedrock.

Both approaches are somewhat open-ended, more so for the latter approach, in so far as the uncertainty about the bedrock topography within the proposed mid-rise footprint, given the level of foundation information at hand. Undertaking of a geophysical seismic refraction survey has the potential to narrow down the uncertainty of the depths to top of bedrock variability. This is strongly recommended.

6.4.2.2 Rock-Socketed Cast-in-Place Concrete Caissons

The mid-rise building and the shear walls can be supported on rock-socketed cast-in-place concrete caissons. Due to the fractured nature of the bedrock, to achieve dry rock-socket conditions can be extremely challenging.

Due to the saturated weak overburden, some of the caissons would be installed from the P1 founding level through the weak overburden where the bedrock topography is lower within the tower footprint. The use of permanent, oversize liners will be required to advance the caisson with minimal loss of ground since the overburden materials will not stand unsupported. The permanent liners must be sealed into bedrock. Additionally, it would require careful construction control to ensure the cleanliness of the socket bases, even if the casings are sealed into the bedrock (See Fig. 6.1: RQD with Rock Core Elevation).

6.4.2.2.1 Axial Resistance

In general, shaft stiffness is known to be significantly higher than base stiffness in a rock-socketed caisson in a rock mass having the same intact rock strengths and discontinuity spacing (fractures). We have taken note of the perceived difficulties in maintaining dry socket conditions and attendant difficulties in ensuring socket base cleanliness. Also, it is to be noted that the concrete compressive strength (f_c') is less than uniaxial compressive strength of intact rock. Based on the above observations, and cognizance of the fact that the bedrock is fractured, i.e., the compressive strength of the rock mass is considerably lower than that of the intact rock, the estimated axial compressive loads for 0.9 m diameter, 1.5 m long rock sockets, are as follows:

Factored Ultimate Design Load in axial compression per rock socket (0.9 m diameter, 1.5 m long) = 3000 kN

The total settlement under the above load is estimated between 5 mm and 10 mm.

A minimum socket diameter of 0.9 m or greater is recommended to facilitate socket base inspection.

Factored Ultimate Design Load in axial uplift per rock socket (0.9 m diameter, 1.5 m long) = 100 kN (Limited by for cone pull-out; FoS = 3); This cone pull-out estimate assumes that the groundwater table is kept below the top of the lowest rock socket until the building dead loads from the structure can suppress any mobilized uplift load. If the mid-rise tower foundation design is dominated by uplift loads (unlikely for the proposed height of tower), then longer rock socket lengths would be required or alternatively, uplift loads should be resisted by vertically anchored caissons. The sides of the rock sockets should be grooved/roughened with a grooving tool to ensure cleanliness of the socket shafts. The skin friction from the overburden must be ignored.

The following constants of vertical subgrade reaction can be used for the design of the rock sockets for vertical loading for single sockets:

K_v_shaft = 960 MN/m^3

K_v_base = 1500 MN/m^3

Downdrag forces are negligible or not applicable for this project.

6.4.2.2.2 Resistance to Lateral Loads

Design for lateral loading should only rely on resistance from the bedrock. Reliance on lateral resistance from the weak overburden should be ignored.

The following design parameters can be used for structural lateral loading design for a 900 mm diameter rock-socketed shaft (single shaft):

Factored ultimate lateral resistance: 1.8 MN/m of socket shaft

Factored serviceability lateral resistance: 1.5 MN/m of socket shaft

Constant of horizontal subgrade reaction, k_h: 1000 MN/m^3 for bedrock within the rock socket shaft

6.4.2.2.3 Pile Group behaviour

Axial Loading:

Providing a centre-to-centre spacing of caissons of at least three (3) diameters is adopted, group effects due to axial loading in bedrock is not considered to be significant.

Lateral Loading:

The coefficient of horizontal subgrade reaction should be reduced, based on the spacing of caissons, and the location of a shaft within the group, to account for pile group effects in lateral loading. The caissons in the lead row have only a minor reduction. The rows that trail any row in front, since they are affected by the voids left behind by the forward movement of the caissons in front of them, suffer more reduction. The fact that in rock sockets, lateral displacements are less than in soil (due to their higher stiffness) has also been factored into the reductions. The reduction factors to be used for a pile group-oriented in the direction of loading are provided in Table 6-3. Intermediate values may be obtained by linear interpolation.

Table 6-3: Coefficient of Horizontal Subgrade Reaction Reduction Factors for Pile Spacing (GEC10 Drilled Shafts Manual)

Pile Spacing Centre-to-Centre	Horizontal Subgrade Reaction Reduction		
	Lead Row	2nd Row	3rd and Higher Rows
3D	0.7	0.5	0.35
4D	0.85	0.65	0.5
5D	1.0	0.85	0.7
Greater or equal to 6D	1.0	1.0	1.0

6.4.2.3 Shallow Foundations

6.4.2.3.1 Overview

For shallow spread footings, the overburden soil below the structural columns, needs to be excavated to the level of founding, into competent bedrock. No part of a spread/strip footing should be founded on the native overburden. It is therefore recommended that all foundations be supported on the underlying bedrock using:

- Spread footings constructed on or into the bedrock surface; or,
- Where the bedrock surface will be deeper, e.g., likely towards the south or the west boundaries of the tower footprint, based on the known bedrock drilling findings, use of rock-socketed caissons should be undertaken.
- If rock socketed caissons are undertaken, then where there is void space between the sloping bedrock and the line of caissons, any overburden present, should be sub-excavated to the bedrock level and backfilled with mass concrete. This mass concrete should be raised to the required level to provide the supporting subgrade for the moisture barrier, underside of the P1 slab-on-grade.

Spread footing bearing issues are further discussed in Section 6.4.2.3.2

6.4.2.3.2 Foundation Design Parameters for Spread Footings

Provided there are no continuous clay-filled seams or mud seams present at shallow depth in the bedrock below the founding level, footings can be supported on a levelled bedrock surface.

The following design recommendations can be used for design of spread/strip footings:

(a) Factored Ultimate Geotechnical Resistance (FULS) factored bearing resistance of 2000 kPa

(b) Factored Serviceability Geotechnical Resistance (FSLS) of 1500 kPa for a total settlement of less than 10 mm.

(c) A coefficient of vertical subgrade reaction of 300 MPa/m

(d) Unfactored Frictional Coefficient of Sliding Resistance = 0.6

Load eccentricity effects should be addressed by the structural designer. Shallow footings must not be smaller than 0.6 m in their smaller dimension. Shallow footings should keep a minimum buffer of 3 m from any bedrock slope crest, i.e., top of bedrock slope. Any load carrying requirements within the buffer zone (3 m) and beyond the crest, if fall within the tower footprint, should be addressed with the use of rock socketed caissons.

If due to varying bedrock topography, should footings at different levels be required, then, in view of the highly jointed nature of the rock mass, structural footings should be placed at the same level by building up structural grade concrete pedestals (with concrete of UCS 30 MPa or more, and any required reinforcements to be detailed by the structural designer) to compensate level differences. To enhance sliding resistance, structural footings should be dowelled into the concrete bases, where such bases are used.

The rock bearing surface should be inspected by qualified geotechnical personnel to confirm that the founding surface has been acceptably cleaned of soil and debris, and any extremely weathered surficial bedrock removed.

6.4.3 Town House Buildings

6.4.3.1.1 Town Houses with Foundation Support at the P1 Level (Units A to H)

Spread footings are feasible. Based on Figs.2 a & 2b, two possible founding scenarios can be identified:

(1) The underside of the footing is at or below the bedrock level: A factored ULS of 250 kPa can be used and settlements are not an issue; an unfactored coefficient of sliding resistance of 0.6 can be used between concrete and clean bedrock and 0.55 between concrete and structural fill.

(2) The underside of the footing is likely marginally above the bedrock level. The structural footing should be built-up form the bedrock level upwards and founding parameters given in Section 6.4.3.1.1 (1) can be adopted.

6.4.3.1.2 Town Houses with Foundation Support likely at the P2 Level (Units I to W)

Based on Fig.2, three possible founding scenarios could be possible:

The first two scenarios would be the same as in Section 6.4.3.1.1 and the founding recommendations given in that section should still be applicable.

The third scenario is likely to occur in the north-east corner of the site, i.e., in the locality of BHs 21-11 and 18-03 where these boreholes were terminated by advancing the DCPT probe below the explored depths of the cited two boreholes and DCPT refusal was met at elevations 91.7 m and 89.8 m, respectively, i.e., no confirmed bedrock. These conditions are very likely to prevail in a localized area. Column loads founded on caissons are recommended to address this scenario, since caissons are required for perimeter shoring in the two-level basement area which includes this localized area (thus avoiding additional mobilization costs for different deep foundation options, such as micro-piles). An option, such as to locally sub-excavate the soft sediments and backfill with mass concrete to the underside of the spread footings, should be avoided in this localized depression, since such measures will result in deeper retainment heights for shoring support to be considered with attendant significant increase in temporary shoring costs. This sub-excavation approach should be particularly avoided as this locality is vulnerable to induce increased lateral movements that would undermine the existing townhouses on to the east. The use of caissons as an underpinning support for the spread footings will also contribute locally to the lateral passive shoring support as well. This will act as an additional restraint in mitigating excavation-induced lateral movements.

For a 600 mm caisson option embedded 300 mm into bedrock, a factored ULS (compression) of 2.7 MPa and a SLS of 2.1 MPa can be used. For a (300 mm dia.) micropile option: foundation loading must by carried by shaft friction within bedrock. A factored ULS compression of 500 kPa can be used for the design of the micropiles. A minimum micropile shaft length of 3.0 m into the bedrock should be used.

6.4.4 Floor Slab and Underdrainage

The floor slabs (one on one-level only basement and the other on the two-level basement of the building complex) can be supported on grade and should be free-floating (not attached to the foundation walls). A moisture barrier consisting of at least 200 mm of 19 mm clear stone should be installed under the floor slab. This moisture barrier has been proven to be effective for conventional floor surfaces and it is assumed that special floor coverings will not be required for parking basements. Any void between the underside of the moisture barrier and top of P1 level bedrock can be backfilled with OPSS 1010 Granular B Type II or better, compacted to 100 percent of the Standard Proctor Maximum Dry Density (SPMDD).

To prevent cross migration of fine particles between subgrade and the clear stone, the clear stone should be fully wrapped with a Class II nonwoven geotextile.

Similar considerations apply for the floor slab at the P2 level.

It is recommended that subgrade preparation and compaction should be carried out under the supervision of a geotechnical representative.

6.4.5 Elevator Pits and Sump Pits

It is anticipated that elevator pits and sump pits will be founded into the bedrock. These pits are generally designed as water-tight structures. The water pressure on the pit walls and the slabs should be designed assuming the water table to be about 0.3 m below the adjacent basement floor, should the basement floor drainage be provided with moisture barriers.

6.4.6 Frost Susceptibility - Foundations

All foundation elements will be founded either on bedrock or on mass concrete, which are both relatively frost resistant. With the provision of moisture barriers, if seepage can be managed, conformance with frost depths should not be applicable in such situations.

6.4.7 Mitigation against Bedrock Induced Hydrostatic Upthrust

If the predicted upward groundwater flows can be managed/drained via moisture barriers or should require more positive restraint, e.g., the use of rock dowels for uplift resistance, will need to await the outcome of hydrogeological input to assess fractured rock hydraulic conductivity. However, in Sections 6.4.7.1 and 6.4.7.2, indicative comments are made about uplift resistance potential within the bedrock mass.

6.4.7.1 Spread/Strip Footings and Caissons Carrying Column Loads

Passive, fully grouted, continuously threaded bar uplift anchors can be used to provide restraint against uplift, if required. Typically, a 25 mm diameter, 3 m long anchor, installed in a 60 mm diameter drill hole into the limestone bedrock, should have a working capacity of about 325 kN (FoS= 3.0), for spread/strip footings.

For rock sockets, if additional uplift resistance is required, for 3.0 m long anchors installed at the bottom of rock sockets, an uplift resistance of 650 kN per anchor (due to the larger uplift cone of rock) can be assumed. These uplift resistances assume that the groundwater table is not above the rock head at any subject uplift anchor. In providing the subject uplift resisting forces, it must be ensured that the working stress (50% of ultimate tensile stress) of passive bars, are not exceeded.

6.4.7.2 Slabs-on-Grade/ Elevator Pits and Sump Pits

In the hydrostatic upthrust scenario, conventional unreinforced slab-on-grade will not be applicable. Structural slabs should be used with required anchorage provided (see Section 6.4.7.1). The coefficient of vertical subgrade reaction between the floor slabs and the underlying compacted granular subgrade can be taken as 50 MN/m^3, if required for structural design purposes.

Once the bedrock hydrogeology regime is established, then the required uplift anchorage for slabs-on-grade, elevator and sump pits needs to be re-visited.

6.5 Geotechnical Considerations for Lateral Support of Excavations

6.5.1 Temporary Shoring

6.5.1.1 Contiguous Caissons

A contiguous caisson wall is composed of a series of vertically drilled holes which are interlocked. A steel beam or soldier pile is typically placed in every third or fourth hole. The intermediate holes are called “filler piles”. Therefore, this Hard/Soft secant pile approach will consist of “soldier piles” and “filler piles”.

Based on the engineering considerations advanced in Section 6.3.2, it was recommended that the lateral excavation support for the P2 basement be provided with contiguous caissons, i.e. secant piles. This method of shoring support provides a much stiffer excavation support than a soldier pile and a lagging wall. Contiguous caissons are also specified in areas with poor/wet ground conditions and where deep dewatering needs to be excluded to avoid dewatering induced settlements. All the above considerations are relevant to the P2 basement area.

The excavation depths to bedrock are expected to exceed 8.0 m and P2 basement shoring will likely require post-tensioned and preloaded tieback support installed into bedrock. The designed interlock between the proposed filler piles and the soldier piles, i.e., the overlap, of adjacent secant piles, should consider the intercepted weak ground conditions, and the requirement for water tightness.

Design parameters for contiguous caisson shoring support and additional construction considerations are discussed in Section 6.5.6.

6.5.1.2 Soldier Piles and Timber Lagging

Soldier piles and timber lagging may be considered as shoring support for the one-level (P1) basement area. Based on the bedrock depths shown in Fig. 2(b), the shoring depths will have a significant variation, from as shallow as 2.7 m or less to as deep as 9 m (BH21-1). At deeper depths, tie backs into bedrock will be required as additional shoring support. Given the silt rich, saturated overburden, stand-up times for timber lagging will be limited. Also, the nature of subsoils has a strong potential for soil erosion between the lagging boards. Hence, dewatering/depressurization of the overburden to the top of bedrock will be required for this method of shoring to be viable.

Design parameters for shoring and additional construction considerations are discussed in Section 6.5.6.

6.5.2 Design Considerations for Permanent Basement Walls

Consideration should also be given to the use of caisson walls for the two-level basement shoring support to act as permanent load-bearing walls. If this is adopted, the tiebacks and contiguous caissons must be designed, in addition to the requirements for temporary shoring, to carry the loads of the permanent structure for the intended design life, whichever scenario governs the critical conditions. Hence, considerations must also include addressing the required strength of concrete for caissons and double-corrosion protection for the tie-back free lengths.

Design parameters for permanent basement walls and additional construction considerations are discussed in Section 6.5.6.

The perimeter drainage requirements for basement walls and underfloor drainage schematics with different perimeter drainage details for shoring systems are shown on Drawings in Appendix G.

6.5.3 Excavation Support and Associated Ground Movements

Even with a supported excavation, some inward lateral movement of the excavated supports is inevitable. In general, the maximum lateral and vertical movements will be of the same order of magnitude. It is generally, the understanding, that the factor of safety against basal heave has a significant effect on lateral wall movements. At the subject site, within the two-level basement area, the perimeter secant caisson shoring will be embedded into bedrock, hence the factor of safety against basal heave is expected to be very high. This should cause a reduction in potential lateral movements (CFEM, Fig. 26.24) and would contribute to mitigating adverse settlement issues on the adjoining townhouse blocks to the east. Well designed, carefully constructed and braced contiguous caisson walls embedded into bedrock at the subject site can be typically expected to have maximum lateral movements less than about 0.2% of the excavation height and the maximum vertical settlement not exceeding the lateral movement. The excavation induced zone of influence behind shoring support (into the backfill), is expected laterally to be not farther than two excavation depths.

Details of a geotechnical related construction monitoring program including monitoring of noise and ground vibrations are described in Section 7.

6.5.4 Bracing Support for Shoring Systems

6.5.4.1 Bottom-up Construction Sequence

In this construction sequence, the permanent works are constructed form the lowest level upwards, beginning with the casting of the lowest basement level foundation slab. Therefore, the bracing for shoring should be provided by temporary tiebacks, rakers and props. The width of the building block is considered too large for use of propping.

6.5.4.1.1 Tiebacks

It is expected that tiebacks will be required for the deeper sections of shoring support, e.g., at the east boundary of the excavation. Given the weak, soil overburden and bedrock is relatively shallow, tieback support via rock anchorage can be considered attractive for bracing. Typically, rock anchors are drilled at 45 degrees. Anchor lengths within the soft alluvium should be permanently cased to avoid hole collapse. These casings must be extended at least 1.5 m below the top of bedrock. Care must be taken during grouting to ensure that the grout is injected from the bottom of the anchor hole to bond the entire length of the grout area. It is also suggested that the anchor holes be thoroughly flushed with water prior to grouting. All rock anchors should be post-tensioned and this will subject the upper zone of the fractured bedrock into a zone of compression (suppressing pull-out) resulting in a stiffer anchor response.

A factored ULS bond strength at the rock- grout interface of 1.25 MPa and a SLS bond strength of 1 MPa can be used for the design rock anchors. Design of rock anchors shall conform to Ch 26 of CFEM (4th ed). It is important to note that the minimum anchor length should lie beyond the $45 - \phi/2 + .15H$ line drawn from the base of the soldier pile. The minimum fixed anchor length (i.e., the length along which the rock to grout bond is developed) should be greater than 3 m.

Bond resistance critically depends sound installation methods and grouting procedures. Hence the contractor must decide on a capacity and confirm its availability by field testing. All anchors must be tested as indicated in the Foundation Engineering Manual, 4th edition. The installation and testing of the anchors must be observed by a geotechnical engineer.

6.5.4.1.2 Rakers

Rakers can be used to render lateral support to a shoring system. A raker is a steel member generally installed at 45 degrees to the shoring. The base of the raker is seated in a concrete footing and the top is welded to a soldier pile. Rakers can interfere with construction, require earth berms to temporarily support the shoring while the rakers are installed and generally do not provide stiffness as much as tiebacks.

They are not prudent in locations where good control of lateral soil movements are required. However, they can take advantage of shallow bedrock conditions. For these reasons, the use of rakers should be confined, if at all, to the one-level (P1) basement area only.

6.5.4.2 Top-down Construction Sequence

In this construction sequence, the use of the permanent structure provides the temporary bracing to the retaining walls. Therefore, the higher-level slabs are cast before the lower-level slabs to act as horizontal frames for wall support as the excavation progresses. The constrictions on the size of plant and limited access, further constrained by the different levels of basements and rock excavation issues associated with the subject site would make this construction approach not favourable for the site conditions.

6.5.5 Design Parameters and Additional Construction Considerations for Temporary Shoring and Permanent Basement Walls

6.5.5.1 Static Lateral Earth Pressure Parameters for Temporary Shoring

- For overburden: ground surface is assumed as El. 102.0 m
- where movement is minimal, the earth pressure coefficient, K = 0.45

  and for shoring on the east boundary, the earth pressure coefficient, K = 0.5

  where minor movement (0.002H) can be tolerated, K = 0.25
- For bedrock: K = 0.2
- Groundwater level: top of existing ground surface (El. 102.0 m) unless dewatering/depressurization is undertaken; top of ground surface for permanent basement walls
- Surcharge: to be determined by the shoring contractor
- The subject excavations will go into bedrock and in most cases, the soldier piles will not extend below the base of the rock excavation due to the associated high cost of rock excavation. So passive earth pressure from bedrock will not be relevant for soldier pile lengths that extend below the top of bedrock but stop short of the base of the excavation as the face of the soldier piles is normally set along the line of the proposed foundation wall. However, all soldier piles extending below the top of bedrock should

be installed to a minimum of 3 m into the bedrock or to the excavation bottom if the rock excavation is less than 3 m deep.

6.5.5.2 Design Parameters for Vertical Loading of Soldier Piles

If in the event, soldier piles are taken down to the level of the excavation base in bedrock, then the safe bearing pressure for soldier pile caissons can be taken as q = 1200 kPa, provided the caisson holes are clean prior to pouring concrete. Assuming a slurry procedure and tremie concrete, then q = 600 kPa.

6.5.5.3 Global Stability of Temporary Shoring

- For global stability check: Soil - ϕ' = 28 deg; c’ = 0; Saturated Unit Weight = 18 kN/m^3
- Bedrock - ϕ' = 35 deg; c’ = 25 kPa; Saturated Unit Weight = 26 kN/m^3
- Surcharge to be determined by the shoring contractor
- Groundwater level: top of existing ground surface (El. 102.0 m) unless dewatering/depressurization is undertaken.

6.5.5.4 Lateral Earth Pressure Parameters for Permanent Basement Walls

Static Earth Pressure Parameters:

- For overburden: ground surface elevation is assumed as El. 102.0 m
  The earth pressure coefficient, K = 0.40
- For bedrock: K = 0.2
- The basement walls should be designed for full hydrostatic pressure (with groundwater table at El. 102.0 m, irrespective of the fact that a drainage board is provided between the basement and the shoring; the long-term effectiveness of geosynthetic drainage for the full design life of the building cannot be relied upon)

Static Earth Pressure Parameters for OPSS 1010 Granular Materials are given in Table 6.4.

Table 6-4: Static Lateral Earth Pressure Parameters

Parameter		Granular A	Granular B	Other OPSS1010 ‘Granular’
Unit Weight (γ) kN/m^3	Above groundwater	22.5	21.7	21.7
	Below groundwater	12.7	11.9	11.9
Angle of Internal Friction (ϕ')		35°	32°	31°
Coefficient of Active Earth Pressure (k_a)		0.27	0.31	0.32
Coefficient of Passive Earth Pressure (k_p)		3.69	3.23	3.12
Coefficient of Earth Pressure at Rest (k_o)		0.43	0.47	0.48

Seismic Earth Pressure Parameters (Fig. 2c shows the vicinity fault lines):

For effective friction angle = 28 deg, Seismic Site Class = D, Site Adjusted PGA = 0.332,

$K_{AE} = 0.478$, $K_{PE} = 2.476$: Seismic Horizontal Acceleration Coefficient, $k_h = 0.5*$ Site Adjusted PGA

For Laterally Restrained: K = 0.651 : Seismic Horizontal Acceleration Coefficient, k_h = Site Adjusted PGA

6.5.5.5 Excavated Bedrock Face Stability of Basements

Vertical excavation faces into bedrock, in view of the fractured nature of bedrock and ground water, may require shotcreting with pinned steel mesh. Based on the visual assessment of the exposed rock face during excavation, the stability of the rock face must be inspected by a geotechnical engineer to assess if the use of short rock dowels is necessary for excavated rock face stabilization.

6.5.5.6 Additional Construction Considerations

The design of the shoring system is the responsibility of the contractor. The contractor shall hire a professionally qualified and experienced shoring designer to provide a detailed design for the shoring system. However, this report has provided some general guidelines on possible concepts for shoring, to be used by the designers for assessing the possible impacts of the shoring design and site works as well as to evaluate the potential for impacts of this shoring on the adjacent properties.

The use of rock anchors would require the permission of the adjacent property owners (including the City) since the anchors would be installed beneath their properties. The feasibility of using tie backs will also depend on the presence of existing utilities or structures which may interfere with the installation of tiebacks. It is important to ensure the need to avoid adverse impacts due to tieback installations on buried utilities and structures, even if tieback trajectories are clear of direct impact on buried elements.

Given the weak saturated overburden, use of sealed permanent casing is strongly recommended for the east perimeter shoring. For the remaining of the basement boundaries, at least, temporary casing should be used. If dry socket bases cannot be established, careful tremie construction should be used for caisson construction.

Potential contractors should seek independent hydrogeological advice to mitigate adverse impacts on adjoining buildings, buried utilities and structures. The groundwater levels across the site are very high and may pose a challenge during excavations. Relatively dry conditions should be maintained within the excavation.

The excavation and the installation of temporary protection systems shall be performed in according with OPSS.MUNI 902, and OPSS.MUNI 539.

All of the backfill should be placed in horizontal lifts of uniform thickness of no more than 300 mm before compaction at appropriate moisture content determined by the Proctor test. The backfill should be compacted to a minimum of 98% Standard Proctor Maximum Dry Density SPMDD.

The shoring method(s) chosen to support the excavation sides must take into account the soil and bedrock stratigraphy, the groundwater conditions, the methods adopted to mange the groundwater and construct the shoring systems, the permissible ground movements associated with the excavation and construction of the shoring system, and their potential impact on adjacent structures and utilities.

The final selection of the type of temporary shoring system and the method of lateral restraint shall be the responsibility of the contractor.

6.5.6 Frost Protection

The minimum frost protection burial depth of water bearing utilities in the Township is typically 2.4 m below the finished grade of the road. The frost penetration depth for the locality is 1.8 m below the ground surface.

The overburden at this site is potentially frost susceptible and should not be used as backfill behind exterior, unheated, or well insulated foundation elements within the depth of potential frost penetration to avoid problems with frost adhesion and heaving.

If shoring is to be carried out over the winter months or if the excavation is to be left open for any period during below zero temperature, shored walls must be protected against frost penetration by means of insulation or heated hoarding.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible materials beyond the wall backfill. To avoid excessive deferential heaving, the backfill adjacent to the wall should be placed to form a frost taper starts from 1.5 m depth below the finished grade to the pavement subgrade level at 3H:1V, or flatter.

All earthworks must be protected from freezing. Backfilling materials should not be placed on frozen ground as per OPSS.MUNI 902.

6.6 Cement Type and Corrosion Potential

Seven soil samples were submitted to Parcel laboratories for testing of chemical properties relevant to exposure of concrete elements to sulphate attacks as well as potential soil corrosivity effects on buried metallic structural elements. Test results are presented in Table 5-7.

The potential for sulphate attack on concrete structures is moderate to low. Therefore, Type GU Portland cement may be adequate to protect buried concrete elements within the subsurface conditions encountered.

Based on electrical resistivity results and chloride content, the corrosion potential for buried steel elements is within the slightly to nonaggressive range.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 General

Additional construction issues that need resolution may be warranted, depending on the contractors' means and methods. Potential contractors should seek independent geotechnical advice for such issues. Potential construction considerations include, but are not necessarily limited to the following:

7.2 Site Preparation

As previously noted, the footprint of the proposed development is underlain by stratified overburden composed of fill, silty clay/clayey silt, silty sand/clayey sand, silty clay and till. The existing overburden contains different portions of sand, gravel, silt and clay with different competence levels throughout the site. The fill and overburden are not suitable to support the structural loads of the proposed development. However, fill can be bulk sampled during construction and tested for gradation and proctor to determine if it can be used or not, for landscaping or general grade raise. Soil disposal issues are addressed in a separate report (please refer to the Environmental Report).

7.3 Excavatability Issues

Excavation of overburden soil can be performed using conventional hydraulic excavating equipment.

Bedrock excavation will be required to accommodate the underground parking levels. The top of bedrock depths varies significantly across the site (see Fig. 2b). Bedrock removal is expected to be achievable through hydraulic excavation equipment and hoe ramming.

Uniaxial compressive strengths ranging from 72 MPa to 197 MPa have been determined on intact limestone bedrock specimens (see Section 5.1.8). Presence of higher strength rock bands cannot be ruled out. The contractor is advised to take these observations, and the reported presence of cobbles and boulders documented in the borehole logs into account, in selecting means and methods for bedrock excavation including for caissons and tiebacks. Such rock excavations could require more aggressive excavation methods, such as by undertaking line drilling, which should be independently assessed by the contractor but blasting should be avoided.

7.4 Open-Cut Excavation Stability

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario and based on engineering experience, the overburden within the limits of the project can be classified as Type 3 Soil, above groundwater, but in view of the fissured nature of the surficial silty clay, should not be sloped steeper than 2H:1V. Soils below groundwater, are classified as OHSA Type 4 Soil. For, OHSA Type 4 Soil, the recommend sloping for the excavation sides are 3H:1V or shallower or be shored. For excavations through multiple soil types or rock, the side slope geometry is governed by the soil with the highest Type number designation. However, during construction, before worker entry, a geotechnical engineer upon inspecting the excavations, may revise these assessments.

If space restrictions exist, excavations deeper than 1.2 m can be supported by temporary support systems in compliance with OPSS.MUNI 539, OHSA, O.Reg. 213/91.

Cobbles or boulders larger than 300 mm in diameter should be removed from the side slopes for worker safety. Please refer to the factual section of this report which includes a description various layers of the overburden.

Large size of rock fill, cobbles, and boulders may be encountered due to rock excavations. The Occupational Health and Safety Act (OHSA) of Ontario indicate that rock could be classified as Type 1 Soil. However, in view of low RQD values recorded (see Fig. 6.1), side slopes in highly fractured bedrock, i.e., with low RQD, are classified as OHSA Type 3 soil and should be sloped no steeper than 1H:1V. For basement side wall excavations which are vertical, recommendations are given in Section 6.5.5.5 for rock face stabilization considerations.

7.5 Required Dewatering Permits

The excavation needs to be kept relatively dry. Due to the monitored high groundwater level, dewatering will be required as per OPSS.MUNI 517.

Based on the O.Reg 63/13 and O.Reg 387/04, if the volume of extracted water expected is greater than 400,000 liters per day, then a Permit to Take Water (PTTW) needs to be obtained from the Ontario Ministry of Environment Conservation and Parks. For expected groundwater extraction between 50,000 and 400,000 liters per day, an Environmental Activity and Sector Registry (EASR) permit is adequate. The volume of pumped water per day is a function of the length of excavated trench and the dewatered zone. Based on the encountered stratigraphy, the amount of groundwater intake, and the size of excavation, an application for a Permit to Take Water (PTTW) will be required.

All backfilling shall comply with the OPSS.MUNI 501 and the City of Ottawa Special Provision General No. D-029 for compaction requirements, unless the design recommendations included in this report exceed provisions of OPSS.MUNI 501 and D-029.

7.6 Construction Monitoring Requirements for Geotechnical Issues

Special attention should be given to excavations in the proximity of the existing utilities. Public and private utility authorities should be notified before the commencement of construction of the proposed development, and all utility clearance documents should be obtained. The contractor shall inform owners of all existing utilities before proceeding with excavation. The utility owners may provide the permissible deformation that a particular utility may tolerate.

A monitoring program should be developed by the shoring designer to monitor the inward movements of the excavation support system to ensure compliance with the design assumptions and avoidance of adverse impacts on nearby structures and buried services. Similar requirements apply for dewatering impacts.

A preconstruction/dilapidation survey is recommended to be carried out on the existing townhouse blocks to the east, utilities, or pavements that may be affected by ground movements. This survey should be carried out prior to the commencement of any dewatering/excavation to establish baseline readings on targets mounted on selected front portions of the existing townhouses and on selected targets for other accessible utilities and structures. At least three sets of baseline readings should be undertaken.

Both vertical and horizontal targets should be installed on the caissons, with a greater number of targes on the east side caissons. Survey monitoring should be undertaken to capture x,y,z of target movements (caissons and townhouses) and at each support level. The monitoring should be continued till the end of below-ground construction or as until determined by the shoring contractor, whichever is later, at least on a weekly basis, including the winter months to detect any damaging movements due to frost. Regular monitoring also provides the means of early detection of potential problems or signs of overloading of the system. Evidence of adverse readings will enable directions to be given to improve shoring support. It is emphasized that shoring monitoring is an integral part of the design philosophy.

Therefore, it is recommended that monitoring points on building foundations in the vicinity of the excavated areas within the larger of the two radii of influence zones, i.e., the excavation induced zone of influence and the dewatering induced zone of influence be installed, to document the monitoring point elevations before and after construction. The reference points on structures should be kept for future reference. This will also help to address third party claims. A monitoring program shall be included in the Quality Control documents submitted by the contractor. The monitoring program where required should conform to industry practices.

If there are any structures that the designers may have concern, McIntosh Perry has the expertise and resources to design and implement case-specific monitoring programs for these structures under a separate scope.

7.7 Ground Vibrations and Noise

Shoring installation operations must ensure that construction-induced vibration levels do not adversely result in soil disturbance and associated unexpected settlements of the nearby roadway, buildings, and utilities. Also, the noise level should be kept at a tolerance level of noise per the City of Ottawa requirements. Applicable non-exceedance noise levels should be adhered to in the operation of pump units and in other construction processes.

Vibration and deformation monitoring will be required throughout the process of shoring installation, excavation, construction, and backfilling.

7.8 Subgrade Preparation and Inspection

The excavated subgrade must be kept dry at all times to minimize disturbance to the subgrade under the influence of dewatering/depressurization requirements as discussed under Section 6.3. The subgrade shall be kept dry at all times, especially before compaction and proof rolling.

Any grade adjustment due to over-excavation shall be Granular A conforming to OPSS.MUNI 1010. Granular A should be placed in loose uniform lifts not thicker than 300 mm and compacted to a minimum of 98% SPMDD.

A geotechnical engineer or technician should attend the site to confirm subgrade excavation, type of fill material and level of compaction.

8.0 PAVEMENT AND HARD SURFACING

8.1 Pavement Structure

It is understood the entire above ground parking area will be located on a reinforced concrete slab. The structural engineer and the architect shall comment on surface covering of their choice. There are two entrances predicted for the site which is expected to be heavily used connecting the road to the structure. The pavement structure shown in the following table shall be used for the road connection to the structure.

Table 8-1: Pavement Structures

Material		Thickness (mm)
Surface	Superpave 12.5 mm, PG 58-34	40
Upper Binder	Superpave 19 mm, PG 58-34	50
Base	OPSS Granular A	150
Sub-base	OPSS Granular B Type II	450

The base and sub-base materials, i.e., Granular A for base and Granular B Type II for sub-base, shall be in accordance with OPSS.MUNI 1010. Both base and sub-base should be compacted to 100% SPMDD. Asphalt layers should be compacted to comply with OPSS 310.

8.2 Sidewalks and Hard Surfacing

The width and extent of the sidewalks will be defined as per the architectural drawings. The designer shall provision adequate slope, based on applicable codes, to provide appropriate runoff discharge. Sidewalks can be categorized under residential/commercial use. The City of Ottawa SC4 'Typical Concrete Sidewalk in Boulevard' standard is recommended for the construction of the concrete sidewalk. Expansion, construction, and dummy joints shall be spaced per the City of Ottawa standards SC5 'Sidewalk Construction Joints' and SC14 'Sidewalk Joints'. A minimum of 150 mm bedding of OPSS Granular 'A' compacted to 100% SPMDD is required for the concrete sidewalk panels.

All proposed new curbs shall be constructed as per applicable standards. It is recommended to follow City of Ottawa detail provided in SC1.4 'Concrete Barrier Curb with Sidewalk'. Curbs should receive a minimum of 150 mm Granular 'A' bedding compacted to 100% SPMDD on a proof rolled subgrade that is free from soft, loose, and organic materials.

Interlocking concrete pavers could be used for the walkways and terraces at this site. The concrete pavers should meet the requirements of ASTM C936. The concrete pavers used for walkways (no vehicle traffic) should be placed on a minimum of 25 mm sand bedding which should meet the gradation requirements for concrete sand as described in CSA A23.2-04, Section 4.2.3.3. Below the sand, a minimum of 300 mm OPSS Granular 'A' compacted to 100% SPMDD should be provided.

9.0 SITE SERVICES

At the subject site, the burial depth of water-bearing utility lines is typically 2.4 m below the ground surface. If this depth is not achievable due to the bedrock level, equivalent thermal insulation should be provided.

The Occupational Health and Safety Act (OHSA) of Ontario indicated that native overburden can be classified as Type 4 soil, and excavation side slopes can be sloped at a minimum of 3H:1V or be shored. If space restrictions are encountered, the excavations can be carried out within closed sheeting, which is fully braced to resist lateral earth pressure.

Utilities should be supported on a minimum of 150 mm bedding of Granular A compacted to a minimum of 98% of SPMDD. Utility cover can be Granular A or Granular B type II compacted to 96% SPMDD. All covers are to be compacted to 100% SPMDD if they are intersecting structural elements. Compaction loads can damage the buried pipes and thus it is recommended to choose utility pipes that can tolerate compaction loads.

A geotechnical engineer or technician should attend the site to confirm the bedrock, type of fill material, and level of compaction. All bearing surfaces should be inspected by experienced geotechnical personnel before pouring the concrete to ensure that strata having adequate bearing capacity have been reached, and the bearing surfaces have been properly prepared.

10.0 CLOSURE

We trust this geotechnical investigation report meets the requirements of your project. The "Limitations of Report" presented in Appendix A are an integral part of this report. Please contact the undersigned should you have any questions or concerns.

McIntosh Perry Consulting Engineers Ltd.

N'eem Tavakkoli, M.Eng., P.Eng.
Senior Geotechnical Engineer

Vasantha Wijeyakulasuriya, M.Eng., P.Eng.
Senior Technical Director, Geotechnical

LICENSED PROFESSIONAL ENGINEER
M. AZIZI TAVAKKOLI
100180911
Oct-25/21
PROVINCE OF ONTARIO

REFERENCES

1) Canadian Geotechnical Society, "Canadian Foundation Engineering Manual", 4th Edition, 2006.

2) Ontario Ministry of Natural Resources (OMNR), Ontario Geological Survey, Special Volume 2, "The Physiography of Southern Ontario", 3rd Edition, 1984.

3) Google Earth, Google, 2015.

4) Government of Canada, National Building Code of Canada (NBCC), "Seismic Hazard Calculation" (online), 2010.

5) Canadian Standards Association (CSA), "Concrete Materials and Methods of Concrete Construction", A23.1, 2009

6) Government of Ontario, "Ontario Building Code (OBC)," (online), 2012.

7) MTO – Pavement Design and Rehabilitation Manual

8) Natural Resources Canada – Seismic Hazard Calculator

APPENDIX A
LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

McIntosh Perry Consulting Engineers Ltd. (McIntosh Perry) carried out the field work and prepared the report. This document is an integral part of the Foundation Investigation and Design report presented.

The conclusions and recommendations provided in this report are based on the information obtained at the borehole locations where the tests were conducted. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations where tests were conducted and conditions may become apparent during construction, which were not detected and could not be anticipated at the time of the site investigation. The benchmark level used and borehole elevations presented in this report are primarily to establish relative differenced in elevations between the borehole locations and should not be used for other purposes such as to establish elevations for grading, depth of excavations or for planning construction.

The recommendations presented in this report for design are applicable only to the intended structure and the project described in the scope of the work, and if constructed in accordance with the details outlined in the report. Unless otherwise noted, the information contained in this report does not reflect on any environmental aspects of either the site or the subsurface conditions.

The comments or recommendation provided in this report on potential construction problems and possible construction methods are intended only to guide the designer. The number of boreholes advanced at this site may not be sufficient or adequate to reveal all the subsurface information or factors that may affect the method and cost of construction. The contractors who are undertaking the construction shall make their own interpretation of the factual data presented in this report and make their conclusions, as to how the subsurface conditions of the site may affect their construction work.

The boundaries between soil strata presented in the report are based on information obtained at the borehole locations. The boundaries of the soil strata between borehole locations are assumed from geological evidences. If differing site conditions are encountered, or if the Client becomes aware of any additional information that differs from or is relevant to the McIntosh Perry findings, the Client agrees to immediately advise McIntosh Perry so that the conclusions presented in this report may be re-evaluated.

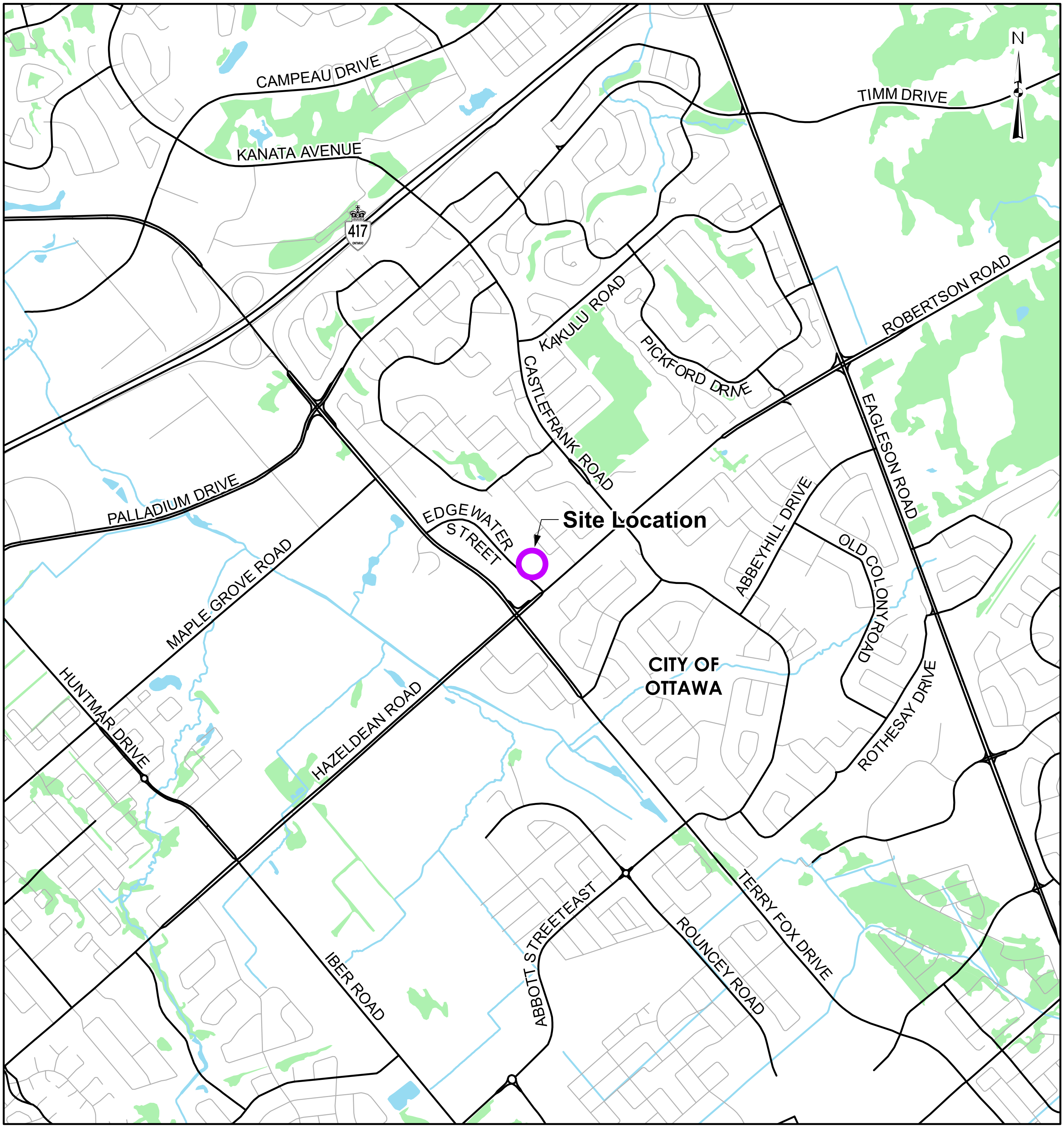
Under no circumstances shall the liability of McIntosh Perry for any claim in contract or in tort, related to the services provided and/or the content and recommendations in this report, exceed the extent that such liability is covered by such professional liability insurance from time to time in effect including the deductible therein, and which is available to indemnify McIntosh Perry. Such errors and omissions policies are available for inspection by the Client at all times upon request, and if the Client desires to obtain further insurance to protect it against any risks beyond the coverage provided by such policies, McIntosh Perry will co-operate with the Client to obtain such insurance.

McIntosh Perry prepared this report for the exclusive use of the Client. Any use which a third party makes of this report, or any reliance on or decision to be made based on it, are the responsibility of such third parties. McIntosh Perry accepts no responsibility and will not be liable for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

16 EDGEWATER STREET KANATA, ON

APPENDIX B
FIGURES

McINTOSH PERRY



600 300 0 600

Scale 1:25,000 Metres

LEGEND

○ Site Location

— Local Road

— Major Road

~ Watercourse

🟦 Waterbody

🟩 Wooded Area

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2021.

CLIENT:

MCCLUSKEY GROUP

PROJECT:

16 EDGEWATER STREET, OTTAWA, ON

TITLE:

SITE LOCATION

MCINTOSH PERRY

115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

PROJECT NO:CCO-22-0244

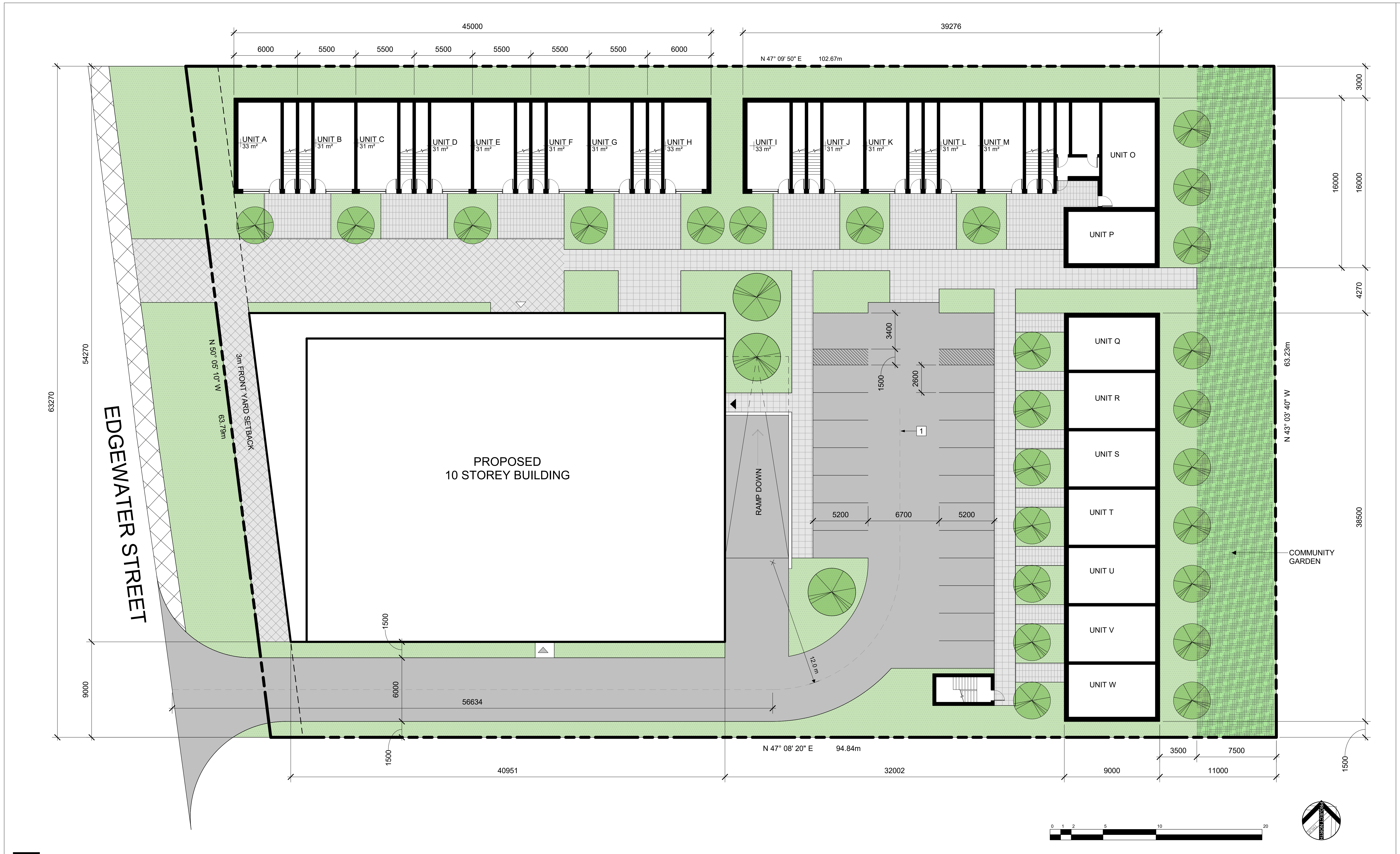
Date	Jul., 22, 2021
GIS	EU
Checked By	MA

FIGURE:

1

a

C:\Users\e.ungun\McIntosh Perry\GIS - Documents\Projects\2022\CCO\CCO-22-0244 Residential Housing - 16 Edgewater Street\aprx\Geotech\CCO-22-0244_Geotech.aprx



1 SPC-SP-OVERALL SITE

A.100 1 : 200

SITE PLAN GENERAL NOTES:

1. [redacted]ORMATION AND CONDITIONS COMPILED FROM EXISTING PLANS AND SURVEYS
2. DO NOT SCALE THIS DRAWING
3. REPORT ANY DISCREPANCIES PRIOR TO COMMENCING WORK. NO RESPONSIBILITY IS BORN BY THE CONSULTANT FOR UNKNOWN SUBSURFACE CONDITIONS
4. CONTRACTOR TO CHECK AND VERIFY ALL DIMENSIONS ON SITE AND REPORT ANY ERRORS AND/OR OMISSIONS TO THE CONSULTANT
5. REINSTATE ALL AREAS AND ITEMS DAMAGED AS A RESULT OF CONSTRUCTION ACTIVITIES TO THE SATISFACTION OF THE CONSULTANT
6. CONTRACTOR TO LAYOUT PLANTING BEDS, PATHWAYS ETC. TO APPROVAL OF CONSULTANT PRIOR TO ANY JOB EXCAVATION
7. THE ACCURACY OF THE POSITION OF UTILITIES IS NOT GUARANTEED - CONTRACTOR TO VERIFY PRIOR TO EXCAVATION
8. INDIVIDUAL UTILITY COMPANY MUST BE CONTACTED FOR CONFIRMATION OF UTILITY EXISTENCE AND LOCATION PRIOR TO DIGGING
9. ALL DISTURBED AREAS TO BE RESTORED TO ORIGINAL CONDITION OR BETTER UNLESS OTHERWISE NOTED

SITE PLAN LEGEND:

■	EXISTING BUILDING
■	ASPHALT PAVING
▨	NEW GRASS
▦	CONCRETE SIDEWALK
▨	CONCRETE PAD
▦	MULCH/PLANTING
▨	GRAVEL/RIVERSTONE/MAINTENANCE STRIP
▨	STONE DUST/SAND
▦	PAVER TYPE 1
▦	PAVER TYPE 2
▦	PAVER TYPE 3
▨	EXISTING MATERIAL 1
▨	EXISTING MATERIAL 2
▨	EXISTING MATERIAL 3
▲	EMERGENCY EXIT
▲	SERVICE DOORS
△	BUILDING MAIN ENTRANCE
—	PROPERTY LINE
—x—x—	FENCE PER LANDSCAPE
—WTR—WTR—	NEW DOMESTIC WATER
—SAN—SAN—	NEW SANITARY
—ST—ST—	NEW STORM
—H—H—	NEW ELECTRICAL SERVICE (BELOW GRADE)
—H—H—	GAS
□ CB	CATCH BASIN
□ EX CB	CATCH BASIN
○ LS	LIGHT STANDARD
○ EX LS	LIGHT STANDARD EXISTING
○ FH	FIRE HYDRANT
○ EX FH	FIRE HYDRANT EXISTING
⊗ MH	MANHOLE
⊗ EX MH	MANHOLE EXISTING
● UP	UTILITY POLE
● EX UP	UTILITY POLE EXISTING
○ XX	CUSTOM SYMBOL
○ EX XX	CUSTOM SYMBOL EXISTING
Y	SIAMESE CONNECTION
—	DROPPED CURB
○	NEW TREE
⊗	NEW SHRUB
○	EXISTING TREE

LEGAL DESCRIPTION:

PART OF LOT 30 CONCESSION 12 GEOGRAPHIC TOWNSHIP OF GOULBOURN

REFERENCE SURVEY:

BASED ON INFORMATION FROM A SURVEY PREPARED BY McINTOSH PERRY SURVEYING INC. DATED MAY 11, 2021.

MUNICIPAL ADDRESS:

16 EDGEWATER STREET

DEVLOPMENT INFORMATION:

SITE AREA	6246.9 m²
BUILDING AREA	m²
GROSS FLOOR AREA	m²
BUILDING HEIGHT	30.0 m 10 STOREYS
ZONE	IG2
SCHEDULE 1:	AREA C SUBURBAN
SCHEDULE 1A:	AREA C
SCHEDULE 2:	DISTANCE EXCEEDS 600 m
UNITS:	UNITS TOTAL

ZONING PROVISION	REQUIRED	PROVIDED
MIN. LOT WIDTH		
MIN. LOT AREA		
MIN. FRONT YARD SETBACK		
MIN. REAR YARD SETBACK		
MIN. INTERIOR YARD SETBACK		
MAX. HEIGHT		
AMENITY AREA		

PARKING QUEING + LOADING	REQUIRED	PROVIDED
RESIDENTIAL SPACES		
VISITOR SPACES		
ACCESSIBLE PARKING		
BICYLCLE PARKING		

SITE PLAN KEYNOTES:

1 FIRE ROUTE

Figure 1b

CSV ARCHITECTS
sustainable design · conception écologique
613.564.8118
www.csv.ca
190 O'Connor Street, Suite 100
Ottawa, Ontario,K2P 2R3

STRUCTURAL ENGINEER
name
address
address
phone
email

MECHANICAL ENGINEER
name
address
address
phone
email

ELECTRICAL ENGINEER
name
address
address
phone
email

CIVIL ENGINEER
name
address
address
phone
email

LANDSCAPE ARCHITECT
name
address
address
phone
email

STAMP

REV	DATE	ISSUE
1	2021-10-04	Issued for Coordination

NOTES

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2. THE DRAWINGS, PRESENTATIONS AND SPECIFICATIONS AS INSTRUMENTS OF SERVICE ARE AND SHALL REMAIN THE PROPERTY OF CSV ARCHITECTS. THEY ARE NOT TO BE USED BY THE CLIENT ON OTHER PROJECTS OR ON EXTENSIONS TO THIS PROJECT WITHOUT THE WRITTEN CONSENT OF CSV ARCHITECTS.
3. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL OTHER PROJECT DRAWINGS AND SPECIFICATIONS.
4. DO NOT SCALE DRAWINGS. CONTRACTOR SHALL BE RESPONSIBLE TO VERIFY DIMENSIONS ON SITE.
5. ALL WORK SHALL BE IN ACCORDANCE WITH THE ONTARIO BUILDING CODE AND ALL SUPPLEMENTS AND APPLICABLE MUNICIPAL REGULATIONS.

CLIENT

PARK RIVER PROPERTIES
OTTAWA
ONTARIO, CANADA

PROJECT

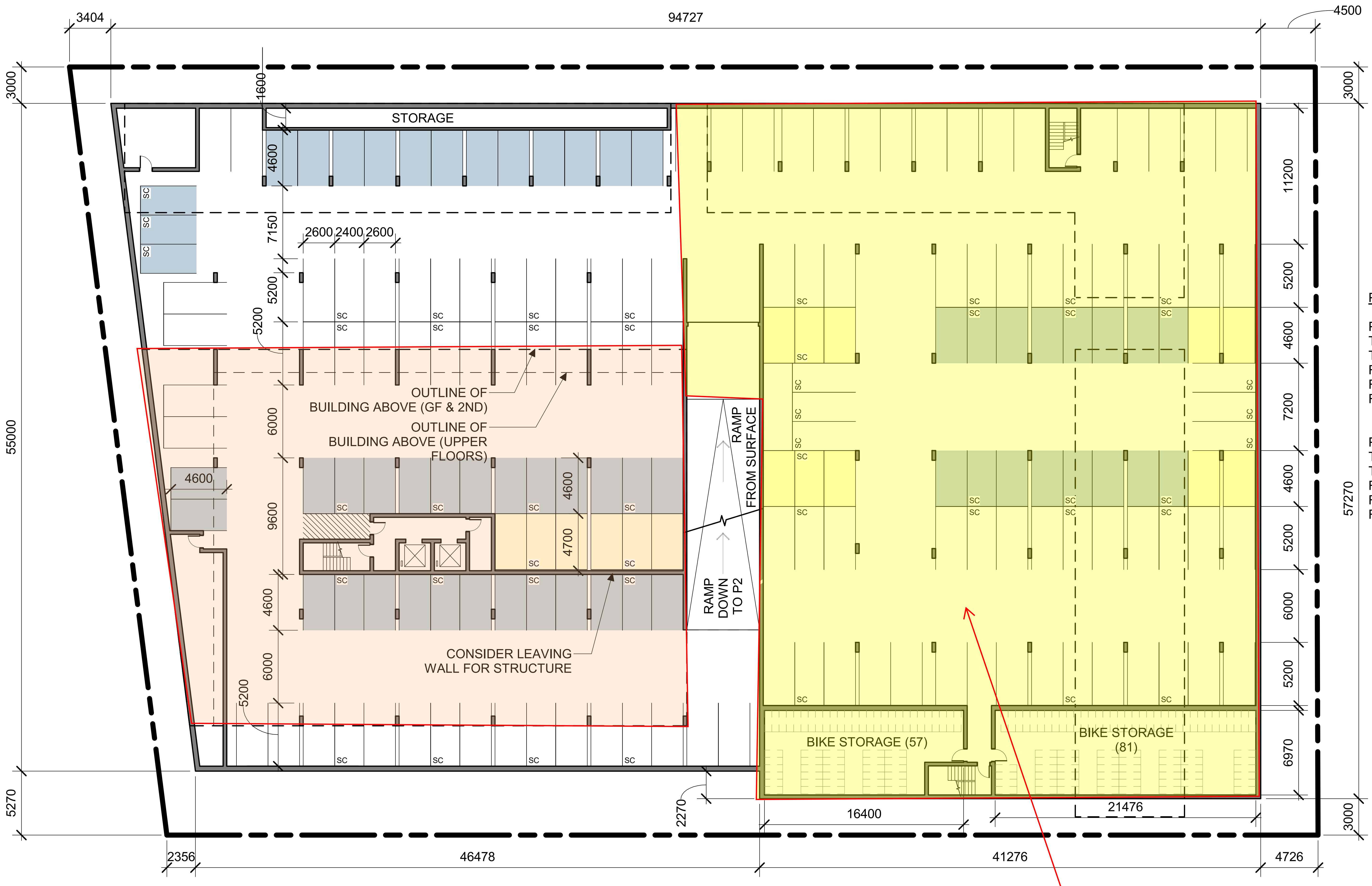
EDGEWATER DEVELOPMENT
16 EDGEWATER
OTTAWA, ONTARIO

TITLE

SITE PLAN

PROJECT NO: PROJECT NUMBER
DRAWN:
APPROVED:
SCALE: 1 : 200
DATE PRINTED: 2021-10-04 4:07:33 PM

REV: 1
DRAWING NO.: A.100



PARKING CALCULATION

P1
TOTAL - 194
TANDUM - 16
REDUCED DEPTH - 52
REDUCED WIDTH - 12
REDUCED WIDTH & DEPTH - 19

P2
TOTAL - 82
TANDUM - 10
REDUCED DEPTH - 18
REDUCED WIDTH - 17
REDUCED WIDTH & DEPTH - 8

A2.04 16 EDGEWATER | BASEMENT PLAN
1 : 300 2021-06-30

Fig. 1c

CSV ARCHITECTS
sustainable design · conception écologique

20 10 0 20

Scale 1:750 Metres

LEGEND

▭ Approximate Site Boundary

⊕ Borehole Location

⊕ Previous Preliminary Geotechnical Investigation in 2018

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2021.

CLIENT:
MCCLUSKEY GROUP

PROJECT:
16 EDGEWATER STREET, OTTAWA, ON

TITLE:
BOREHOLE LOCATIONS

MCINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

PROJECT NO:CCO-22-0244		FIGURE:
Date	Aug., 05, 2021	2a
GIS	EU	
Checked By	MA	

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20 10 0 20

Scale 1:750 Metres

LEGEND

▭ Approximate Site Boundary

⊕ Borehole Location

⊕ Previous Preliminary Geotechnical Investigation in 2018

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2021.

CLIENT:
MCCLUSKEY GROUP

PROJECT:
AFFORDABLE HOUSING
16 EDGEWATER STREET, OTTAWA, ON

TITLE:
BOREHOLE LOCATIONS

MCINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

PROJECT NO:CCO-22-0244		FIGURE:
Date	Aug., 05, 2021	2b
GIS	EU	
Checked By	MA	

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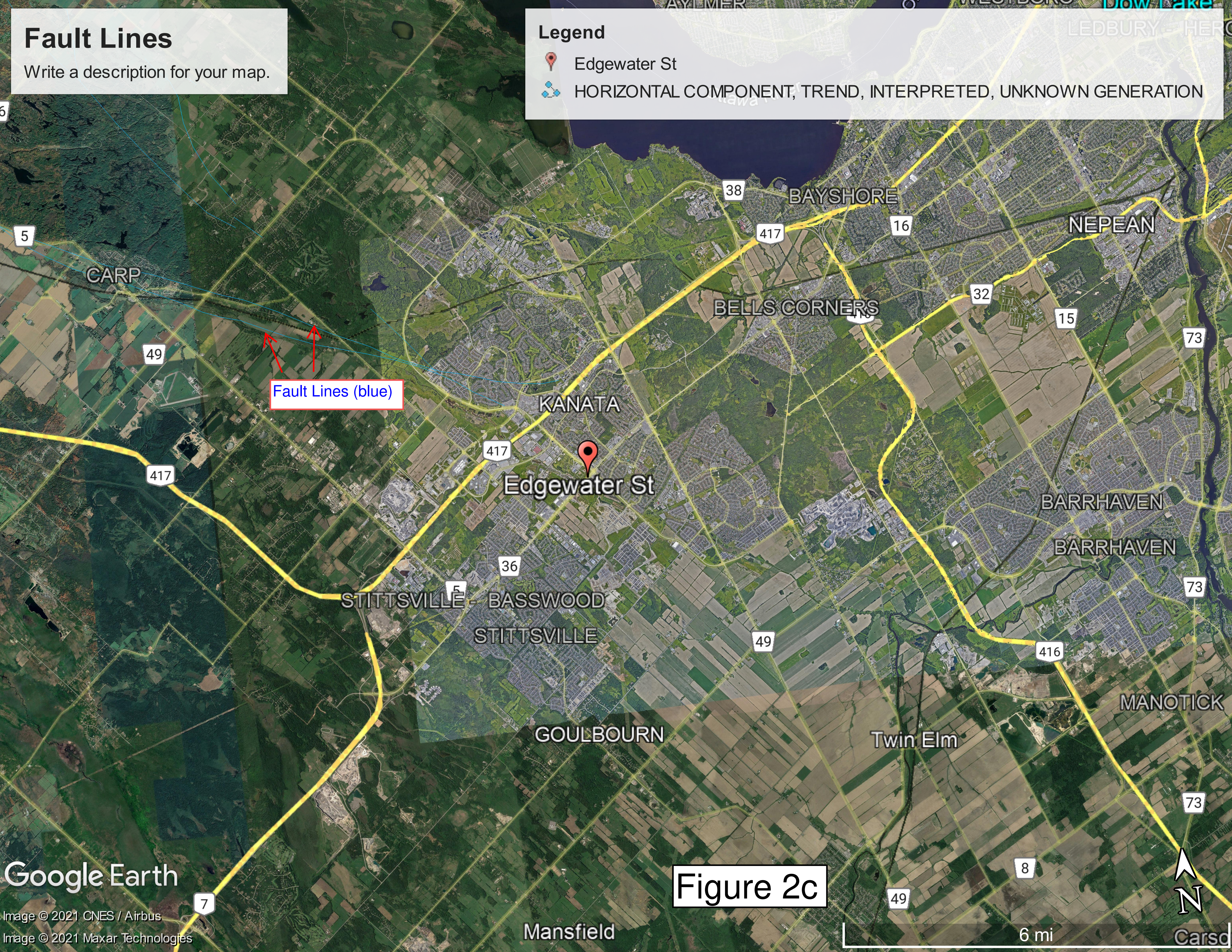
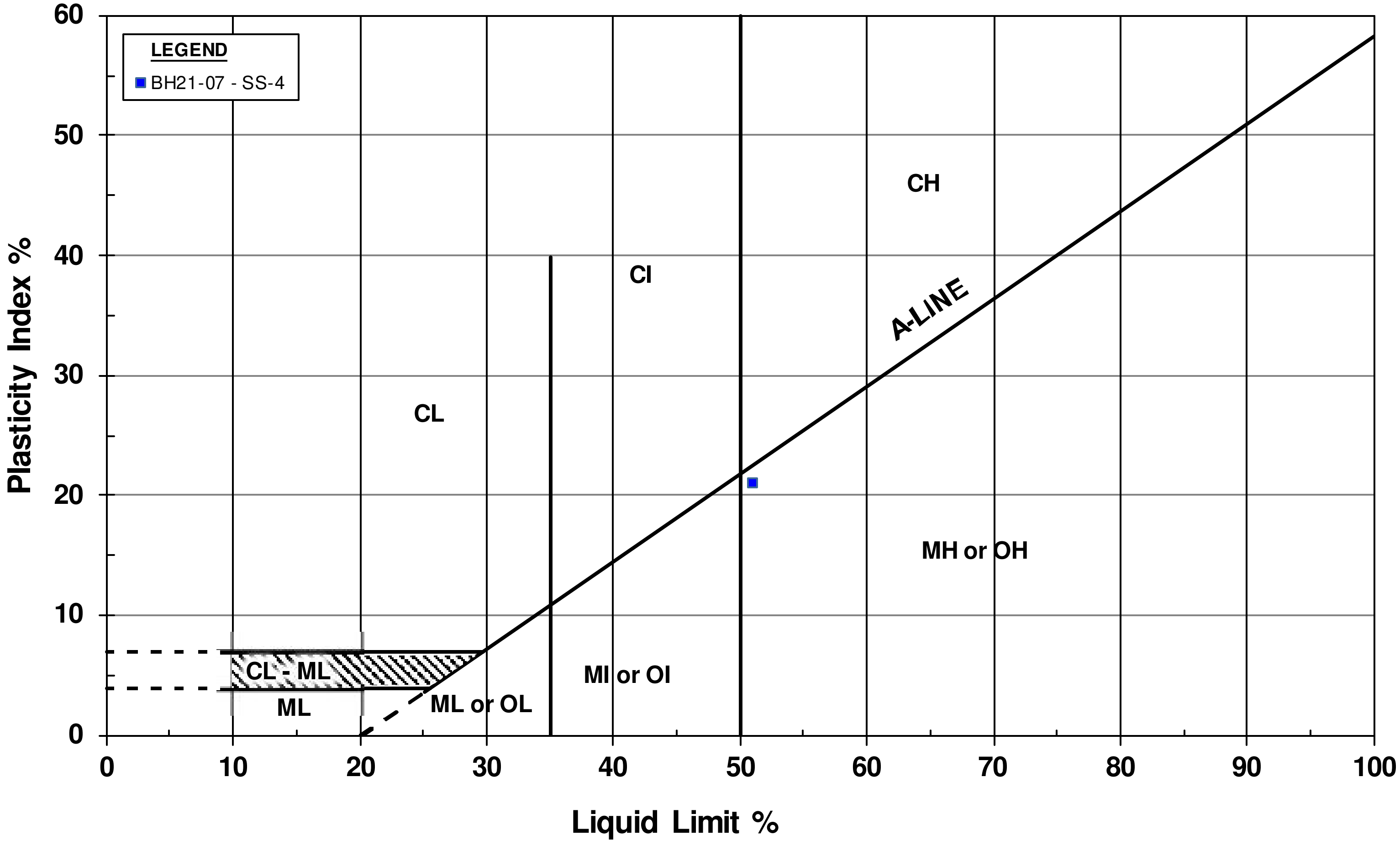

Fault Lines
Write a description for your map.
Legend
Edgewater St
HORIZONTAL COMPONENT, TREND, INTERPRETED, UNKNOWN GENERATION
Fault Lines (blue)
CARP
BAYSHORE
NEPEAN
BELLS CORNERS
KANATA
Edgewater St
BARRHAVEN
BARRHAVEN
STITTSVILLE - BASSWOOD
STITTSVILLE
GOULBOURN
Twin Elm
MANOTICK
Mansfield
417
416
38
16
32
15
73
5
49
36
7
8
6 mi
N
Google Earth
Image © 2021 CNES / Airbus
Image © 2021 Maxar Technologies

Figure 2c



McINTOSH PERRY

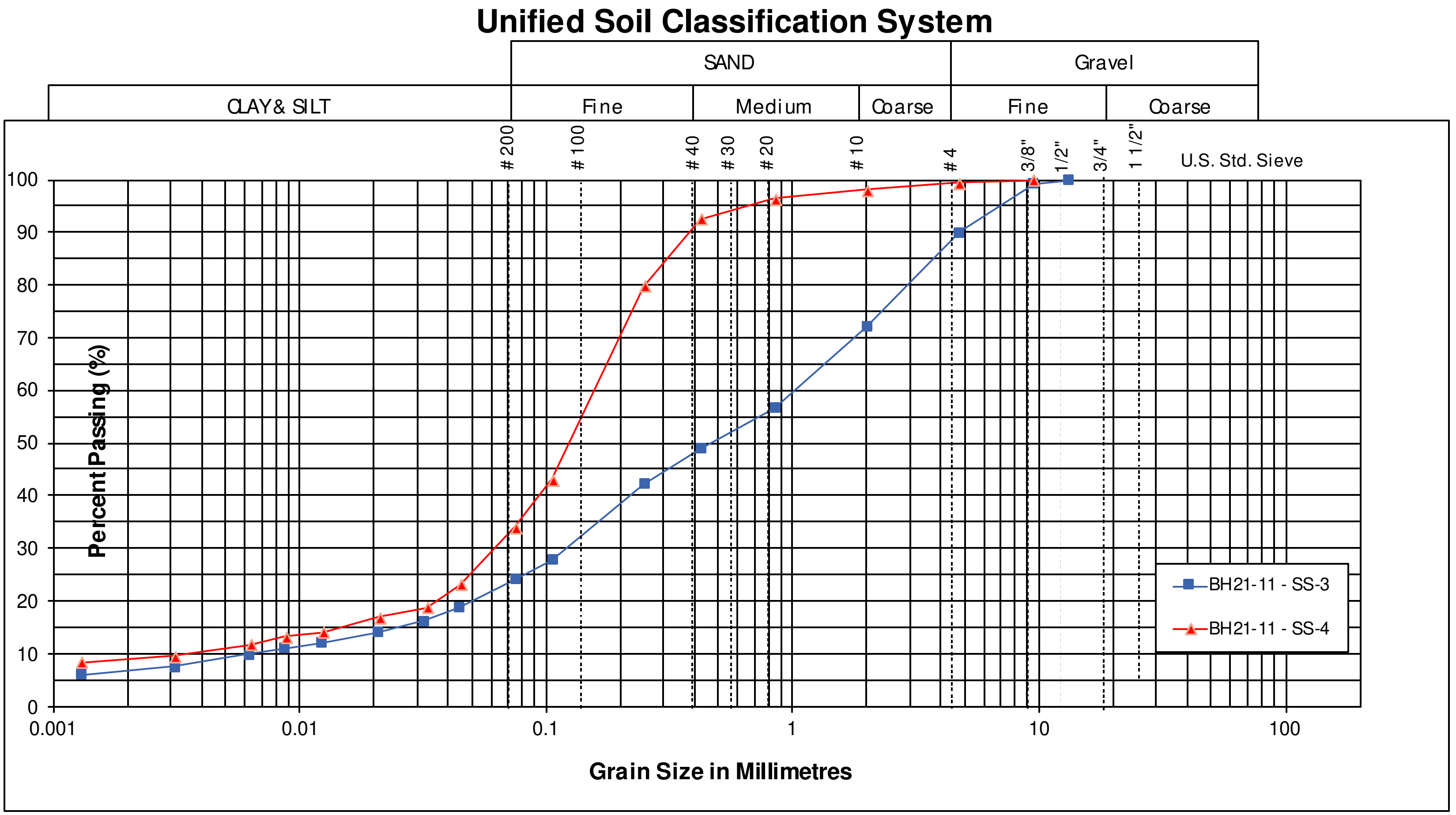
PLASTICITY CHART

16 Edgewater Street, Kanata, ON

Silty Clay/ Clayey Silt

Figure 3

CCO-22-0244



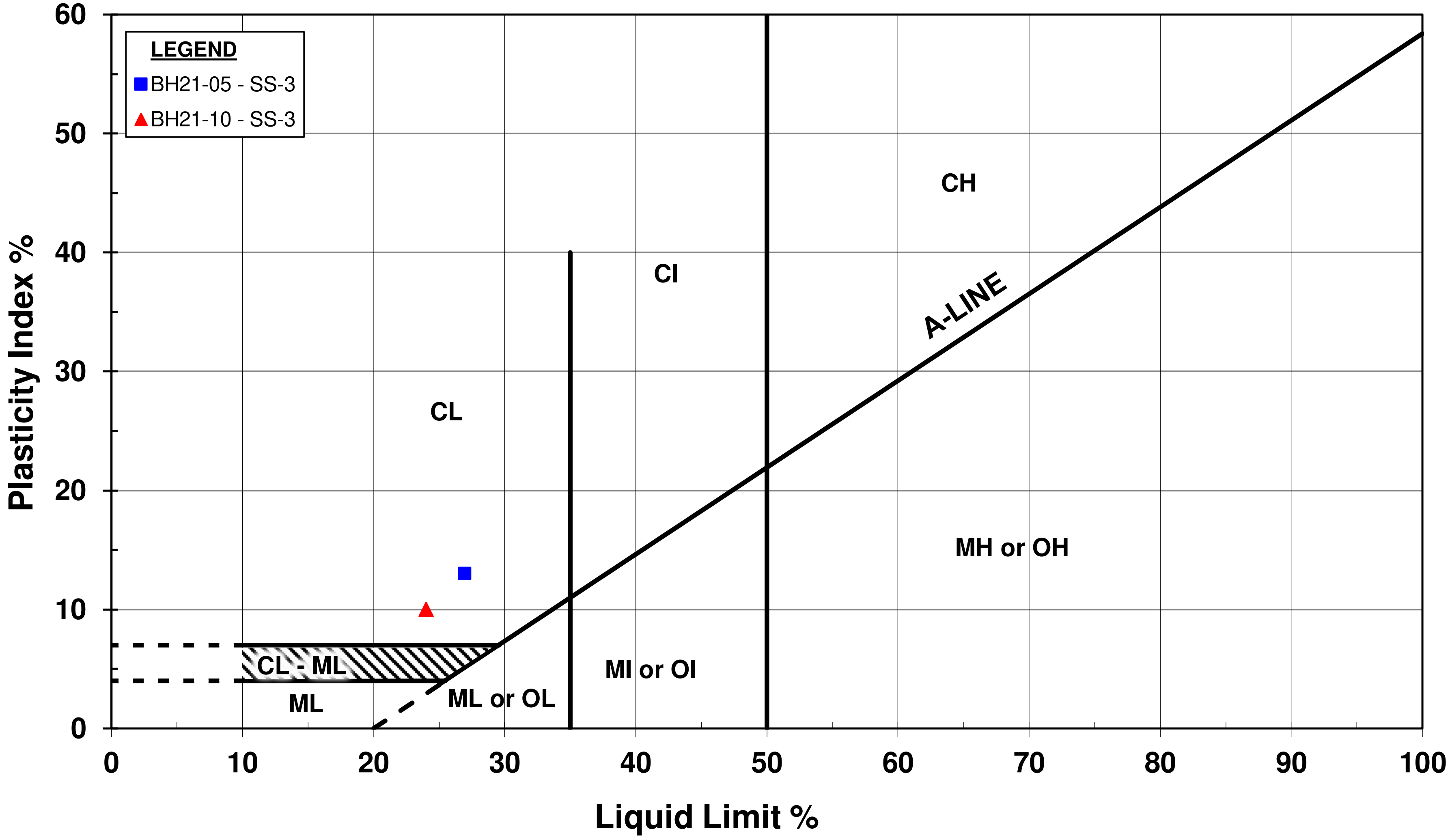
GRAIN SIZE DISTRIBUTION - Silty Sand

McINTOSH PERRY

Project Name: 16 Edgewater
Street, Kanata, ON.

Project Number: CCO-22-0244

Figure 4



McINTOSH PERRY

PLASTICITY CHART

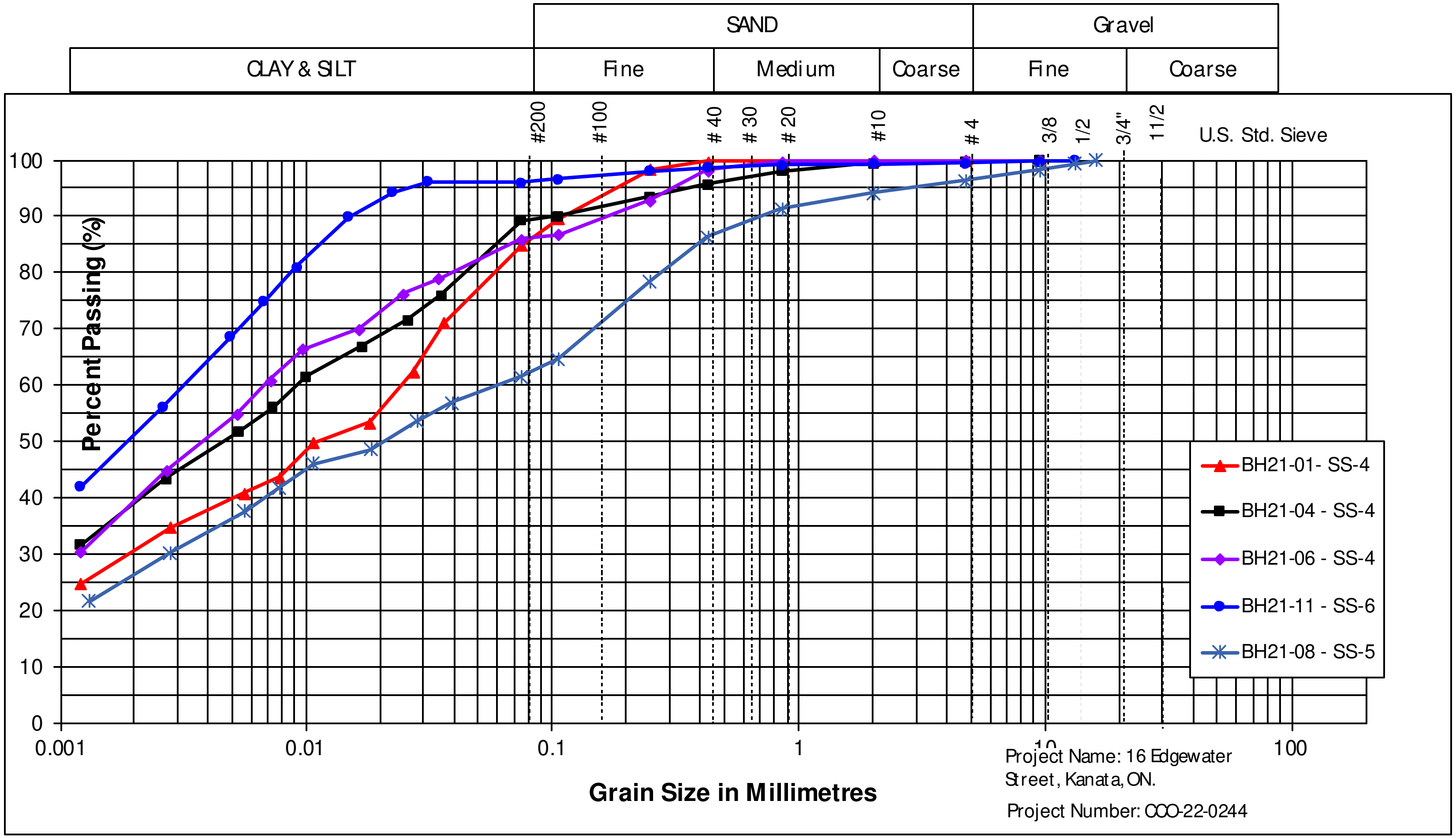
16 Edgewater Street, Kanata, ON

Clayey Sand

Figure 5

CCO-22-0244

Unified Soil Classification System



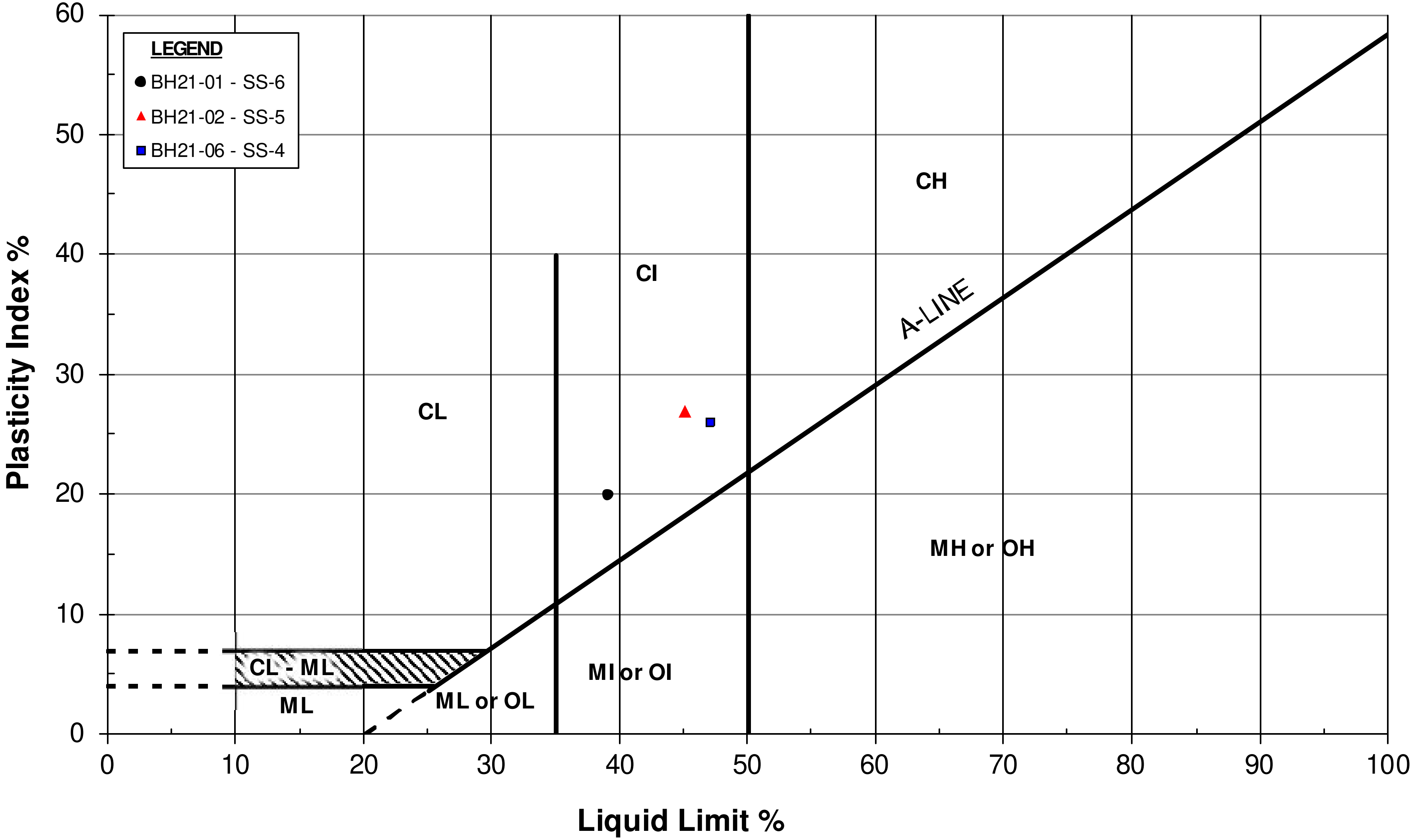
GRAIN SIZE DISTRIBUTION - Silty Clay

MCINTOSH PERRY

Project Name: 16 Edgewater
Street, Kanata, ON.

Project Number: CCO-22-0244

Figure 6



McINTOSH PERRY

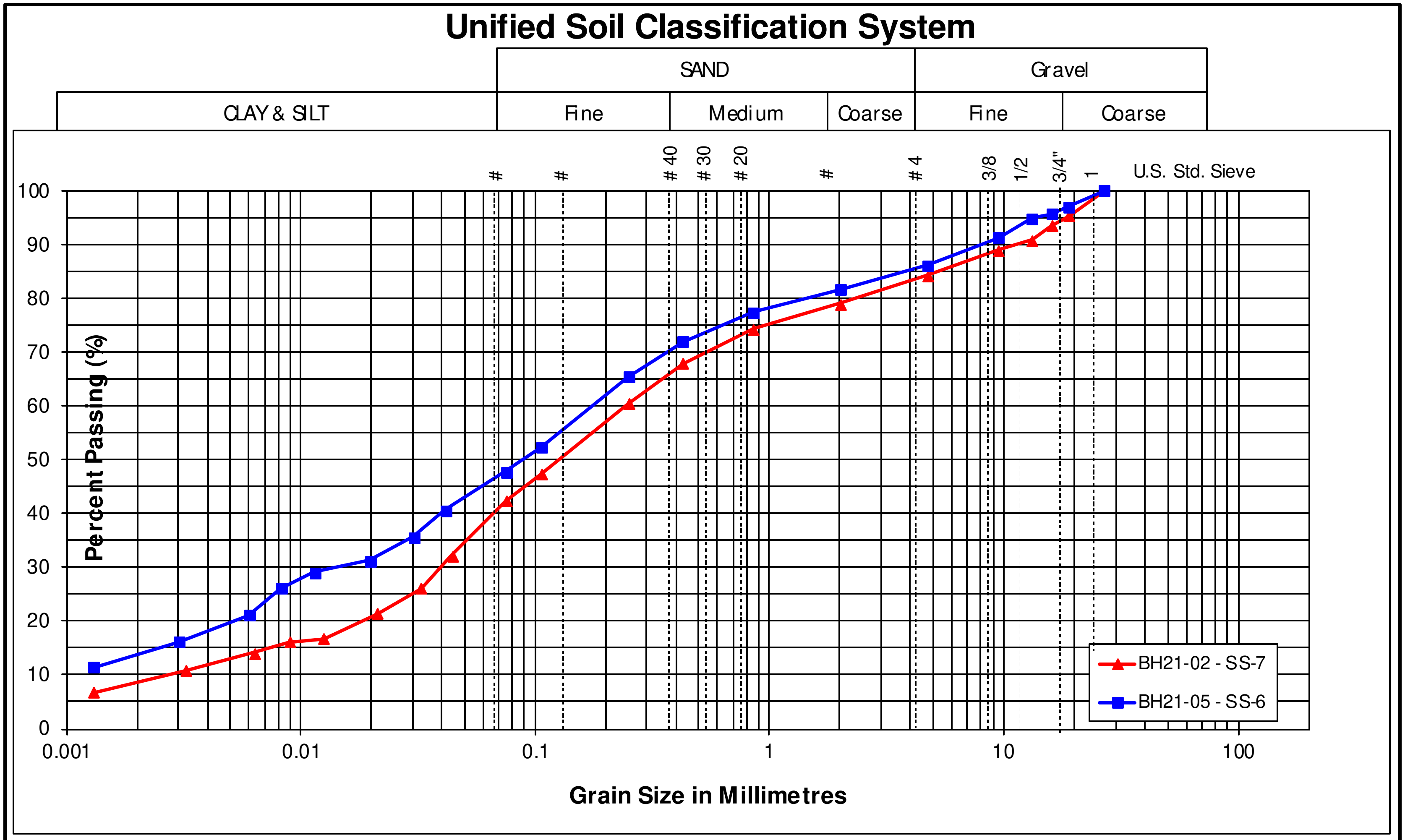
PLASTICITY CHART

16 Edgewater Street, Kanata, ON

Silty Clay

Figure No. 7

CCO-22-0244



GRAIN SIZE DISTRIBUTION - Till

McINTOSH PERRY

Project Name: 16 Edgewater
Street, Kanata, ON.

Project Number: CCO-22-0244

Figure 8

16 EDGEWATER STREET KANATA, ON

APPENDIX C
BOREHOLE LOGS

McINTOSH PERRY

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_I, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ'	$-^{o}$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	$-^{o}$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L - W_L)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_P)/ I_P$	i	1	HYDAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L - W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERED SOIL	$e_{,max}$	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

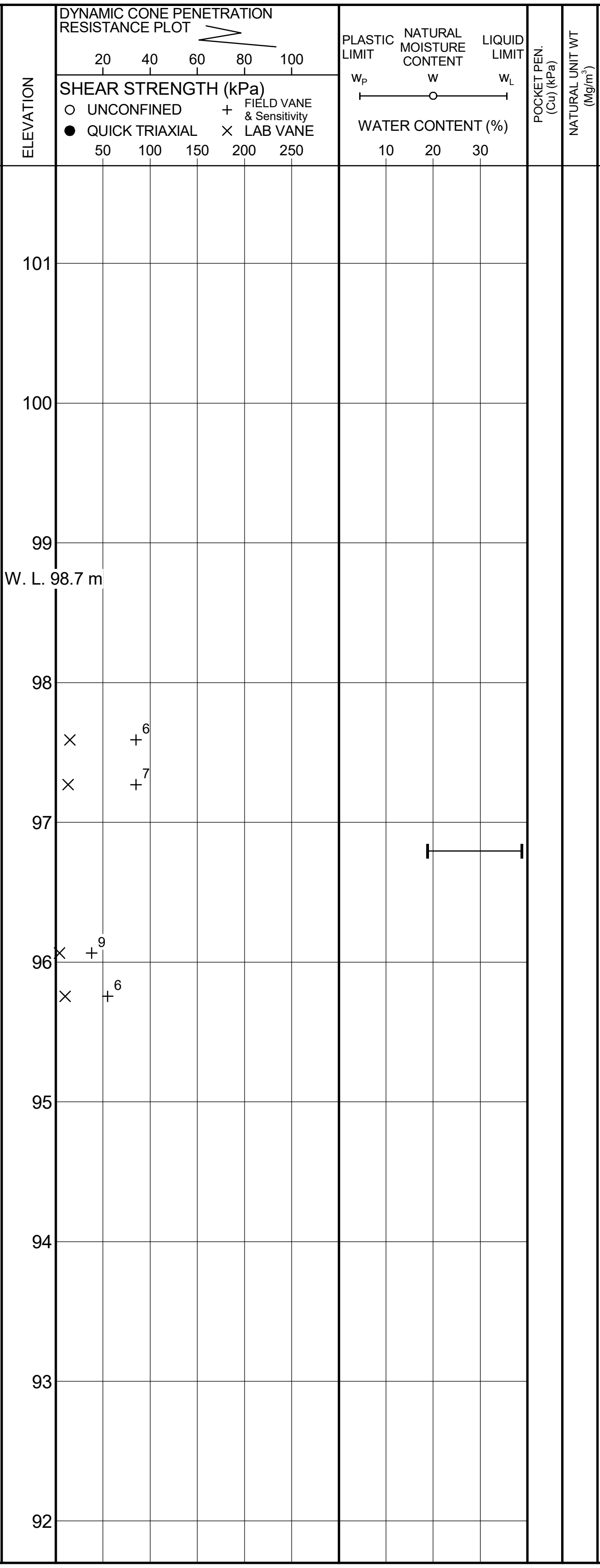
LOG OF BOREHOLE BH21-01

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017440 E 352257

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/09/2021

REF. NO.: CCO-22-0244
ENCL NO.: 1

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
101.7	Natural gound surface					
0.0	**Sand and Gravel**, some silt, traces of clay, brown, dry, compact, [Fill]	1	SS	29		
100.9 0.8	**Silty Clay**, brown, damp to moist, stiff	2	SS	7		
100.2 1.5	**Silty Clay**, some gravel, trace clay, trace sand, brown, damp to moist, stiff	3	SS	3		
99.4 2.3	**Silty Clay**, some sand, brown, damp to moist, soft	4	SS	2		0 15 54 31
98.7 3.0	**Silty Clay**, brown, moist, stiff	5	SS	1	∇ W. L. 98.7 m	Groundwater was observed in auger at 3.0 m BGS Sensitivity = 6 Sensitivity = 7
97.1 4.6	**Silty Clay**, grey, wet, firm to stiff	6	SS	WOH		Sensitivity = 9 Sensitivity = 6
95.6 6.1	**Clayey Sand**, some gravel, trace silt, gray, wet, loose to compact, presence of cobbles and boulders, [Till]	7	SS	26		Auguer grinds possibly due presence of cobbles and boulders. SPT spoon sampler refusal at 6.5 m BGS. Auger grinds.
		8	SS	58		
		9	SS	REF		
		10	SS	75		
92.7 9.0	**Weathered Limestone**	11	RC	53		SPT spoon sampler refusal at 9 m BGS.



Continued Next Page

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity   ○ $^{\varepsilon}$=3% Strain at Failure

LOG OF BOREHOLE BH21-01

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017440 E 352257

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/09/2021

REF. NO.: CCO-22-0244
ENCL NO.: 1

SOIL PROFILE			SAMPLES									
(m) ELEV / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 50 100 150 200 250	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (Mg/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Weathered Limestone(Continued)											
							91					
			12	RC	65							
							90					
89.7 12.0	**END OF BOREHOLE** **Groundwater was observed in auger @ 3.0 m BGS**											

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH21-02

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017447 E 352240

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/09/2021

REF. NO.: CCO-22-0244
ENCL NO.: 2

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE: 50 100 150 200 250
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (Mg/m³)

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
101.7	Natural gound surface						
101.6 / 0.0	**Topsoil** 50 mm						
0.1	**Sand and Gravel**, some silt, brown, dry, compact	1	SS	27			
100.9 / 0.8	**Silty Clay**, desiccated, some sand, trace gravel, brown, dry, stiff	2	SS	6			
100.2 / 1.5	**Clayey Silty Sand** (CL), trace clay, brown, moist, loose	3	SS	2	∇ W. L. 99.8 m		Groundwater was observed in open borehole on June 10, 2021 @ 1.9 m BGS.
		4	SS	2			
99.1 / 2.6	**Silty Clay**, some sand, brown, moist, firm to soft	5	SS	1		45.3	
97.1 / 4.6	**Clayey Sand**, some gravel, trace clay, brown to grey, wet, loose, inferred cobbles and boulders. - [TILL]	6	SS	3			
		7	SS	3			16 43 34 9
		8	SS	REF			
95.6 / 6.1	**Weathered Limestone**	9	RC	67			
		10	RC	67			
92.7 / 9.0	**END OF BOREHOLE** Ground water was observed inopen borehole on 10/06/2021 @ 1.9 BGS.						

ELEVATION: 101, 100, 99, 98, 97, 96, 95, 94, 93

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH21-03

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017468 E 352233

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/09/2021

REF. NO.: CCO-22-0244
ENCL NO.: 3

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE: 50 100 150 200 250
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (Mg/m^3)

SOIL PROFILE		SAMPLES					REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	ELEVATION	
102.1	Natural gound surface						
102.1 0.0	**Topsoil** 40 mm					102	
0.0	**Sandy and Gravelly Silt**, trace of clay, brown, dry, compact, presence of organic matter, - [FILL]	1	SS	24			Crushed stone in SPT spoon sampler. Auger grinds
101.2 0.9	**Silty Clay**, brown, damp, stiff	2	SS	7		101	
100.6 1.5	-- **Silty Clay**, soft	3	SS	0		100	Empty SPT spoon sampler.
99.8 2.3	**Sandy Silt** some clay, trace gravel, brown, damp to wet - [TILL]	4	SS	21			Auger grinds.
99.4 2.7	**Highly Weathered Limestone**	5	RC	70		99	SPT spoon sampler and auger refusal @ 2.74 m BGS
						98	
		6	RC	83		97	
		7	RC	100			
96.1 6.0	**END OF BOREHOLE**						

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH21-04

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017466 E 352247

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/10/2021

REF. NO.: CCO-22-0244
ENCL NO.: 4

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 50 100 150 200 250	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT w LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (Mg/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLS / 0.3 m / RQD							
102.0	Natural gound surface											
102.0 0.0	**Topsoil** ~ 40 mm **Sand and Gravel**, some silt, brown, dry, dense - [FILL]		1	SS	36							A stone in the SPT spoon tip. Auger grinds.
101.0 1.0	**Silty Clay**, brown to grey, damp, firm		2	SS	6		101					
100.6 1.4	**Clayey Sand**, some silt, brown, moist, loose		3A	SS	3							
100.2 1.8	**Silty Clay**, some sand, brown, moist, firm		3B 4	SS SS	3 1	▽ W. L. 99.7 m	100					Groundwater was observed in open borehole @ 2.3 m BGS. 0 11 50 39
99.2 2.8	**Sand**, brown, wet, compact (Till)		5	SS	REF		99					
98.9 3.1	**END OF BOREHOLE** **Inferred bedrock/boulders.** **Groundwater was observed in auger @ 3.0 m BGS.** **Groundwater was observed in open borehole on completiong @ 2.3 m BGS.**											Groundwater in auger @ 3.0 m BGS. SPT spoon sampler and auger refusal at 3.1 m BGS.

GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity ○ $^{\varepsilon}$=3% Strain at Failure

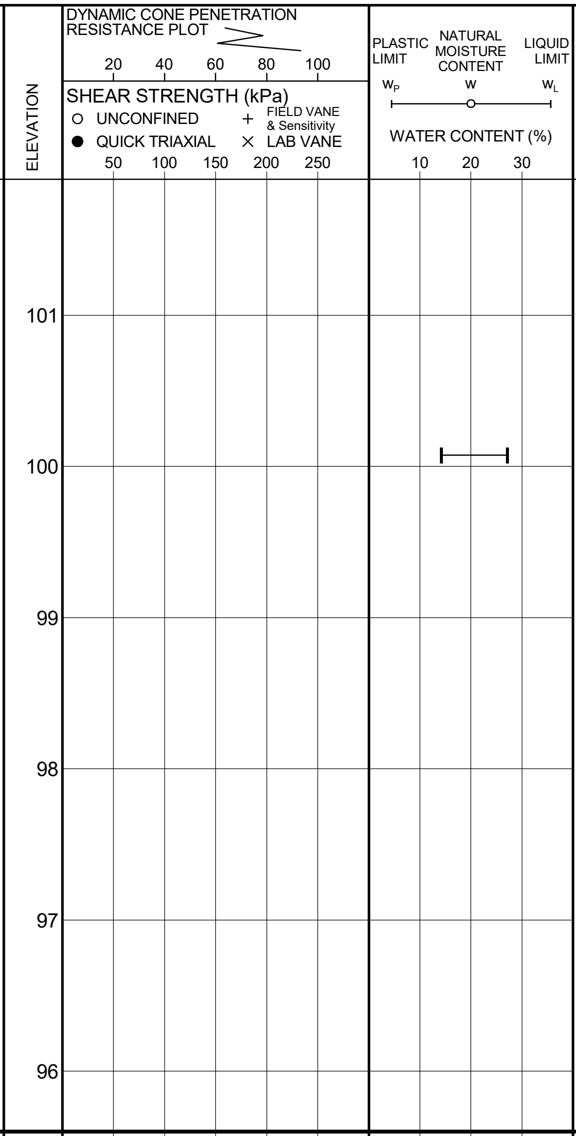
LOG OF BOREHOLE BH21-05

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017456 E 352263

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/09/2021

REF. NO.: CCO-22-0244
ENCL NO.: 5

SOIL PROFILE		SAMPLES							
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (Mg/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)	
101.9	Natural gound surface							GR SA SI CL	
0.0 101.8 0.1	**Topsoil** ~50 mm **Silty Sand and Gravel**, some clay, brown, damp, compact, presence of organic matter -[FILL]	1	SS	48					
101.1 0.8	**Clayey Silt**, trace sand, brown, dry, stiff	2	SS	11				Groundwater was observed in open borehole @ 1.1 m BGS.	
100.3 1.6	**Clayey Sand**, brown, damp to wet, loose	3	SS	3					
99.6 2.3	**Silty Clay**, some sand, brown, moist, soft to firm	4	SS	2					
98.9 3.0	-- **Clayey Silt**, trace sand, trace gravel, brown to grey, moist, soft to firm	5	SS	2				Groundwater was observed in auger @ 3.0 m BGS. Vane push refusal	
97.9 4.0	**Silty Sand**, some clay, some gravel, grey to reddish brown, wet, loose to compact. Presence of cobbles and boulders. - [TILL]	6	SS	7				14 38 33 15	
		7	SS	6				Auger grinds from 4.0 m BGS to the EOB.	
		8	SS	22					
95.6		9	SS	REF					
6.3	**END OF BOREHOLE** **Inferred bedrock/boulders.** **Groundwater was observed in open borehole @ 1.1 m BGS.** **Groundwater was observed in auger @ 3.0 m BGS.**								



GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity   ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH21-06

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017460 E 352284

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/11/2021

REF. NO.: CCO-22-0244
ENCL NO.: 6

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE: 50 100 150 200 250
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30
POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (Mg/m^3)

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLS / 0.3 m	ELEVATION	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
101.9	Natural gound surface						
101.8 0.1	**Topsoil** ~ 50 mm **Sand and Gravel** trace silt, brown, dry, compact - [Fill]	1	SS	37			Auger grinds
101.1 0.8	**Silty Clay** trace sand, brown dry to damp, firm	2	SS	8	101		
100.3 1.6	**Silty Clay**, some sand, brown, wet to moist, firm to soft	3	SS	3	100		
		4	SS	2	99	46.6	0 13 47 40
		5	SS	1			
98.1 3.8	**Clayey Sand**, some gravel, grey, wet, soft to firm Presence of cobbles and boulders - [TILL]	6	SS	WOH	98		Groundwater was observed in auger @ 3.7 m BGS.
		7	SS	5	97		
96.7 5.2	**END OF BOREHOLE** **Inferred bedrock/boulders** **Groundwater was observed inauger @ 3.7 m BGS.**						SPT spoon sampler and auger refusal @ 5.2 m BGS.

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH21-07 MW

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017481 E 352271

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/10/2021

REF. NO.: CCO-22-0244
ENCL NO.: 7

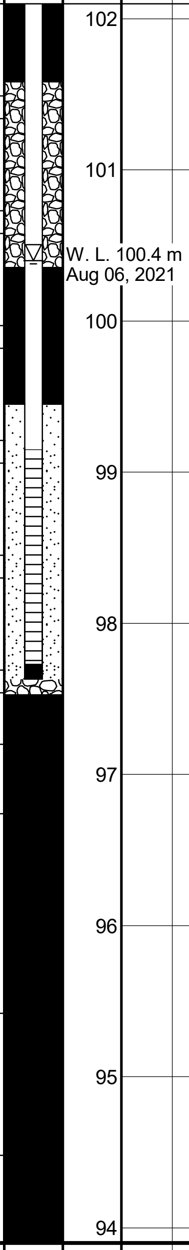
SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION
102.1	Natural gound surface
102.1 0.0	**Topsoil** ~ 25 mm **Sand and Gravel**, some silt, trace clay, brown, dry , compact to dense. Presence of organic matter [FILL]
101.3 0.8	**Silty Clay**, brown, damp to moist, soft to stiff
99.0 3.1	**Clayey Sand**, brown to grey, wet, loose
98.3 3.8	**Silty and Sandy Clay**, some gravel, grey, wet, firm - [Till]
97.5 4.6	**Silty Sand and Gravel**, grey, wet, compact, [Till]
97.2 4.9	**Hightly Weathered Limestone** Vertical cracks.
93.9 8.2	**END OF BOREHOLE** **Groundwater was observed in the well on June 11, 2021 @ 2.6 m BGS.**

SAMPLES

NUMBER	TYPE	"N" BLS / 0.3 m / RQD
1	SS	49
2	SS	8
3	SS	4
4	SS	1
5	SS	5
6	SS	5
7	SS	14
8	RC	0
9	RC	90
10	RC	65
11	RC	52

GROUND WATER CONDITIONS



DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED; + FIELD VANE & Sensitivity; ● QUICK TRIAXIAL; × LAB VANE; 50 100 150 200 250

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

Sample 4: 51

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (Mg/m^3): 67.0

REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

The well is protected with a stickup case.

Stone in the SPT tip. Auger grinds.

Black stain in the sample (smells/oudor)

Empty SPT spoon sampler (SS-03)

Groundwater was observed in the well on June 11, 2021 @ 2.6 m BGS.

SPT spoon sampler and auger refusals @ 4.9 m BGS.

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ^{8}=3% Strain at Failure

LOG OF BOREHOLE BH21-08

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017500 E 352264

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/10/2021

REF. NO.: CCO-22-0244
ENCL NO.: 8

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE: 50 100 150 200 250
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (Mg/m³)

SOIL PROFILE (m) ELEV / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
102.5	Natural gound surface						
0.0	**Topsoil** ~50 mm						
102.4 0.1	**Sand and Gravel**, some silt, brown, dry. Presence of organic matter. [FILL]	1	SS	24		102	A stone in the SPT tip
101.7 0.8	**Clayey Silt**, some sand, brown, damp, firm to stiff	2	SS	6			
101.1 1.4	**Silty Clay**, brown, firm to soft	3	SS	2		101	
		4	SS	WOH		100	
99.2 3.3	**Clayey Sand**, some gravel, trace clay, grey, moist to wet, compact. Presence of cobbles and boulder - [TILL]	5	SS	WOH		99	3 36 34 27 Auger grinds
		6	SS	27		98	
97.9 4.6	**Highly Weathered Limestone**	7	RC	77		97	SPT spoon sampler and auger refusals @ 4.6 m BGS.
		8	RC	0			
		9	RC	10		96 95	
94.8 7.7	**END OF BOREHOLE**						

GRAPH NOTES: + ³, × ³: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH21-09

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017478 E 352293

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/11/2021

REF. NO.: CCO-22-0244
ENCL NO.: 9

SOIL PROFILE (m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
102.1	Natural gound surface						
102.0 0.1	**Topsoil** ~ 75 mm **Sand and Gravel**, brown to dark brown, dry, compact, [FILL]	1	SS	20		102	
101.5 0.6	**Silty Clay**, trace sand, brown, damp, firm to stiff	2	SS	6		101	
100.9 1.2	**Silty Sand**, brown, damp						
100.6 1.5	**Silty Clay**, trace sand, brown, moist, firm	3	SS	3			
100.0 2.1	**Clayey Sand**, some gravel, brown, moist to wet, compact, [Till] Precence of cobbles and boulders	4	SS	REF		100	Shattered stone in the SPT tip.
		5	SS	43		99	SPT spoon sampler refusal @ 2.7 m BGS. Auger grinds. Crushed and shattered stone in SPT spoon and tip. Auger grinds and rattles. Groundwater was observed in auger @ 3.8 m BGS.
98.3 3.8	**Sand and Gravel**, trace clay, trace silt, brown, wet, [Till]	6	SS	REF	⊻	W. L. 98.3 m	
98.1 4.0	**Highly Weathered Limestone**	7	RC	26		98	
		8	RC	59		97	
		9	RC	63		96 95	
94.7 7.4	**END OF BOREHOL** **Groundwater was observed in auger @ 3.8 m BGS.**						

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED + FIELD VANE & Sensitivity
● QUICK TRIAXIAL × LAB VANE
50 100 150 200 250

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%)
10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (Mg/m^3)

GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity ○ $^{\varepsilon}$=3% Strain at Failure

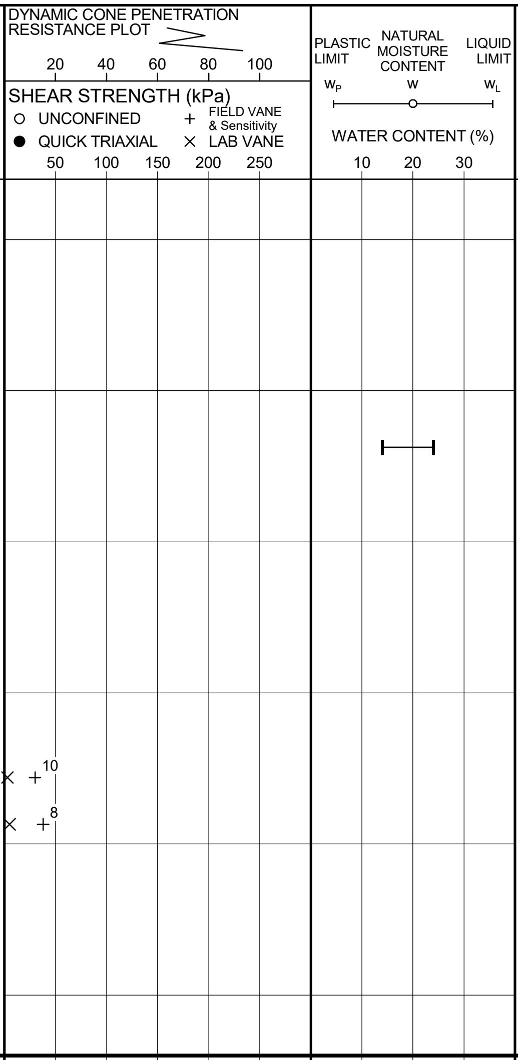
LOG OF BOREHOLE BH21-10

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017499 E 352297

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/11/2021

REF. NO.: CCO-22-0244
ENCL NO.: 10

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION	NATURAL UNIT WT (Mg/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLS / 0.3 m / RQD				
102.4	Natural gound surface							
0.0	**Topsoil** ~ 50 mm							
102.3 0.1	**Sand and Gravel**, some silt, brown, dry, compact, [FILL]	1	SS	13		102		
101.8 0.6	**Silty Clay**, trace sand, brown, damp	2A	SS	4				
101.4 1.0	**Clayey Sand**, brown, moist, loose	2B	SS	4		101		
		3	SS	1			32.0	
100.0 2.4	**Silty Clay**, some sand, grey, wet, soft	4	SS	WOH		100		Driller added water to facilitate drilling
		5	SS	WOH		99		
			VANE					Sensitivity = 10
			VANE			98		Sensitivity = 8
		6	SS	1				
97.1 5.3	**Sand and Gravel**, trace clay, trace silt, compact, [TILL] Presence of cobbles and boulders	7	SS	REF		97		Auger grinds SPT spoon sampler refusal @ 5.5 m BGS.
96.6 5.8	**END OF BOREHOLE** **Inferred bedrock/boulders**							Rock in SPT tip. Auger refusal @ 5.8 m BGS.



POCKET PEN. (Cu) (kPa)

GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

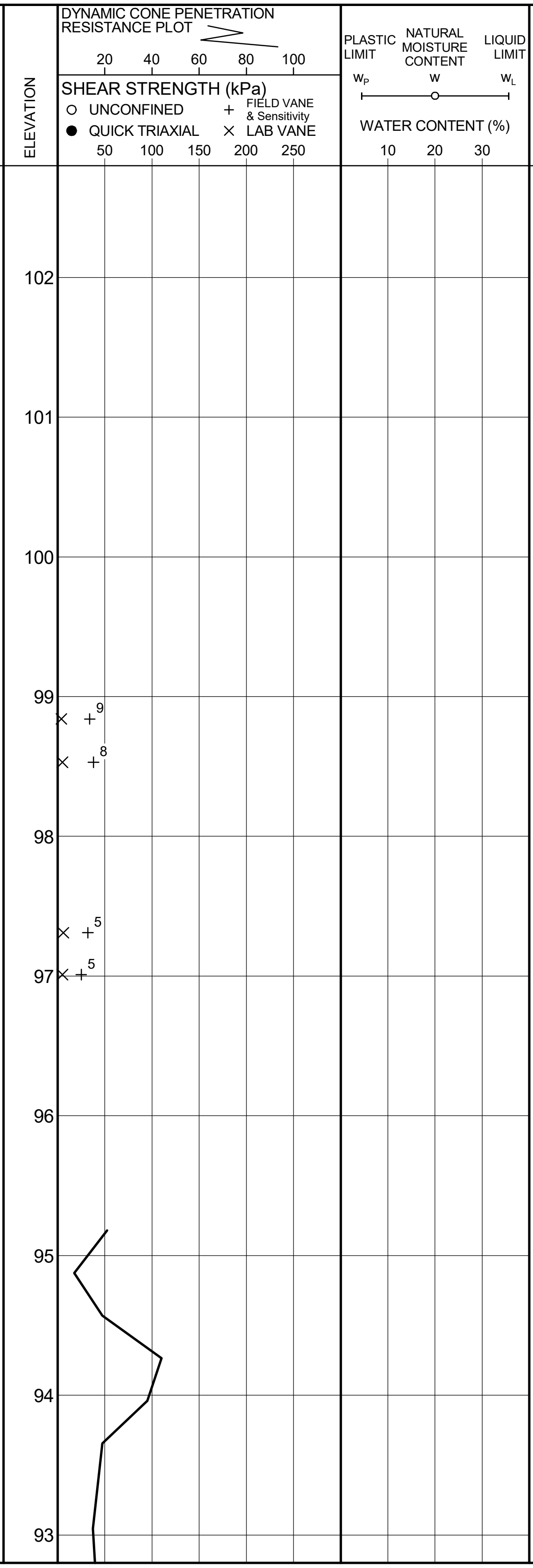
LOG OF BOREHOLE BH21-11

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017519 E 352276

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/11/2021

REF. NO.: CCO-22-0244
ENCL NO.: 11

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (Mg/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)			
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLS / 0.3 m / RQD				GR	SA	SI	CL
102.8	Natural gound surface										
0.0	**Topsoil** ~ 50 mm										
102.7 0.1	**Sand and Gravel**, some silt, brown, dry, compact, (FILL)	1	SS	28							
102.0 0.8	**Clayey Silt**, some sand, trace gravel, brown, damp, soft to firm	2	SS	5				White substance/ debris. Could be gypsum.			
101.1 1.7	**Silty Sand**, trace gravel, brown, wet, loose	3	SS	7				10	66	18	7
		4	SS	7				1	65	25	9
								Groundwater was observed in auger @ 2.6 m BGS.			
99.7 3.1	**Silty Clay**, trace sand, gray, wet, soft	5	SS	1							
			VANE					Sensitivity = 9			
			VANE					Sensitivity = 8			
		6	SS	WOH				1	4	45	52
			VANE					Sensitivity = 5			
			VANE					Sensitivity = 5			
96.7 6.1	**Sand and Gravel**, trace silt, grey, wet, very loose (Till)	7	SS	WOH							
		8	SS	1							
95.3 7.5	**Inferred Till**										



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GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

LOG OF BOREHOLE BH21-11

PROJECT: RESIDENTIAL HOUSING
CLIENT: McCluskey Group
PROJECT LOCATION: 16 Edgewater Street, Kanata, ON.
DATUM: MTM Zone 9
BH LOCATION: 16 Edgewater Street, Kanata, ON. N 5017519 E 352276

DRILLING DATA
Method: Split Spoon Auger
Diameter: 200
Date: Jun/11/2021

REF. NO.: CCO-22-0244
ENCL NO.: 11

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 50 100 150 200 250	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30			
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	ELEVATION			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (Mg/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
91.7	**Inferred Till**(Continued)						92					
11.1	**END OF BOREHOLE** **Inferred bedrock/boulders.** **Groundwater was observed in auger @ 2.6 m BGS.**											DCPT refusal

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity   ○ ε=3% Strain at Failure

16 EDGEWATER STREET KANATA, ON

APPENDIX D
ROCK CORES

McINTOSH PERRY

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO-22-0244	**Date Issued:**	June 30,2021
Lab No.:	OL-21037	**Report No.:**	OL-1
Project Name:	Geotech Investigation - 16 Edgewater, Kanata, ON		

Core No.:	1	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-01	**Run:**	RC-12	**Depth (ft):**	35'9"-36'2"
Date Sampled:	06/09/21	**Received:**	06/30/21	**Tested:**	07/06/21
Core No.:	2	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-02	**Run:**	RC-9	**Depth (ft):**	22'2"-22'7"
Date Sampled:	06/09/21	**Received:**	06/30/21	**Tested:**	07/06/21
Core No.:	3	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-3	**Run:**	RC-5	**Depth (ft):**	12'5"-12'10"
Date Sampled:	06/09/21	**Received:**	06/30/21	**Tested:**	07/06/21

Core No. :	1	2	3
Diameter (mm)	44.9	45.0	45.0
Thickness/Height (mm)	108.5	112.0	109.4
Density (Kg/m^3)	2716	2702	2696
Compressive Strength (Mpa)	197.3	180.6	150.3
Corr. Compressive Strength (Mpa)	N/A	N/A	N/A
Description of Failure	Type 4	Type 2	Type 2

Remarks: *Note - Type 2 fracture: Well-formed cone on one end, vertical cracks running through ends, no-well defined cone on the other end.

*Note - Type 4 fracture: Diagonal fracture with no to slight cracking through ends.

Reviewed By: **Date:** July 7,2021

Jason Hopwood-Jones
Laboratory Manager

Unconfined Compressive Strength of Intact Rock Cores
ASTM D7012 Method C

Project No.:	CCO-22-0244	**Date Issued:**	June 30,2021
Lab No.:	OL-21037	**Report No.:**	OL-1
Project Name:	Geotech Investigation - 16 Edgewater, Kanata, ON		

Core No.:	4	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-07	**Run:**	RC-9	**Depth (ft):**	19'1"-19'6"
Date Sampled:	06/09/21	**Received:**	06/30/21	**Tested:**	07/06/21
Core No.:	5	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-08	**Run:**	RC-7	**Depth (ft):**	17'2"-17'7"
Date Sampled:	06/09/21	**Received:**	06/30/21	**Tested:**	07/06/21
Core No.:	6	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-9	**Run:**	RC-8	**Depth (ft):**	16'0"-16'5"
Date Sampled:	06/09/21	**Received:**	06/30/21	**Tested:**	07/06/21

Core No. :	4	5	6
Diameter (mm)	45.0	44.9	45.0
Thickness/Height (mm)	109.4	109.0	108.7
Density (Kg/m^3)	2681	2634	2678
Compressive Strength (Mpa)	118.5	72.8	88.1
Corr. Compressive Strength (Mpa)	N/A	N/A	N/A
Description of Failure	Type 4	Type 4	Type 1

Remarks: *Note: Microcracks throughout number 6 rock core.

*Note - Type 4 fracture: Diagonal fracture with no to slight cracking through ends.

*Note: Rock core # 4 had a seem of quarts on a diagonal plane at failure.

Reviewed By: **Date:** 07-Jul-21

Jason Hopwood-Jones
Laboratory Manager

McINTOSH PERRY

Client: McCluskey Group
Project: 16 Edgewater Street, Kanata, ON.

BH21-01 **Core#:** RC – 11 **Run:** 9.0 m – 10.5 m
BH21-01 **Core#:** RC – 12 **Run:** 10.5 m – 12.0 m

Project No.: CCO-22-0244

McINTOSH PERRY

Client: McCluskey Group

Project: 16 Edgewater Street, Kanata, ON. .

BH21-02 **Core#:** RC – 09 **Run:** 6.1 m – 7.4 m

BH21-02 **Core#:** RC – 10 **Run:** 7.4 m – 9.0 m

Project No.: CCO-22-0244

McINTOSH PERRY

Client: McCluskey Group

Project: 16 Edgewater Street, Kanata, ON.

BH21-03 **Core#:** RC – 05 **Run:** 2.7 m – 4.2 m
BH21-03 **Core#:** RC – 06 **Run:** 4.2 m – 5.8 m
BH21-03 **Core#:** RC – 07 **Run:** 5.8 m – 6.0 m

Project No.: CCO-22-0244

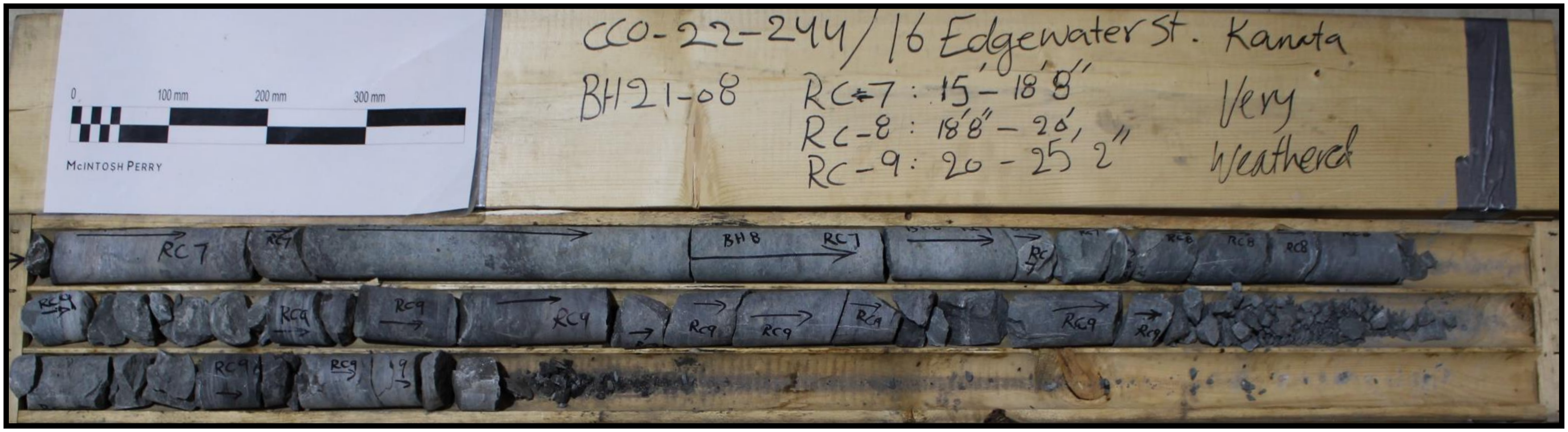
McINTOSH PERRY

Client: McCluskey Group

Project: 16 Edgewater Street, Kanata, ON.

BH21-07 **Core#:** RC – 08 **Run:** 4.9 m – 5.4 m
BH21-07 **Core#:** RC – 09 **Run:** 5.4 m – 6.7 m
BH21-07 **Core#:** RC – 10 **Run:** 6.7 m – 7.6 m
BH21-07 **Core#:** RC – 10 **Run:** 7.6 m – 8.2 m

Project No.: CCO-22-0244



McINTOSH PERRY

Client: McCluskey Group

Project: 16 Edgewater Street, Kanata, ON.

BH21-08 **Core#:** RC – 07 **Run:** 4.6 m – 5.7 m
BH21-08 **Core#:** RC – 08 **Run:** 5.7 m – 6.1 m
BH21-08 **Core#:** RC – 09 **Run:** 6.1 m – 7.7 m

Project No.: CCO-22-0244

McINTOSH PERRY

Client: McCluskey Group

Project: 16 Edgewater Street, Kanata, ON.

BH21-09	**Core#:** RC – 07	**Run:** 4.0 m – 4.8 m
BH21-09	**Core#:** RC – 08	**Run:** 4.8 m – 6.0 m
BH21-09	**Core#:** RC – 09	**Run:** 6.0 m – 7.4 m

Project No.: CCO-22-0244

16 EDGEWATER STREET KANATA, ON

APPENDIX E
LAB RESULTS

McINTOSH PERRY

PARACEL LABORATORIES LTD.
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1-800-749-1947
www.paracellabs.com

Certificate of Analysis

McIntosh Perry Consulting Eng. (Nepean)

215 Menten Place, Unit 104
Nepean, ON K2H 9C1
Attn: Jason Hopwood-Jones

Client PO: CCO-22-0244
Project: 16 Edgewater
Custody: 128753

Report Date: 12-Aug-2021
Order Date: 6-Aug-2021

Order #: 2133058

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	**Client ID**
2133058-01	BH21-02 SS-05
2133058-02	BH21-10 SS-05

Approved By:

Dale Robertson, BSc
Laboratory Director

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	10-Aug-21	10-Aug-21
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	12-Aug-21	12-Aug-21
Resistivity	EPA 120.1 - probe, water extraction	11-Aug-21	12-Aug-21
Solids, %	Gravimetric, calculation	10-Aug-21	10-Aug-21

Certificate of Analysis

Report Date: 12-Aug-2021

Client: **McIntosh Perry Consulting Eng. (Nepean)**

Order Date: 6-Aug-2021

Client PO: CCO-22-0244

Project Description: 16 Edgewater

	Client ID:	BH21-02 SS-05	BH21-10 SS-05	-	-
	Sample Date:	09-Jun-21 09:00	09-Jun-21 09:00	-	-
	Sample ID:	2133058-01	2133058-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	61.9	60.4	-	-
General Inorganics					
pH	0.05 pH Units	7.70 [1]	7.92 [1]	-	-
Resistivity	0.10 Ohm.m	23.8	15.2	-	-
Anions					
Chloride	5 ug/g dry	141 [1]	12 [1]	-	-
Sulphate	5 ug/g dry	74 [1]	471 [1]	-	-

Qualifier Notes:

Login Qualifiers :

Received at temperature > 25C

Applies to samples:

Sample - One or more parameter received past hold time - Chloride, pH, sulphate

Applies to samples: BH21-02 SS-05, BH21-10 SS-05

Sample Qualifiers :

1 : Holding time had been exceeded upon receipt of the sample at the laboratory.

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

16 EDGEWATER STREET KANATA, ON

APPENDIX F
SEISMIC HAZARD CALCULATION

McINTOSH PERRY

2010 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

Site: 45.295N 75.895W **User File Reference:** 16 Edgewater Street, Kanata, ON 2021-08-12 14:07 UT

Requested by: McIntosh Perry

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.2)	0.619	0.376	0.240	0.085
Sa (0.5)	0.299	0.181	0.119	0.042
Sa (1.0)	0.134	0.085	0.054	0.017
Sa (2.0)	0.045	0.027	0.017	0.006
PGA (g)	0.316	0.195	0.118	0.036

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" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. 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.**

References

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

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
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

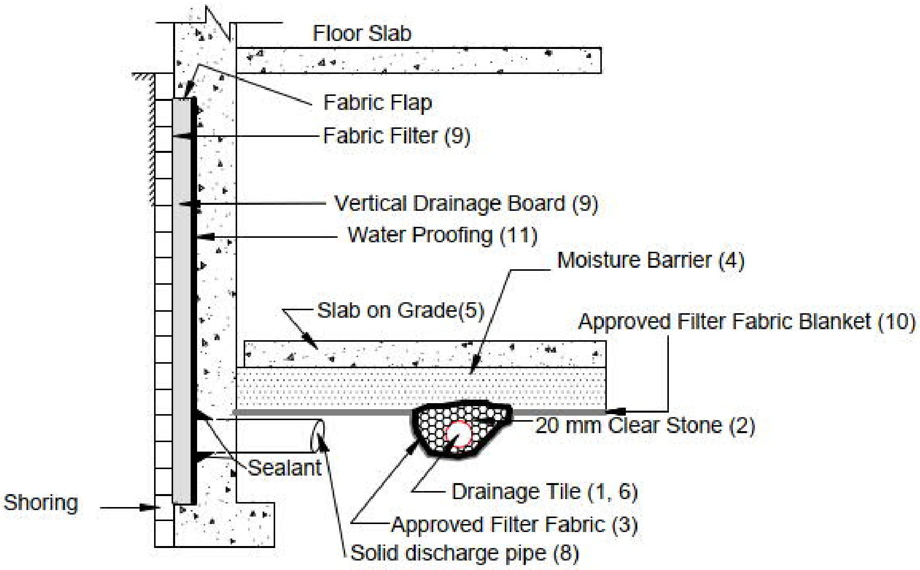
Natural Resources Canada Ressources naturelles Canada

Canada

16 EDGEWATER STREET KANATA, ON

APPENDIX G
DRAINAGE DETIALS

McINTOSH PERRY



EXTERIOR FOOTING

Notes

1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place100 mm (4 inches) of stone below drain .
3. Wrap the clear stone with an approved filter membrane (Terrafix 270R or equivalent).
4. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for specialty floors.
5. Slab on grade should not be structurally connected to the wall or footing.
6. Underfloor drain invert to be at least 300 mm (12") below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25') centers one way. Place drain on 100 mm (4") clear stone with 150 mm (6") of clear stone on top and sides. Enclose stone with filter fabric as noted in (3).
7. Do not connect the underfloor drains to perimeter drains.
8. Solid discharge pipe located at the middle of each bay between the solider piles, approximate spacing 2.5 m, outletting into a solid pipe leading to a sump.
9. Vertical drainage board with filter cloth should be kept a minium of 1.2 m below exterior finished grade.
10. The entire subgrade to be sealed with approved filter fabric (Terrafix 270R or equivalent) if non-cohesive (sandy) soils below ground water table encountered.
11. The basement walls should be water proofed using bentonite or equivalent water-proofing system.
12. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.

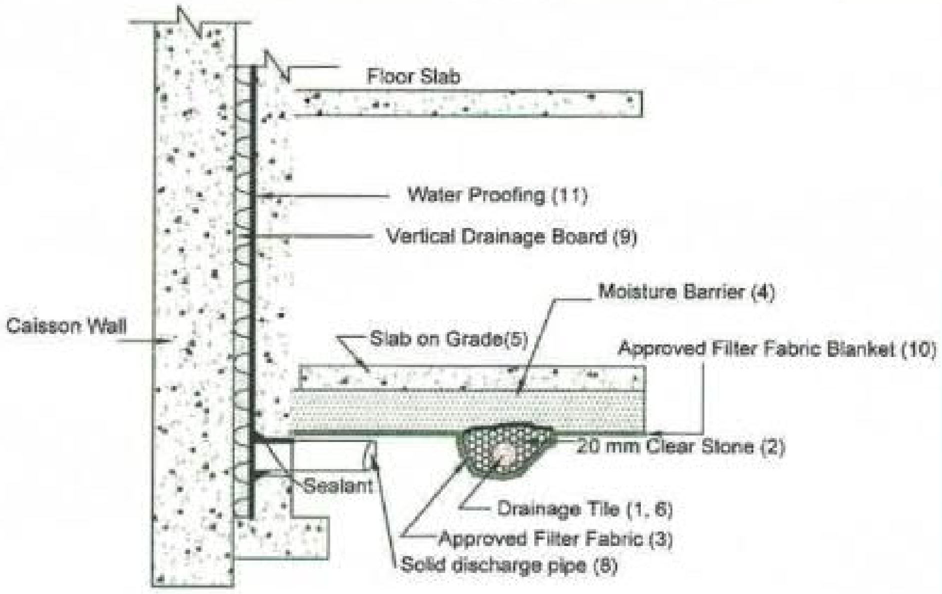
Drainage Recommendations
Shored Basement Wall with Underfloor Drainage System
(not to scale)

McINTOSH PERRY

Project Name: 16 Edgewater Street, Kanata, ON.

Project Number: CCO-22-0244

Figure G-1



EXTERIOR FOOTING

Notes

1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place100 mm (4 inches) of stone below drain .
3. Wrap the clear stone with an approved filter membrane (Terrafix 270R or equivalent).
4. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for specialty floors.
5. Slab on grade should not be structurally connected to the wall or footing.
6. Underfloor drain invert to be at least 300 mm (12") below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25') centers one way. Place drain on 100 mm (4") clear stone with 150 mm (6") of clear stone on top and sides. Enclose stone with filter fabric as noted in (3).
7. Do not connect the underfloor drains to perimeter drains.
8. Solid discharge pipe located at the middle of each bay between the solider piles, approximate spacing 2.5 m, outletting into a solid pipe leading to a sump.
9. Vertical drainage board mira-drain 6000 or eqivalent with filter cloth should be continous from bottom to 1.2 m below exterior finished grade.
10. The entire subgrade to be sealed with approved filter fabric (Terrafix 270R or equivalent) if non-cohesive (sandy) soils below ground water table encountered.
11. The basement walls must be water proofed using bentonite or equivalent water-proofing system.
12. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.

Drainage Recommendations
Shored Basement Wall with Underfloor Drainage System
(not to scale)

McINTOSH PERRY

Project Name: 16 Edgewater Street, Kanata, ON.

Project Number: CCO-22-0244

Figure G-2

16 EDGEWATER STREET KANATA, ON

APPENDIX H
GEOTECHNICAL INVESTIGATION - 2018

McINTOSH PERRY

TECHNICAL MEMORANDUM

To:	Benjamin Clare, MCIP, RPP Senior Land Use Planner
From:	Geotechnical Division
Date:	December 21, 2018
Re:	Preliminary Geotechnical Investigation Results (UPDATED), 6 Edgewater Street, Ottawa, Ontario

The intent of this memorandum is not to replace the Geotechnical Report, it is only intended to provide general expert opinion on site conditions and its suitability for development. At this point all boreholes have been drilled as intended. Conceptual site plans indicated potential development opportunities along the north and the east of the property. Therefore, boreholes were focused on those parts of the land. The quick turnaround requested by the client did not provide adequate time for laboratory testing and detailed analysis process. Judgements provided in this memorandum are based on our previous experience with soils of similar properties an design procedures indicated in relevant standards, codes and guidelines.

Field Procedure

An environmental drilling program was scheduled for December 13, 2018. Our geotechnical field staff attended the site during environmental drilling and performed geotechnical testing and limited sampling in two of the environmental boreholes which were drilled for monitoring well installations. A separate geotechnical drilling program was conducted on December 19 and more rigorous in-situ geotechnical testing and sampling, including one undisturbed sample, were completed during site investigation. Borehole location sketch and draft borehole logs are appended to this memorandum.

Subsurface Conditions

Soil descriptions provided in this section are solely based on visual observations and in-situ testing. At present there is no lab testing results available to confirm soil descriptions, however soil descriptions can be considered accurate for the purpose of this preliminary discussion.

Topsoil

A layer of approximately 150 mm of topsoil was encountered across all boreholes.

Fill

Topsoil was underlain by fill which consistently encountered in all boreholes. Fill thickness ranged between 0.3 m and 1 m with an average thickness of 0.7 m. SPT 'N' values within the fill ranged between 6 and 16 indicating a relatively non-homogeneous mix. Fill is clayey silt with some sand and gravel. Based on the visual observation the existing fill does not comply with OPSS Granular criteria therefore it cannot be used as engineered fill. The fill, as observed, is not suitable to provide structural support or drainage. However, fill can be bulk sampled during construction and tested for gradation and proctor to determine if it can be used or not for landscaping or general grade raise.

Clayey Silt

Fill was underlain by clayey silt trace to some sand in all boreholes except BH18-2. The clayey silt was considered 'firm' with SPT blow counts between 0 to 7 blows/300 mm. Presence of cobbles was inferred within this layer. This layer ranged between 1 to 3 m in thickness.

Silty Sand

A layer of silty sand trace to some clay encountered in borehole BH18-2 underneath the fill and in borehole BH18-3 underneath the clayey silt. SPT 'N' values ranged between 1 to 7 for this layer and it was categorized as 'loose' according to Canadian Foundation Engineering Manual (CFEM).

Sand

A layer of sand encountered in borehole BH18-1, 1.1 m in thickness and SPT 'N' value of 8 and was categorized as 'loose'.

Clay

A layer of very soft to soft clay encountered in boreholes BH18-2 (3.1 m thick), BH18-3 (3.1 m thick), and BH18-5 (5.7 m thick). Undrained shear tests were completed within this layer which indicated shear strength of between 25 kPa to 51 kPa which an average value of 35 kPa undrained shear strength. Remoulded strength was also measured and indicated sensitivity of between 5 to 16 which indicates the clay is extremely sensitive. Monitoring well MW18-3 which was drilled and installed by the environmental team also indicated existence of this soft clay.

Clay/silt/sand

Various compositions of clay and silt and sand on top of the bedrock/refusal was encountered in all boreholes except borehole BH18-5 with SPT 'N' values ranging between 0 and 9. Thicknesses are indicated in the borehole logs.

Bedrock

Boreholes BH18-1 and BH18-2 were terminated at auger refusal, borehole BH18-3 was terminated at DCPT cone refusal and boreholes BH18-4 and BH18-5 were cored to prove the rock. Bedrock was identified as limestone interbedded with shale and calcite. Coring of borehole BH18-4 indicated lower Rock Quality Designation (RQD) between 0 to 65 with highest value in run 3 out of 4. Bedrock was cored from El. 98.9 to 95.2 in borehole BH18-4. In borehole BH18-5 rock was cored from El. 94.6 down to 92.3 with measured RQD of 74 and 78 in two runs. It is reasonable to assume auger refusals and DCPT refusals also indicate an approximate rock surface. It is evident that the rock slopes from northwest corner (4.6 m) to northeast corner (13.2 m) and east (8 m).

Groundwater

Groundwater was measured in boreholes BH18-1 and BH18-2 (monitoring wells) at the two opposite corners of the site 1.1 m and 0.9 m below ground surface respectively (El. 101.2 and 101.5).

Discussion

This discussion is provided to offer an overview on the feasibility of any proposed development and the opportunities and challenges which the site may pose from foundation engineering perspective.

It is a common practice to lower the footings in Ottawa area down to below the expected frost penetration depth or approximately 1.8 m, unless frost protection is provided by synthetic insulation. However, considering required removal of all unsuitable topsoil and fill, founding at frost depth seems a plausible choice for this site.

With light-weight buildings in mind, only findings of boreholes BH18-1 and BH18-4 can support construction of light-weight buildings with no site improvement. Although the bearing capacities might be still low. Findings of borehole BH18-1 can support a relatively low bearing capacity of approximately 75 kPa SLS for shallow footings. borehole BH18-4 indicates silt under hydrostatic pressure therefore demonstrating low capacities in saturated condition. However, bedrock is not far in that borehole and the silt can be dug and replaced by engineered fill.

The very soft clay of high sensitivity which was encountered in all other boreholes (boreholes 2, 3, and 5) pose a challenge to any development. This clay is expected to undergo large deformations under superimposed loads of construction. The clay exists along the east side of the property. One solution for construction on such deposits are to use deep foundations in the design. Given the depth of bedrock at that location, piles are the most practical options. This option will add the cost of mobilization of pile installation rig and design and construction of pile caps to the design. However, the issue will still remain with grade raise, land scaping and pavement load, which depending on the load, may trigger consolidation settlements of their own.

The other option to overcome challenges involving construction on soft clays is to preload the site. To determine the required surcharge and the length of time required on site, the laboratory test on the obtained undisturbed sample has to be completed. Based on our previous experiences for a soil similar to the soft clay encountered at this site, an embankment of approximately 3 m to 4 m in height and projecting beyond the footprints of the buildings by 10 m is required to remain on site for approximately 1 year to 18 months. Consolidation process can be accelerated by adding more surcharge and installation of wick drains at a cost. All grade raise and pavement structures can be constructed once the building surcharges are placed and adjusted/re-graded upon completion of construction. Pre-loading process also needs an instrumentation and monitoring program to confirm projected settlements have been achieved. Instrumentation is involved installation vibrating wire piezometers and settlement plates.

Groundwater level is relatively high at this site. High groundwater table reduces the effective strength of the soil therefore the bearing capacity. If there is no basement proposed, there shouldn't be any concerns for building operations by having high groundwater table. However, that requires groundwater lowering and disposal during the course of construction.

Conclusion

In conclusion, the northwest corner of the site is the only location encountered feasible for construction with relatively low improvement cost. A low bearing capacity can be offered for that portion of the site or depending on the proposed building location the unsuitable soil can be removed and replaced with engineered fill (approximately 1 to 2 m in thickness).

The rest of the site is underlain by a layer of very soft and sensitive clay. That clay will undergo noticeably large deformations due to development loads. In case development is needed for those portions of the site, upon acquisition, the site has to be preloaded for approximately 12 to 18 months (to be confirmed by lab testing) before construction can begin. Otherwise deep foundations are needed to support the buildings.

Please do not hesitate to contact the undersigned should you have any questions or concerns.

McIntosh Perry Consulting Engineers

N’eem Tavakkoli, P.Eng.
Manager – Foundation Engineering

McINTOSH PERRY

Page 1 of 1

RECORD OF BOREHOLE No BH18-1

DATE:	13/12/2018 -	**LOCATION:**	6 Edgewater ()	**ORIGINATED BY:**	PH
PROJECT:	CP-17-0635-EDGEWATER	**COORDINATES:**	Lat: 45.295502521 , Lon: -75.895502089	**COMPILED BY:**	MG
CLIENT:	RK Porter	**DATUM:**	Geodetic	**CHECKED BY:**	NT
ELEVATION:	102.3 m	**REMARK:**		**REPORT DATE:**	20/12/2018

DYNAMIC CONE PEN. RESISTANCE PLOT 20 40 60 80

SHEAR STRENGTH (kPa) Vane test: ◇ Intact, ◆ Remolded; Lab vane: ☐ Intact, ■ Remolded; 20 40 60 80 100

WATER CONTENT and LIMITS (%) W_P W W_L; 25 50 75

REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C

SOIL PROFILE		SAMPLES			
ELEVATION - m DEPTH - m	DESCRIPTION	TYPE AND NUMBER	RECOVERY	"N" or RQD	GROUNDWATER CONDITIONS
102.3	Natural ground surface				
0.0	150 mm Topsoil.				
102.1 0.2	Fill. Clayey silt, some gravel, some sand, brown, moist, very stiff.	SS-01	42	16	
		SS-02	25	6	1.1 m
101.1 1.2	Clayey silt, some sand, brown, moist to wet, firm.	SS-03	83	5	
	- wet	SS-04	92	5	
	- presence of cobbles.	SS-05	58	65	
99.2 3.0	Sand, some gravel, traces of silt, brown, wet, loose.	SS-06	54	8	
		SS-07	58	16	
98.2 4.1	Silty and sandy clay, some gravel, grey, wet, compact to dense (Till).	SS-08	75	REF	
97.7 4.6	**END OF BOREHOLE** **Auger refusal on probable bedrock.**				

McINTOSH PERRY

RECORD OF BOREHOLE No BH18-2

DATE:	13/12/2018 -	**LOCATION:**	6 Edgewater ()	**ORIGINATED BY:**	PH
PROJECT:	CP-17-0635-EDGEWATER	**COORDINATES:**	Lat: 45.295803731 , Lon: -75.894296329	**COMPILED BY:**	MG
CLIENT:	RK Porter	**DATUM:**	Geodetic	**CHECKED BY:**	NT
ELEVATION:	102.4 m	**REMARK:**		**REPORT DATE:**	20/12/2018

SOIL PROFILE

ELEVATION - m DEPTH - m	DESCRIPTION
102.4	Natural ground surface
0.0	150 mm Topsoil.
102.2 0.2	Fill. Clayey silt, some gravel, some sand, brown, moist, stiff.
101.9 0.5	Fill. Clayey silt, trace sand, brown, moist, firm.
101.3 1.1	Silty sand, some clay, brown, moist to wet, loose. - wet - trace clay
100.0 2.4	Silty clay, grey, wet, very soft to soft.
96.9 5.5	Silty and sandy clay, some gravel, grey, wet, compact to dense (TILL).
96.3 6.1	**END OF BOREHOLE** **Auger refusal on probable bedrock.**

SAMPLES

TYPE AND NUMBER	STATE	RECOVERY	"N" or RQD
SS-01		17	8
SS-02		92	8
SS-03		100	4
SS-04		100	1
SS-05		75	0
SS-06		100	0
SS-07		76	REF

GROUNDWATER CONDITIONS

0.93 m

DEPTH - feet: 5, 10, 15, 20, 25, 30

DEPTH - meters: 1, 2, 3, 4, 5, 6, 7, 8, 9

SYMBOL

DYNAMIC CONE PEN. RESISTANCE PLOT

20 40 60 80

SHEAR STRENGTH (kPa)

Vane test: ◇ Intact, ◆ Remolded

Lab vane: ☐ Intact, ■ Remolded

20 40 60 80 100

WATER CONTENT and LIMITS (%)

W_P W W_L

25 50 75

REMARKS & GRAIN SIZE DISTRIBUTION (%)

G S M C

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McINTOSH PERRY

RECORD OF BOREHOLE No BH18-3

DATE:	19/12/2018 -	**LOCATION:**	6 Edgewater ()	**ORIGINATED BY:**	PH
PROJECT:	CP-17-0635-EDGEWATER	**COORDINATES:**	Lat: 45.296027536 , Lon: -75.894645507	**COMPILED BY:**	MG
CLIENT:	RK Porter	**DATUM:**	Geodetic	**CHECKED BY:**	NT
ELEVATION:	102.9 m	**REMARK:**		**REPORT DATE:**	20/12/2018

DEPTH - feet	DEPTH - meters	SOIL PROFILE ELEVATION - m DEPTH - m	SOIL PROFILE DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	SAMPLES STATE	SAMPLES RECOVERY	SAMPLES "N" or RQD	GROUNDWATER CONDITIONS	DYNAMIC CONE PEN. RESISTANCE PLOT 20 40 60 80 SHEAR STRENGTH (kPa) Vane test ◇ Intact ◆ Remolded; Lab vane □ Intact ■ Remolded 20 40 60 80 100	WATER CONTENT and LIMITS (%) W_P W W_L 25 50 75	REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C
		102.9	Natural ground surface									
		0.0	150 mm Topsoil.									
		102.8 0.2	Fill. Clayey silt, trace to some sand, trace gravel, brown, moist.									
		102.5 0.5	Clayey silt, some sand, grey to brown, moist to wet, firm. -Shell fragments						0.84 m			
	1				SS-01		79	5				
5		101.4 1.5	Silty sand, some to trace clay, brown, wet, loose.		SS-02		83	5				
	2				SS-03		71	7				
10	3	99.9 3.0	Silty clay, traces of sand, grey, wet, soft. -sand layers/seams		SS-04		100	0				
	4									◆2.0 ◇32.0		
										◆3.0 ◇25.0		
15	5				SS-05		100	0				
										◆4.0 ◇25.0		
										◆5.0 ◇32.0		
20	6	96.8 6.1	Clayey and sandy silt, some gravel, grey, wet, soft to stiff.		SS-06		83	0				
	7				SS-07		100	1				
25	8				SS-08		100	9				
					SS-09		17	11				
30	9	93.9 9.0	Probable Clayey and sandy silt.									

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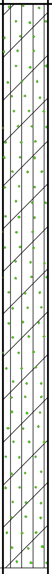
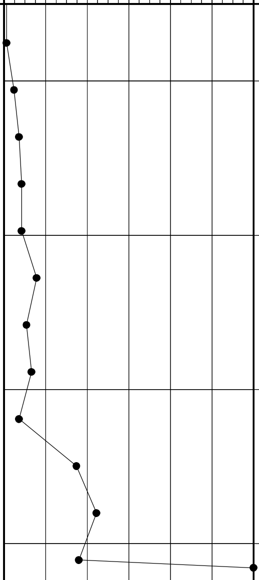
McINTOSH PERRY

RECORD OF BOREHOLE No BH18-3

DATE:	19/12/2018 -	**LOCATION:**	6 Edgewater ()	**ORIGINATED BY:**	PH
PROJECT:	CP-17-0635-EDGEWATER	**COORDINATES:**	Lat: 45.296027536 , Lon: -75.894645507	**COMPILED BY:**	MG
CLIENT:	RK Porter	**DATUM:**	Geodetic	**CHECKED BY:**	NT
ELEVATION:	102.9 m	**REMARK:**		**REPORT DATE:**	20/12/2018

DEPTH - feet	DEPTH - meters	SOIL PROFILE ELEVATION - m DEPTH - m	SOIL PROFILE DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	SAMPLES STATE	SAMPLES RECOVERY	SAMPLES "N" or RQD	GROUNDWATER CONDITIONS	DYNAMIC CONE PEN. RESISTANCE PLOT 20 40 60 80 SHEAR STRENGTH (kPa) Vane test: ◇ Intact ◆ Remolded; Lab vane: ☐ Intact ■ Remolded 20 40 60 80 100	WATER CONTENT and LIMITS (%) W_P W W_L 25 50 75	REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C
	10											
35	11											
40	12											
	13	89.8 13.2	**END OF BOREHOLE** **DCPT refusal on probable bedrock.**									
45	14											
50	15											
55	16											
	17											
60	18											





McINTOSH PERRY

RECORD OF BOREHOLE No BH18-4

Page 1 of 1

DATE: 19/12/2018 -
LOCATION: 6 Edgewater ()
ORIGINATED BY: PH
PROJECT: CP-17-0635-EDGEWATER
COORDINATES: Lat: 45.295829043 , Lon: -75.895014359
COMPILED BY: MG
CLIENT: RK Porter
DATUM: Geodetic
CHECKED BY: NT
ELEVATION: 102.7 m
REMARK: Water not measured due to core water in borehole.
REPORT DATE: 20/12/2018

DYNAMIC CONE PEN. RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH (kPa): Vane test ◇ Intact ◆ Remolded; Lab vane ☐ Intact ■ Remolded; 20 40 60 80 100

WATER CONTENT and LIMITS (%): W_P W W_L; 25 50 75

REMARKS & GRAIN SIZE DISTRIBUTION (%): G S M C

GROUNDWATER CONDITIONS

ELEVATION - m	DEPTH - m	DESCRIPTION	TYPE AND NUMBER	RECOVERY	"N" or RQD
102.7		Natural ground surface			
	0.0	150 mm Topsoil.			
102.5	0.2	Fill. Clayey silt, some sand, grey brown, moist, firm.			
101.9	0.8	Clayey silt, some sand, grey brown, moist, firm.	SS-01	25	7
			SS-02	17	2
100.4	2.3	Clayey silt, traces of sand, grey, wet, soft.	SS-03	100	0
			SS-04	83	0
98.9	3.8	Silty and sandy clay, some gravel, gret, wet, dense (Till).	SS-05	63	REF
98.7	4.0	Limestone Bedrock, interbedded with Shale, calcite veins.	RC-1	100	*0*
			RC-2	100	*24*
			RC-3	93	*65*
			RC-4	100	*20*
95.2	7.5	**END OF BOREHOLE**			

McINTOSH PERRY

RECORD OF BOREHOLE No BH18-5

DATE: 19/12/2018 -
PROJECT: CP-17-0635-EDGEWATER
CLIENT: RK Porter
ELEVATION: 102.6 m

LOCATION: 6 Edgewater ()
COORDINATES: Lat: 45.29590655 , Lon: -75.894479194
DATUM: Geodetic
REMARK: Water not measured due to core water in borehole.

ORIGINATED BY: PH
COMPILED BY: MG
CHECKED BY: NT
REPORT DATE: 20/12/2018

DYNAMIC CONE PEN. RESISTANCE PLOT 20 40 60 80
SHEAR STRENGTH (kPa) Vane test: ◇ Intact ◆ Remolded; Lab vane: ☐ Intact ■ Remolded; 20 40 60 80 100
WATER CONTENT and LIMITS (%) W_P W W_L; 25 50 75
REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C

DEPTH - feet: 5 10 15 20 25 30
DEPTH - meters: 1 2 3 4 5 6 7 8 9

SOIL PROFILE

ELEVATION - m DEPTH - m	DESCRIPTION
102.6	Natural ground surface
0.0	150 mm Topsoil.
102.5 0.2	Fill : Clayey silt, some sand, traces of gravel, brown, moist.
101.8 0.8	Clayey silt, some sand, grey brown, moist to wet, soft to firm.
100.3 2.3	Silty clay, some sand, grey, wet, soft.
94.6 8.0	Limestone Bedrock, interbedded with Shale, calcite veins.

SAMPLES

TYPE AND NUMBER	RECOVERY	"N" or RQD
SS-01	67	6
SS-02	4	2
SS-03	42	0
ST-04	100	
SS-05	100	0
SS-06	100	4
RC-1	100	*74*
RC-2	94	*78*

SHEAR STRENGTH (kPa)

Remolded	Intact
◆ 3.0	◇ 36.0
◆ 6.0	◇ 28.0
◆ 6.0	◇ 51.0
◆ 6.0	◇ 47.0

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McINTOSH PERRY

RECORD OF BOREHOLE No BH18-5

DATE: 19/12/2018 -
PROJECT: CP-17-0635-EDGEWATER
CLIENT: RK Porter
ELEVATION: 102.6 m

LOCATION: 6 Edgewater ()
COORDINATES: Lat: 45.29590655 , Lon: -75.894479194
DATUM: Geodetic
REMARK: Water not measured due to core water in borehole.

ORIGINATED BY: PH
COMPILED BY: MG
CHECKED BY: NT
REPORT DATE: 20/12/2018

DEPTH - feet	DEPTH - meters	SOIL PROFILE ELEVATION - m DEPTH - m	SOIL PROFILE DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	SAMPLES STATE	SAMPLES RECOVERY	SAMPLES "N" or RQD	GROUNDWATER CONDITIONS	DYNAMIC CONE PEN. RESISTANCE PLOT 20 40 60 80 SHEAR STRENGTH (kPa) Vane test: ◇ Intact ◆ Remolded Lab vane: ☐ Intact ■ Remolded 20 40 60 80 100	WATER CONTENT and LIMITS (%) W_P W W_L 25 50 75	REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C
	10											
		92.3 10.3	**END OF BOREHOLE**									
35	11											
40	12											
	13											
45	14											
	15											
50	16											
55	17											
	18											
60												

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