MULTI-STOREY RESIDENTIAL BUILDING
1806 SCOTT STREET, OTTAWA, ON.
FOUNDATION INVESTIGATION AND DESIGN REPORT

Project No.: CCO-23-1093

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1.0 INTRODUCTION

This report presents the factual findings of a geotechnical engineering investigation conducted for the proposed development site at 1806 Scott Street, Ottawa, ON. The proposed development involves the construction of a four-storey residential apartment building.

The report involves the methodology and findings of the geotechnical engineering investigation which consists of five (5) exploratory subsurface boreholes, laboratory testing procedures, and subsurface soil stratigraphy of the Proposed Site. The report will also include the anticipated geotechnical engineering conditions influencing the design and construction of the proposed development, and recommendations for the foundation design.

2.0 PROJECT UNDERSTANDING

It is understood that the proposed building would be a four-storey structure with a full basement, and there is no underground parking facility planned. The building plan includes a residential rental apartment.

3.0 SITE DESCRIPTION

The Proposed Site is located at the southwest corner of Scott Street and Rockhurst Road, in The City of Ottawa. Currently, the site contains a two-storey building, paved vehicle parking areas to the north and east, and a fenced-in backyard (lawn). The surrounding area of the building is flat with residential buildings to the south and west, and roads on the north and east of the property site. The Proposed Site location is shown in Figure 1. Appendix A.

3.1 Site Geology

A desk-top study using the published physiography maps of the area (Ontario Geological Survey or OGS) [1] indicates the Site is located on clay plains, and the surficial geology indicates a range from stone-poor, sandy silt to silty sand-textured till on Paleozoic terrain. The bedrock geology of the area consists of sandstone, shale, and dolostone of the Shadow Lake Formation from the Simcoe group.

Based on published physiography maps of the area (Ontario Geological Survey, OGS) [1], the site is located within the Till Plains which is drumlin that is caused by streamlined movement of glacial ice sheets across rock debris, or till. The Surficial geology maps of southern Ontario indicate the site is found on Till Plains comprising of stone-poor, sandy silty to silty sand-textured till on Paleozoic terrain [2].

The Paleozoic geology formation is Gull River of Simcoe Group, with identifying lithology of limestone, dolostone, shale, and sandstone bedding. The bedrock formation within this area is identified as Shadow Lake Formation, containing the Ottawa and Simcoe group with limestone, dolostone, shale, arkose, and sandstone [2].

4.0 FIELD PROCEDURES

The staff of McIntosh Perry Consulting Engineers (McIntosh Perry) conducted a site investigation prior to the planned drill date to mark the proposed borehole locations. Additionally, requisitions were submitted to Ontario One Call (ON1Call) for utility clearance and coordinated with The Client regarding the intended geotechnical exploration drill date.

The geotechnical investigation was continuously supervised and monitored by McIntosh Perry staff in accordance with Ontario Regulations (O. Reg.)[3], and applicable standards and procedures (American Society for Testing and Materials, ASTM) [4]. The drilling operation was performed by Marathon Environmental and Geotechnical drilling Ltd. from Ottawa. The boreholes were drilled using a geo-probe rubber track drill rig: A combination machine that performed both the Standard Penetration Testing (SPT) and rock coring operations. The drill was advanced using a 100 mm casing for wash boring/rock coring, and a 200 mm hollow stem helical auger during the drilling operation.

Boreholes 22-1, and 4 were drilled using the rock coring method to advance through overburden soil and to core the bedrock, whereas boreholes 22-2, and 5 were terminated at the casing refusal at the inferred surface of the bedrock, and the borehole 22-3 was drilled using the hollow stem helical auger and terminated at auger refusal at the inferred rock surface.

The auger/casing was incrementally advanced below ground surface (bgs), while overburden soil samples were intermittently taken at 0.75 m intervals. Each soil sample was retrieved with a 51 mm outside diameter (OD) Standard Penetration Test (SPT) sampler (SS) in accordance with ASTM D1586, SPT test procedures [4].

The soil samples retrieved from the SS sampler were examined, hermetically sealed in plastic bags, labeled, and packaged for transportation. The rock core samples were examined, measured, labeled, and packaged in protective rock core boxes for transportation to McIntosh Perry Geotechnical laboratory Ottawa (MP Geotech lab) in accordance with ASTM D 4220-95 Preserving and Transporting Soil Samples [4].

The five (5) boreholes BH22-1 to 5 were advanced into the subsurface to depths ranging from 0.7 to 6.5 m below ground surface, the bedrock was cored in boreholes BH22-1 and 4, from a depth of 1.3 to 6.4 m and 1.3 to 6.5 m below ground surface respectively. The borehole information summary is shown in Table 4-1.

Table 4-1. Borehole Information Summary

Borehole ID	Drilled Date	Coordinates (Geodetic)			Borehole		Termination Type
		Latitude	Longitude	Surface El. (m)	Depth (m)	Elevation El. (m)	
BH22-1	2022-08-11	45°23'58.51"N	75°44'37.50"W	62.7	6.4	56.4	Intended
BH22-2	2022-08-11	45°23'58.80"N	75°44'36.99"W	62.7	0.7	62.0	Refusal
BH22-3	2022-08-12	45°23'58.60"N	75°44'36.67"W	62.7	1.3	61.4	Refusal
BH22-4	2022-08-12	45°23'58.15"N	75°44'36.50"W	62.9	6.5	56.5	Intended
BH22-5	2022-08-11	45°23'58.09"N	75°44'36.73"W	62.7	1.4	61.3	Refusal

At the end of the drilling operations, all boreholes were backfilled with auger cuttings, Bentonite hole-plug, and asphalt cold patch as required and restored to their original surface condition. A monitoring standpipe piezometer was installed within borehole BH22-4. The Borehole locations on the proposed property are shown in Figure 2. Appendix A.

5.0 LABORATORY TEST PROCEDURES

All soil and rock core samples received at the MP Geotech lab were logged, and soil descriptions were verified by additional tactile examination in the laboratory. Representative soil and rock core samples from specific soil layers and depths corresponding to the foundation design requirements were identified and submitted to MP Geotech lab for detailed soil and rock core analysis.

Two (2) grain-size distribution sieve analysis and five (5) rock core Unconfined Compressive Strength (UCS) was carried out on representative soil and rock core samples at the MP Geotech lab.

All laboratory tests to determine the index properties were performed in accordance with ASTM test procedures. The relevant test procedures adopted are listed below.

- ASTM C136/LS-602 – Sieve Analysis of Fine and Coarse Aggregates
- ASTM D7012 – Unconfined Compressive Strength of Intact Rock Cores

Analytical and corrosivity testing was conducted on one (1) representative soil sample for the following analysis: pH level, electrical resistivity, chloride, and sulphate concentration levels.

All remaining samples are stored at MP Geotech lab for 90 days after the final report is submitted, thereafter the soil samples are disposed of according to MP Geotech lab policies. Unless The Client notifies MP Geotech lab in writing.

6.0 SUBSURFACE CONDITIONS

6.1 Subsurface

The site stratigraphy consisted of several layers, these layers were identified fill (including asphalt surface), native sand and grave, and bedrock. The notable subsurface layers encountered in the five (5) boreholes were subdivided into three (3) distinct strata and were identified according to the Unified Soil Classification System (USCS) [3] as;

1. Fill
2. Sand and Gravel
3. Bedrock

The borehole logs show a cross-section view of the subsurface soil stratigraphy of the location. The Borehole logs and bedrock cores are shown in Appendix C, and Appendix D.

6.1.1 Fill (Granular)

A cohesionless fill comprising of granular material was observed in boreholes BH22-1 and 3, which underlie the ≈ 75 mm thick paved asphalt layer. The fill layer consisted of sand and gravel with fractions of fine materials of silt and clay. The soil characteristics of the fill appeared brown, dry to moist, with SPT N-index values for this layer ranging from ≈ 17 – 29 blows/0.3 m, indicating an approximate compactness condition of compact to a dense layer of fill, according to table 3.1 of the CFEM [3].

One (1) representative sample from the fill layer was subjected to grain size distribution sieve analysis, the fill constituent percentage in weight contained ≈ 47% gravel, 34% Sand, and fractions of fine material of clay and silt. The fill layer's grain-size distribution summary is shown in Table 6-1.

Table 6-1. Fill Grain-Size Distribution Summary

Borehole	Sample	Constituent Materials in percent weight			
		Gravel (%)	Sand (%)	Fines	
				Silt (%)	Clay (%)
BH22-3	GS-1	47	34	19	

The grain-size distribution curve of the fill material was compared to a USCS granular type specifications envelope, and the distribution curves of the tested sample approximately conformed to The USCS Granular B Type I Specification (see Figure 3, Appendix D).

Some organic fill like topsoil and growth medium was encountered at the surface in boreholes BH22-2, BH22-4, and BH22-5.

6.1.2 Sand and Gravel

A cohesionless layer of expectedly native soil comprising sand and gravel material was observed in borehole BH22- 5. This layer consisted predominately of sand and gravel with fractions of fine materials of silt and clay. The soil characteristics of the fill appeared brown, dry to moist, with SPT N-index values for this layer ≈ 6 – 56 blows/0.3 m, indicating an approximate compactness condition of loose to a very dense layer of fill, according to table 3.1 of the CFEM [3].

One (1) representative sample from the sand and gravel layer was subjected to grain size distribution sieve analysis, in which constituent percentage in weight contained ≈ 57% gravel, 34% Sand, and fractions of fine materials of silt and clay. The fill layer's grain-size distribution summary is shown in Table 6-2, and the grain-size distribution curve for the fill material is shown in Figure 4, Appendix D.

Table 6-2. Sand and Gravel Grain-Size Distribution Summary

Borehole	Sample	Constituent Materials in percent weight			
		Gravel (%)	Sand (%)	Fines	
				Silt (%)	Clay (%)
BH22-5	SS-2	57	34	9	

6.1.3 Bedrock

Bedrock was cored in boreholes BH21-1 and 4 and inferred in the remainder at casing/auger refusal. The bedrock depth ranged from ≈ 0.7 – 1.4 m below the existing ground surface which corresponds to a range of elevations from El. 62.0 m to El 61.3 m.

Confirmation of bedrock was attained by coring boreholes BH22-1, and 4 from a depth of ≈ 1.3 to 6.4 m bgs., and 1.3 to 6.5 m bgs. respectively. The bedrock was identified as a sedimentary rock with horizontal beddings of sandstone, shale, and dolostone with planar joints.

The rock core (RC) samples recovered from bedrock were carefully recorded based on the length of each run and the samples encountered were evaluated for Total Core Recovery (TCR), and Rock Quality Designation (RQD). The rock core sample recovery quantity and quality results are shown in Table 6-3.

Table 6-3. Bedrock Core Recovery Summary

Borehole #	RC Sample #	Depth (m)	Theoretical Length of RC (m)	Length of RC Recovered (m)	Total Core Recovery (%)	RQD (%)
BH22-1	3	1.30	0.81	0.69	94	69
BH22-1	4	2.11	1.25	1.25	100	96
BH22-1	5	3.40	1.50	1.50	100	97
BH22-1	6	4.88	1.52	1.39	97	93
BH22-4	4	1.32	0.10	0.10	100	0
BH22-4	5	1.42	0.71	0.56	84	63
BH22-4	6	2.13	1.47	1.37	97	81
BH22-4	7	3.61	1.12	1.04	89	85
BH22-4	8	4.72	1.60	1.55	100	98

Five (5) samples of bedrock core were tested for UCS at the MP Geotech lab, and the resulting bedrock strength summary is shown in Table 6-4, the laboratory results and bedrock core images are shown in Appendix D.

Table 6-4. Bedrock Strength Summary

Borehole	Core No.	Run No.	Depth (m)	UCS (MPa)
BH22-1	1	1	1.3 - 2.1	257
BH22-1	2	2	2.1 - 3.4	181
BH22-4	3	2	1.3 - 2.1	187
BH22-4	4	3	2.1 - 3.6	203
BH22-4	5	4	3.6 - 4.7	237

6.2 Chemical Analysis

One (1) representative soil sample BH22-5 / SS-2 was sent for soil chemical analysis testing for the following; pH level, resistivity level, chloride, and sulphate concentration. The corresponding test results indicate the following levels and concentrations shown in Table 6-5. The laboratory test result “Certificate of Analysis” is shown in Appendix C.

Table 6-5. Chemical Analysis Summary

BOREHOLE	SAMPLE	DEPTH (m)	Chemical Analysis			
			pH (pH units)	Resistivity (Ohm.cm)	Chloride (ppm)	sulphate (ppm)
BH22-5	SS-2	0.6 - 1.2	7.79	7170	< 5	32

6.3 Groundwater

Groundwater (GW) was not encountered in the open boreholes; however, a single standpipe piezometer was installed in borehole BH22-4 within the layer of the bedrock. The last groundwater measurement was done on October 24, 2022, and no groundwater was observed in the standpipe piezometer. Groundwater level is expected to fluctuate seasonally and may be encountered in the future.

7.0 DISCUSSIONS AND RECOMMENDATIONS

7.1 General

This section of the report provides engineering recommendations on the geotechnical design aspect of the project based on the project requirements and our interpretation of the subsurface soil information. The recommendations presented herein are subject to the limitations noted in Appendix A "Limitations of Report" which forms an integral part of this document.

The foundation engineering recommendations presented in this section have been developed following Part 4 of the 2015 National Building Code of Canada (NBCC) and 2012 Ontario Building Code (OBC) extending the Limit State Design approach.

7.2 Overview

It is understood that the proposed apartment building is a four-storey mid-rise structure with one underground basement level. The finished floor elevation is El. 62.90 based on the site plan issued November 23, 2022.

For the current project, the following list summarizes some key geotechnical details that were considered in the suggested geotechnical recommendations:

- Shallow bedrock is either sampled or inferred across the site. It is concluded provision of shallow spread and strip footing is adequate for the proposed mid-rise structure.

- The proposed structure can be designed using a seismic Site Class C provided that the boundary zones of the shear walls and all column loads are extended to and supported on the bedrock, confirmed by geotechnical staff upon completion of excavation.
- The contractor shall submit the excavation plan for geotechnical review. The plan shall be prepared based on the final site layout, depth of excavation, and offset from adjacent buildings to ensure the protection of those building are considered.
- Based on the observed RQD, if rock excavation is needed hoe ramming and line drilling shall be adequate for leveling the rock surface. Rock blasting is not envisioned based on the proximity of the existing structure. If blasting is required, a blasting plan including health and safety and monitoring programs shall be submitted by the contractor.
- No major issues are expected with groundwater management during construction even if excavations are advanced below the rock surface. One standpipe piezometer was installed in borehole BH22-4 and it was read two months after the initial installation and no water was encountered in the monitoring well. The chance of water seepage into the excavation is low. Based on the current information on design requirements, an application for Permits to Take Water (PTTW) is not required.

7.3 Site Preparation

The expected subgrade is bedrock. All fill, topsoil, and sandy silt overburden shall be removed from the footing subgrade. All loose rock pieces shall be removed, and the subgrade shall be approved by a geotechnical staff. The contractor shall use the information on the rock RQD and unconfined compressive strength to design the proper rock excavation methodology.

Upon completion of the excavation, depending on the subgrade condition, subgrade grouting or poured mud slab may be required. The mud slab shall provide a minimum of 15 MPa compressive strength at 28-day age testing.

The foundation design recommendations provided in this report are based on the assumption of flat subgrade. This report does not support the construction of step footings unless confirmed by the geotechnical engineer upon site review. This condition is put in place to ensure proper subgrade preparation for individual strip or spread footings. This disclaimer does not apply to the construction of elevator pit lower than the other footings.

7.4 Foundations

Bearing resistance is calculated for the bedrock surface.

Provided there are no continuous soil-filled seams or mud seams present at shallow depth in the bedrock below the founding level, footings can be supported on the bedrock surface, or a platform of lean concrete of compressive strength of greater than 15 MPa extending down to the bedrock surface.

The Ultimate Limit States (ULS) factored bearing resistance was estimated using the Rock Mass Rating (RMR) method by Bieniawski (1989). The RMR method was utilized to determine the required parameters for bearing capacity resistance at ULS conditions for the bedrock.

Based on the bedrock cores quality and uniaxial compressive strength tests, the following ratings are estimated:

- The lower bond compressive strength of intact rock rating: The uniaxial compressive strength was taken as 180 MPa, which results in rating = 12,
- RQD rating: The RQD of the rock core is 63 at the surface (falls at the lower boundary value), which results in rating = 13,
- Joint spacing rating: The joint spacing for the rock core samples is occasionally less than 50 mm, which gives an estimated rating = 5,
- Joint condition: The joint condition was observed to be slightly rough, and the rating is estimated to be = 12,
- Groundwater rating: the groundwater elevation was not observed in the monitoring well. Therefore, the estimated rating for water condition = 4; and
- Orientation rating: Horizontal to 25° joints; therefore, a fair to favorable rating was estimated = -2.

The RMR for the rock approximately equals (44) which can be classified to have fair rock quality.

Assuming the above-noted conditions are provided, the estimated factored ULS bearing resistance is 1350 kPa for a minimum of 2 m depth below the existing ground surface which equals to the rock at approximately El. 60.7 m or below the weathered rock surface, whichever is lower. It is understood the elevator pit will be dug over 1.5 m deeper than the rest of the footings.

The provided factored bearing resistance at ULS is based on the uniaxial compressive strength of the rock. The size of the selected footing shall be determined by a structural engineer. The selected size of the footing shall have adequate compressive strength to provide resistance to the structural loads from the building and to avoid failure in concrete material under the applied pressure. Shallow footings shall not be smaller than 0.6 m in their smaller dimension.

Provided the bedrock surface is properly cleaned of soil and weathered material at the time of construction, the settlement of footing size using the above factored bearing resistance should be negligible. The bearing capacities are calculated for a flat subgrade.

Table 7-1: Rock Bearing Resistance

Footing Type	ULS (kPa)
Spread Footings	1,350

The ultimate bearing capacity will govern the design. The serviceability limit state as defined by allowable settlements is not applicable for this project on rock subgrade.

Highly weathered or fractured bedrock, which includes bedrock that can be excavated using hydraulic excavating equipment with only moderate effort, would need to be removed and replaced with concrete.

The rock bearing surface should be inspected by qualified geotechnical personnel of McIntosh Perry to confirm that the surface has been acceptably cleaned of soil, and that weathered, or excessively fractured bedrock has been removed.

7.4.1 Rock Anchors

It is expected that the foundations may be required to resist uplift forces related to unbalanced lateral loads. The uplift forces may be resisted using grouted anchors in the bedrock. The presence of fractured rock conditions and groundwater should be considered carefully by the specialty contractor and may require post-grouting to ensure adequate anchor resistance is obtained.

In designing grouted rock anchors, consideration should be given to four potential anchor failure modes:

1. Failure of the steel tendon or top anchorage;
2. Failure of the grout/tendon bond;
3. Failure of the rock/grout bond; and,
4. Failure within the rock mass, or rock cone bull-out.

Potential failure modes “1 and 2” are structural and are required to be addressed by the structural engineer.

For potential failure mode “3”, a static proof test in tension during construction, as per OBC 2012, will be required to assist the unfactored ULS bond strength at the concrete-rock interface. A resistance factor of 0.4

may be used to estimate the factored ULS. As a general guide, the ULS for limestone ranges between 1.0 to 1.4 MPa as per Post-Tensioning Institution (PTI) Recommendations for Prestressed Rock and Soil Anchors, 1996.

For potential failure mode “4”, the resistance should be calculated based on the buoyant weight of the potential mass of rock that could be mobilized by the anchor. This is typically considered as the mass of rock and surface shear resistance within a cone or wedge for a line of closely spaced anchors having an apex at the tip of the anchor that forms an angle between 60o to 90o. For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors.

As stated earlier, proof tests should be performed to confirm the pull-out capacity. The proof tests should be carried out to 1.5 times the anchor service loads, and at least 10 percent of the anchors should be tested. The testing procedure should be in accordance with either OPSS 942 or the PTI (1996) for proof testing.

The installation and testing of the anchors should be observed by a geotechnical engineer. 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 to remove debris, scum/sludge, and rock flour prior to grouting.

7.4.2 Frost Protection

Based on the subsurface investigation results, the encountered bedrock subgrade is of low frost susceptibility. Frost penetration depth is 1.5 m below the surface for the subject site. Frost penetration depth is estimated based on the OPSD 3090.101, Foundation Frost Penetration Depths for Southern Ontario.

All perimeter and exterior foundation elements, or interior foundation elements in unheated areas should be provided with a minimum of 1.8 m of earth cover or equivalent thermal rigid insulation for frost protection purposes.

7.5 Seismic Site Classification

Seismic hazard calculations are provided in Appendix E for a combination of probabilities and spectral responses. The provided values are for reference only and the designers shall verify these values for their design.

Seismic site classification is completed based on NBCC (2015) and 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 of soil and rock, standard penetration resistance, and plasticity parameters of cohesive soils. Based on the investigation results the site can be classified as Seismic Site Class (C) for this bedrock subgrade.

7.6 Engineered Fill

Footings shall be installed on the bedrock. Any over-excavation shall be leveled by lean concrete of a minimum 15 MPa at 28 days strength or matching the strength of the structural footings.

The proposed engineered fill, beyond the footings' influence zone, can be any material conforming to granular criteria as outlined in OPSS 1010. Material conforming to 'Granular' criteria is considered free draining and compactable and can be utilized as the engineered fill. This can apply to the backfill beyond foundation walls and engineered fill in between the footings. The engineered fill shall be compacted to a minimum of 98% SPMDD.

All fill material 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 requirement for fill material and compaction may be addressed with a note on the structural drawing for foundation or grading drawing, and with a Non-Standard Special Provision (NSSP). Any topsoil, organics, or loose sand should be removed before placing engineered fill material.

The existing fill and/or native soil does not qualify as backfill or grading fill material.

As long as all structural elements are beading on bedrock subgrade, there is no restriction on grade raise.

7.7 Slabs-on-Grade

Slab-on-grades are considered free-floating (not attached to the foundation walls) and should be supported on a minimum of 200 mm of Granular A bedding compacted to 100% Standard Proctor Maximum Dry Density (SPMDD).

The rest of the fill, above the rock and below the slab can be filled with 'Granular' material as per the OPSS 1010 and compacted to a minimum of 98% SPMDD. If the slab on grade is to carry structural loads, the grading fill shall be Granular B Type II and compacted to a minimum of 100% SPMDD.

Subgrade preparation and compaction efforts shall be approved under the supervision of a geotechnical representative from McIntosh Perry.

If for the design of any portions of the slab-on-grade, the modulus of subgrade reaction (k) is required, the following recommendation can be used for structural modeling. The modulus of the subgrade reaction is a multi-function complex correlation that varies with the subgrade material, grade-raise fill material, and the flexural stiffness of the structural slab. However, simplified assumptions were made to estimate the spring modulus for slab-on-grade on compacted Granular A. To estimate the modulus of subgrade reaction, through a simplistic approach, a 2 m square section of the concrete slab-on-grade under the applied loads. Since the modulus of subgrade reaction is needed for the ultimate failure design of the slab, it is assumed the failure can occur at a 25 mm deformation. Considering these assumptions, a subgrade reaction modulus of 10,000 kN/m2/m can be used for the design of the interior slab-on-grade. This k-value is only valid for the construction of slab-on-grade on compacted Granular A bedding. This value shall not be used for the native subgrade.

7.8 Lateral Earth Pressure

Free-draining material should be used as backfill material for foundation walls. If proper drainage is provided, "at rest" condition may be assumed for the calculation of earth pressure on foundation walls. The following parameters are recommended for the granular backfill.

Table 7-2. Lateral Pressure parameters for Granular A and B and Horizontal Backfill

Pressure Parameter		Expected Value		
		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

The native sandy silt is not suitable for backfilling foundation walls.

7.9 Flexible Pavement

For most of the site, the pavement structure is most likely to be placed on engineered fill material overlaying the bedrock or the existing fill or the native sand and gravel. All fill and organic material shall be removed from the proposed pavement site and replaced with engineered fill. The existing non-organic material can act as the pavement subgrade if verified by visual confirmation and proof rolling.

The pavement structure proposed in this design considers the very low traffic volume of lightweight passenger vehicles. It is understood moving trucks and firetrucks traffic will be limited to the public road. The light-duty pavement structure design specifications are given in Table 7-3.

Table 7-3: Pavement Structure for the Parking Lot

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

It is understood there is a provision of permeable pavement options. Permeable asphalt or concrete surface course is not recommended due to complications associated with maintenance of such pavement. These pavement options are prone to salt, de-icer, freeze-thaw cycles and requires trained staff for maintenance. The only plausible permeable pavement option recommended in this report are the stone pavers (concrete paver blocks).

To facilitate rapid drainage, a permeable pavement structure is proposed in Table 6-4. Subdrain pipe shall be placed on subdrain trench imbedded within or below the subbase with positive drainage to the storm catch basins. All subdrains shall receive a non-woven sock. Pavers are expected to received periodic maintenance for adjustment and leveling depending on the applied traffic load.

Table 7-4: Permeable Pavement Structure Alternative

Material		Thickness (mm)
Surface	Concrete Paver	min. 80
Bedding	Loose Sand	25
Drainage	OPSS Granular O	100
Base	OPSS Granular A	150
Sub-base	OPSS Granular B Type II	450

The base and sub-base materials, i.e., Granular A for the base and Granular B Type II for the 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.

7.10 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. Expansion, construction, and dummy joints shall be spaced as required by the applicable standards. Sidewalks can be categorized under residential/commercial use, and therefore, the concrete sidewalks should have a thickness of 125 mm. Requirements of OPSD 310.010 ‘Concrete Sidewalk’, OPSD 310.020 ‘Concrete Sidewalks Adjacent to Curb and Gutter’, and OPSD 310.030 ‘Concrete Sidewalk Ramps at the intersection’ are recommended for the construction of the concrete sidewalk. A minimum of 150 mm bedding of OPSS Granular A compacted to 100% SPMDD is required for the concrete sidewalk panels.

7.11 Cement Type and Corrosion Potential

One soil sample was 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 6-5.

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 in the subsurface conditions encountered.

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

8.0 CONSTRUCTION CONSIDERATIONS

Any organic material and loose sand of any kind should be removed from the footprint of the footings and all structurally load-bearing elements. Site preparation and requirements of engineered fill placement are noted in previous sections. Refer to relevant sections for material and compaction requirements.

The Occupational Health and Safety Act (OHSA) of Ontario indicated that side slopes in the sandy silt could be classified as Type 4 soil and sloped no steeper than 3H:1V or be shored for any excavation deeper than 1.2 m. If space restrictions exist, the excavations can be carried out within temporary retaining systems, which is fully braced to resist lateral earth pressure.

As noted in the previous sections, all grade adjustments due to over-excavation, within the shallow footing influence zone, shall be done using lean concrete of minimum 15 MPa mature strength.

Foundation walls should be backfilled with free-draining material with granular material conforming to OPSS 1010 Granular criteria. The native soil is not suitable for backfill due to its high fine content.

A geotechnical engineer or technician should attend the site to confirm the native subgrade, type of fill material, and level of compaction. All bearing surfaces should be inspected by experienced geotechnical personnel prior to placing the footings to ensure the excavated subgrade is in the reported and recommended condition.

Rock excavation through blasting is not recommended. At the contractor or owner’s discretion, vibration monitoring may be carried out during the excavation and construction phases to ensure that the vibration levels at the existing surrounding structures and utilities are maintained below tolerable levels.

Installation of weeping tiles is necessary below the lowest habitable elevation.

9.0 GROUNDWATER SEEPAGE

The groundwater is expected below the rock excavation depth. However, the weathered rock at higher elevations may collect and outlet the surface runoff after major precipitation events, snow melt, or generally wet seasons with higher groundwater tables. The monitoring well is kept on-site for future reference.

Under the new regulations (O.Reg 63/16 and O.Reg 387/04), a Permit To Take Water (PTTW) is required from the Ministry of the Environment, Conservation, and Parks (MOECP) if a volume of water greater than 400,000 liters per day is pumped from the excavation under normal operation, but less than 50,000 liters per day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity. Since the excavations will likely be above the groundwater level, it is considered unlikely that a PTTW would be required. The site designer shall decide on the permit application based on the expected excavation volume.

The design of the dewatering system should be the responsibility of the contractor. An outlet(s) should be identified, which the contractor can use to dispose of the pumped groundwater and incident precipitation. In order for pumped groundwater to be discharged to a city sewer, the groundwater quality needs to meet the Sewer Use By-law limits, and a separate approval is required.

10.0 SITE SERVICES

At the subject site, the burial depth of water-bearing utility lines is typically 2.2 m below the ground surface. If this depth is not achievable, equivalent thermal insulation should be provided. The contractor should retain a professional engineer to provide detailed drawings for excavation and temporary support of the excavation walls during construction.

Excavation will proceed through the topsoil and native shallow deposits. Excavating of overburden soil shall be performed using conventional hydraulic excavating equipment. Cobbles or boulders larger than 300 mm in diameter, if encountered, should be removed from the side slopes for worker safety.

The Occupational Health and Safety Act (OHSA) of Ontario indicated that side slopes in the sandy silt could be classified as Type 4 soil and sloped no steeper than 3H:1V or be shored. If space restrictions exist, the excavations can be carried out within trench boxes for utility installation, which is fully braced to resist lateral earth pressure.

Due to the potential for long-term settlement of topsoil and organic materials and the effects of this settlement on service lines sensitive to level change, the existing topsoil, and organic materials are not considered suitable for the support of site services. 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. The engineer designing utilities shall ensure the proposed utility pipes can tolerate compaction loads.

11.0 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.

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Engineering Intern
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12.0 REFERENCES

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1806 SCOTT STREET, OTTAWA, ONTARIO.
GEOTECHNICAL AND FOUNDATION REPORT

APPENDIX A
LIMITATIONS OF REPORT

MCINTOSH PERRY

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.

1806 SCOTT STREET, OTTAWA, ONTARIO.
GEOTECHNICAL AND FOUNDATION REPORT

APPENDIX B
SITE LOCATION

McINTOSH PERRY

500 250 0 500

Scale 1:15,000 Metres

LEGEND

- Site Location
- Local Road
- Major Road
- Waterbody
- Wooded Area

REFERENCE

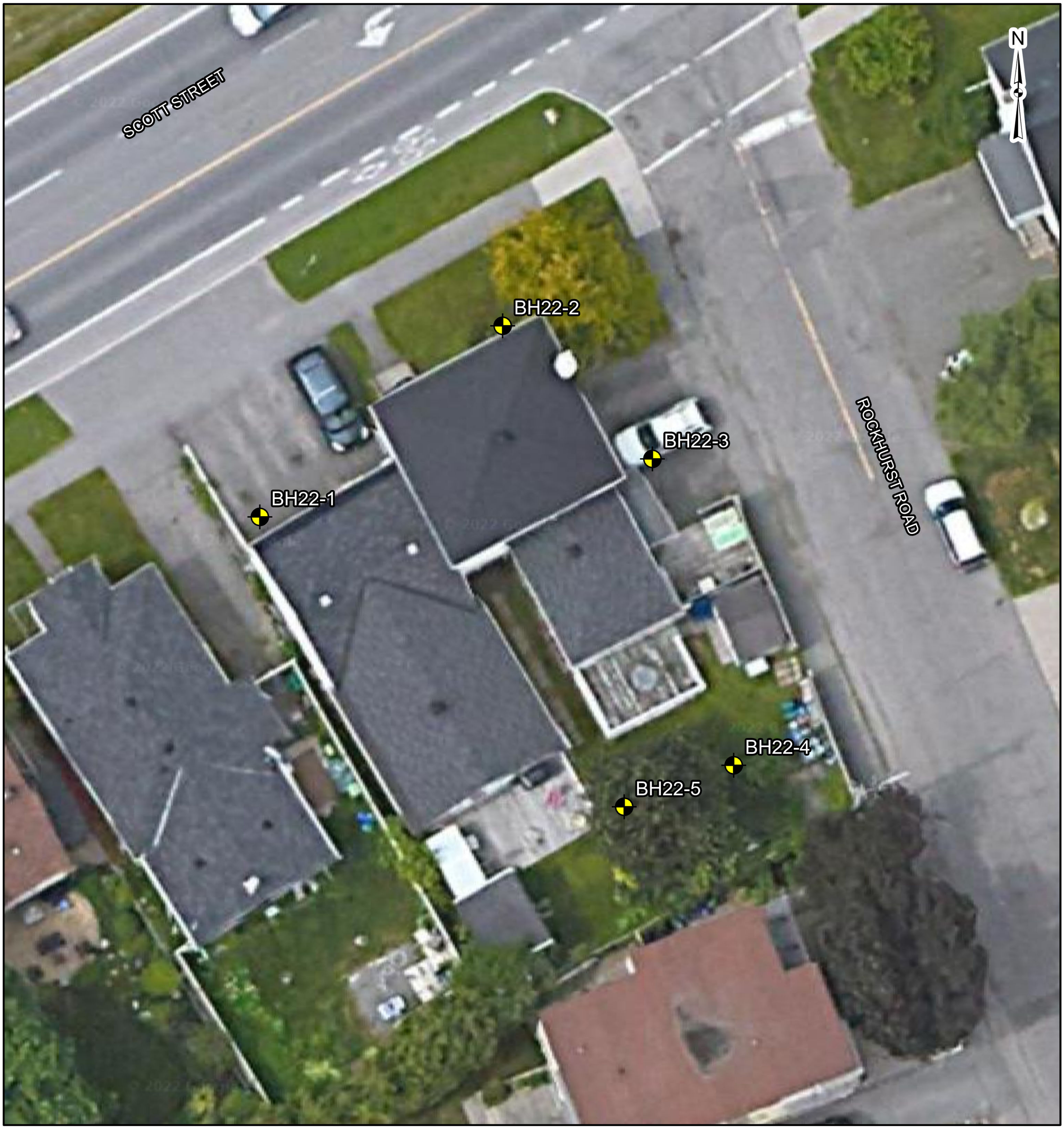
GIS data provided by the Ontario Ministry of Northern Development, Mines, Natural Resources and Forestry, 2022.

CLIENT:	GABRIELA GODINEZ-LAVERTY
PROJECT:	GEOTECHNICAL INVESTIGATION 1806 SCOTT STREET
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-23-1093-02	FIGURE:
Date	Sep., 27, 2022	1
GIS	AH	
Checked By	MH	

C:\Users\a.hamed\McIntosh Perry\GIS - Documents\Projects\2023\CCO\CCO-23-1093 1806 Scott St\aprx\Geotech\CCO-23-1093-02_Geotech.aprx



5 2.5 0 5

Scale 1:250

Metres

LEGEND

Borehole Locations

REFERENCE

GIS data provided by the Ontario Ministry of Northern Development, Mines, Natural Resources and Forestry, 2022.

CLIENT: GABRIELA GODINEZ-LAVERTY

PROJECT: GEOTECHNICAL INVESTIGATION
1806 SCOTT STREET

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-23-1093-02
Date	Sep., 27, 2022
GIS	AH
Checked By	MH

FIGURE: 2

1806 SCOTT STREET, OTTAWA, ONTARIO.
GEOTECHNICAL AND FOUNDATION REPORT

APPENDIX C

BORENOLE 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
$\sigma_1, \sigma_2, \sigma_3$	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'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	-°	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

ρ_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 = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_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
ρ	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
ρ_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
ρ_{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
ρ'	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						

PROJECT NO.: **CCO-23-1093**
PROJECT: Four Storey Building
CLIENT: Babriela Godinez-Laverty
PROJECT LOCATION: 1806 Scott Street, Ottawa, ON.

DRILLING DATA
Date: Aug-11-2022
Method: Wash Bore - NQ
Diameter: 100 mm
BH Location: N 5029128 E 364006

BH No: **22-1**
DATUM: MTM Zone 9
ENCL NO.: 1

ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	DEPTH	ELEVATION	Remarks and Grain Size Distribution (%)
62.7 0.0	Paved Driveway						0.0		
62.6 0.1	**ASPHALT: 75mm**								
	FILL: Silty Sand and Gravel, brown, dry, compact		1	SS	17			62	
			2	SS	46		1.0		Spoon bouncing @ 1.3 m bgs.
61.4 1.3	**BEDROCK** Sedimentary, horizontal thin laminated bedding, with calcite, mudstone, siltstone, horizontal bedding with moderate joints		3	RC	RQD = 69%		2.0	61	Run 1: TCR = 94%, UCS = 257 MPa
			4	RC	RQD = 96%		3.0	60	Run 2: TCR = 100%, UCS = 181 MPa
			5	RC	RQD = 97%		4.0	59	Run 3: TCR = 100%
			6	RC	RQD = 93%		5.0	58	Run 4: TCR = 97%
56.4 6.4	**End of Borehole** - Borehole terminated at intended depth @ 6.4 m bgs.						6.0	57	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80
SHEAR STRENGTH (kPa): Field. Shear Vane (x) & Sensitivity; ● Quick Triaxial; O Unconfined: 20 40 60 80
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30 40 50 60 70 80 90
Unit Weight (kN/m³) Pocket Penetro. (kPa)
GR SA SI CL

GRAPH NOTES
30/3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity
O ε=3% Strain at Failure

PROJECT NO.: **CCO-23-1093**
PROJECT: Four Storey Building
CLIENT: Babriela Godinez-Laverty
PROJECT LOCATION: 1806 Scott Street, Ottawa, ON.

DRILLING DATA
Date: Aug-11-2022
Method: Wash Bore
Diameter: 100 mm
BH Location: N 5029137 E 364017

BH No: **22-2**
DATUM: MTM Zone 9
ENCL NO.: 2

ELEV / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	DEPTH	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80) / SHEAR STRENGTH (kPa) (Field. Shear Vane (x) & Sensitivity; ● Quick Triaxial; O Unconfined) (20 40 60 80)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30 40 50 60 70 80 90	Remarks and Grain Size Distribution (%) Unit Weight (kN/m³) Pocket Penetro. (kPa) GR SA SI CL
62.7 / 0.0	Grass Cover						0.0				
62.5 / 0.0	**TOPSOIL: Silty Sand with Organic**, dark brown, moist, loose										
0.2	**FILL: Silty sand with gravel**, moist, loose		1	SS	18						
62.0 / 0.7			2	SS	50/ 100mm			62			Spoon bouncing @ 0.7 m bgs
	End of Borehole										
	- Refusal on inferred bedrock										

GRAPH NOTES
30 / 3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity
O ε=3% Strain at Failure

1MP SOIL LOG 1806 SCOTT ST.GPJ MP_OTTAWA_FOUNDATIONS.GDT 22-10-21

PROJECT NO.: **CCO-23-1093**	DRILLING DATA
PROJECT: Four Storey Building	Date: Aug-12-2022
CLIENT: Babriela Godinez-Laverty	Method: Hollow Stem Augers
PROJECT LOCATION: 1806 Scott Street, Ottawa, ON.	Diameter: 200 mm
	BH Location: N 5029131 E 364024

BH No: **22-3**

DATUM: MTM Zone 9

ENCL NO.: 3

ELEV / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	DEPTH	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80) / SHEAR STRENGTH (kPa) (Field. Shear Vane (x) & Sensitivity; ● Quick Triaxial; O Unconfined) 20 40 60 80	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30 40 50 60 70 80 90	Remarks and Grain Size Distribution (%) Unit Weight (kN/m³) Pocket Penetro. (kPa) GR SA SI CL
62.7 / 0.0	Paved Driveway						0.0				
62.6 / 0.1	**ASPHALT: 75 mm**										
	FILL: Sandy Gravel, some silt and clay, brown, dry to moist, compact to dense		1	SS	29						47 34 (19)
								62			
			2	SS	50/ 100mm		1.0				
61.4 / 1.3	**End of Borehole**										Auger grinding @ 1.3 m bgs.
	- Auger Refusal on inferred bedrock										

GRAPH NOTES: $\overset{30}{\underset{3}{\times}}$ Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity. O 8 =3% Strain at Failure

PROJECT NO.: **CCO-23-1093**
PROJECT: Four Storey Building
CLIENT: Babriela Godinez-Laverty
PROJECT LOCATION: 1806 Scott Street, Ottawa, ON.

DRILLING DATA
Date: Aug-12-2022
Method: Wash Bore - NQ
Diameter: 100 mm
BH Location: N 5029117 E 364028

BH No: **22-4**
DATUM: MTM Zone 9
ENCL NO.: 4

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS / 0.3 m	ELEVATION	Remarks and Grain Size Distribution (%)
62.9 / 0.0	Grass Cover					
0.0	**TOPSOIL: Silty Sand with Organic**, dark brown, moist, loose	1	SS	6		
62.3 / 0.7	**FILL: Silty Sand and Gravel**, grey, moist, very dense	2	SS	63	62	
61.6 / 1.3	**BEDROCK**	3	SS	10/ 0mm		
	Sedimentary, horizontal thin laminated bedding, with calcite, mudstone, siltstone, horizontal bedding with moderate joints	4	RC	RQD = 0%		Run 1: TCR = 100%
		5	RC	RQD = 63%	61	Run 2: TCR = 84%, UCS = 187 MPa
		6	RC	RQD = 81%	60	Run 3: TCR = 97%, UCS = 203 MPa
		7	RC	RQD = 85%	59	Run 4: TCR = 89%, UCS = 237 MPa
		8	RC	RQD = 98%	58, 57	Run 5: TCR = 100%
56.5 / 6.5	**End of Borehole**					

End of Borehole

- Standpipe installed
- Borehole terminated at intended depth @ 6.5 m bgs.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80
SHEAR STRENGTH (kPa): Field. Shear Vane (x) & Sensitivity; ● Quick Triaxial; ○ Unconfined; 20 40 60 80
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 10 20 30 40 50 60 70 80 90
Unit Weight (kN/m³), Pocket Penetro. (kPa); GR SA SI CL

GRAPH NOTES: 30 / 3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity; ○ ε=3% Strain at Failure

PROJECT NO.: **CCO-23-1093**
PROJECT: Four Storey Building
CLIENT: Babriela Godinez-Laverty
PROJECT LOCATION: 1806 Scott Street, Ottawa, ON.

DRILLING DATA
Date: Aug-11-2022
Method: Wash Bore
Diameter: 100 mm
BH Location: N 5029115 E 364023

BH No: **22-5**
DATUM: MTM Zone 9
ENCL NO.: 5

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80) / SHEAR STRENGTH (kPa) ● Quick Triaxial O Unconfined, Field. Shear Vane (x) & Sensitivity (20 40 60 80)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30 40 50 60 70 80 90	Remarks and Grain Size Distribution (%) Unit Weight (kN/m³) Pocket Penetro. (kPa) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m						
62.7	Grass Cover						0.0				
62.5 / 0.0	**TOPSOIL: Silty Sand with Organic**, dark brown, moist, loose		1	SS	6						
62.3 / 0.2	**Silty Sand**, some gravel, dark brown, moist, loose										
0.4	**Sandy Gravel**, trace silty and clay, brown, moist, compact		2	SS	56		1.0	62			57 34 (9)
61.3 / 1.4			3	SS	10/ 0mm						Splitspoon bouncing @ 1.4 m bgs.
	End of Borehole										
	Auger refusal on inferred bedrock										

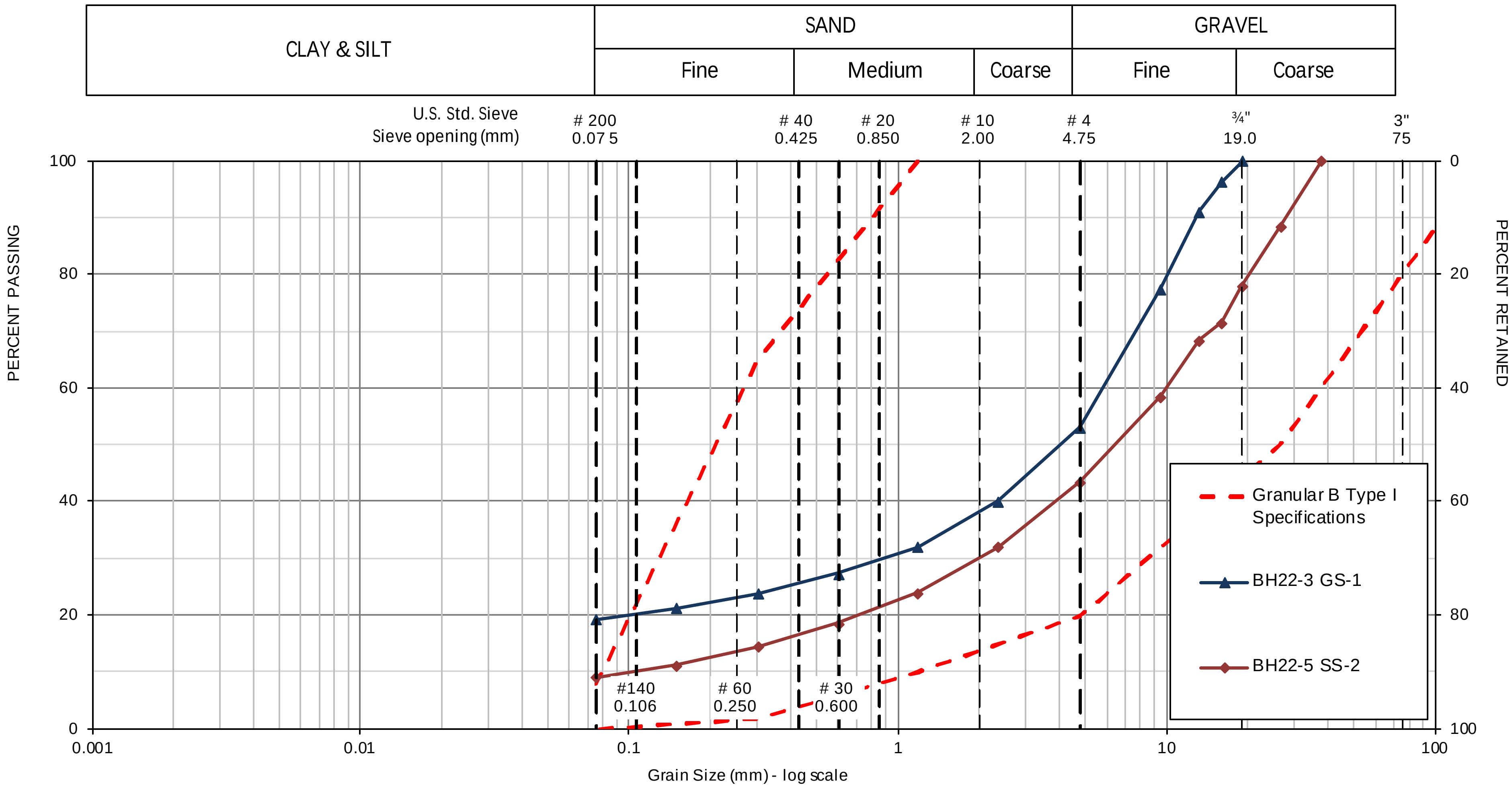
GRAPH NOTES
30 / 3: Upper value = Field Vane Shear Strength, Lower value = Vane Sensitivity
O 8 =3% Strain at Failure

1806 SCOTT STREET, OTTAWA, ONTARIO.
GEOTECHNICAL AND FOUNDATION REPORT

APPENDIX D
LABORATORY TEST RESULTS

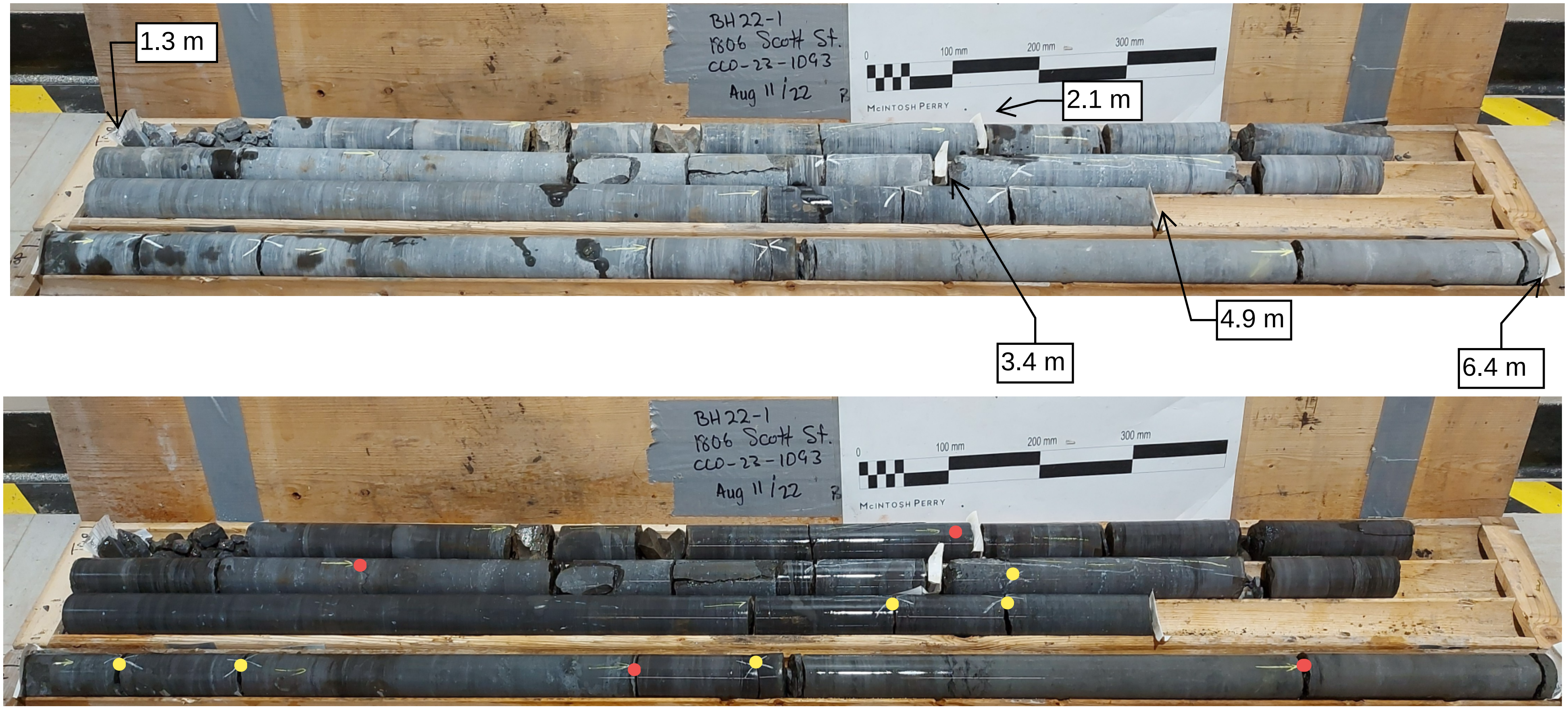
McINTOSH PERRY

Unified Soil Classification System (USCS)



McINTOSH PERRY	Grain-Size Distribution Curve	Figure No. 3
	Fill	
	1806 Scott Street, Ottawa.	CCO-23-1093

BH22-1
Rock Core



- Mechanical Breaks
- Joints

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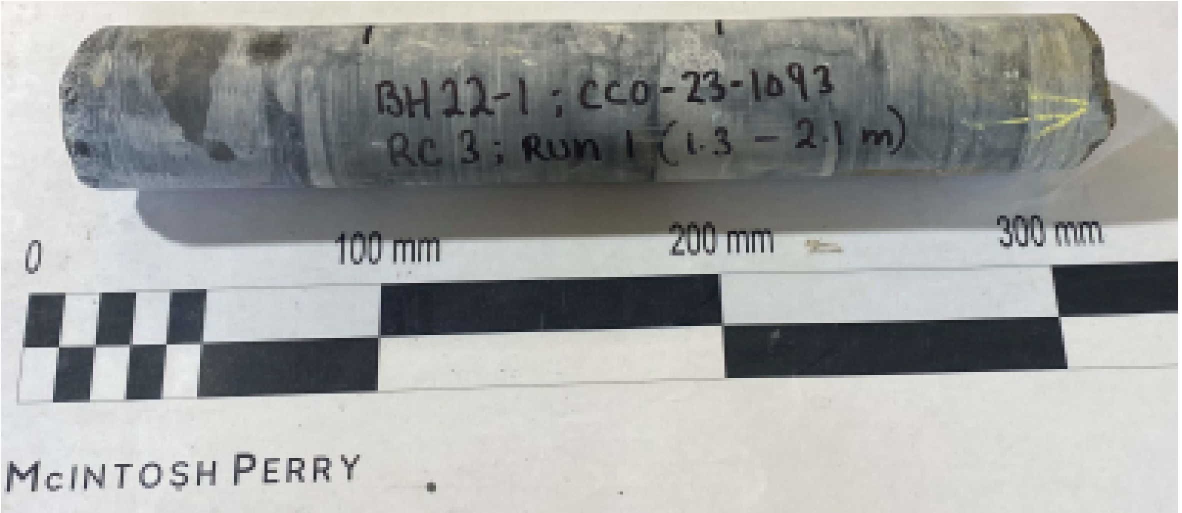
Tel: 613-836-2194 Fax: 613-836-3742
www.mcintoshperry.com

Client: Gabriela Godinez-Lavery
Project Name: 1806 Scott St.
Project Location: Ottawa, ON.

Project No.: CCO-23-1093

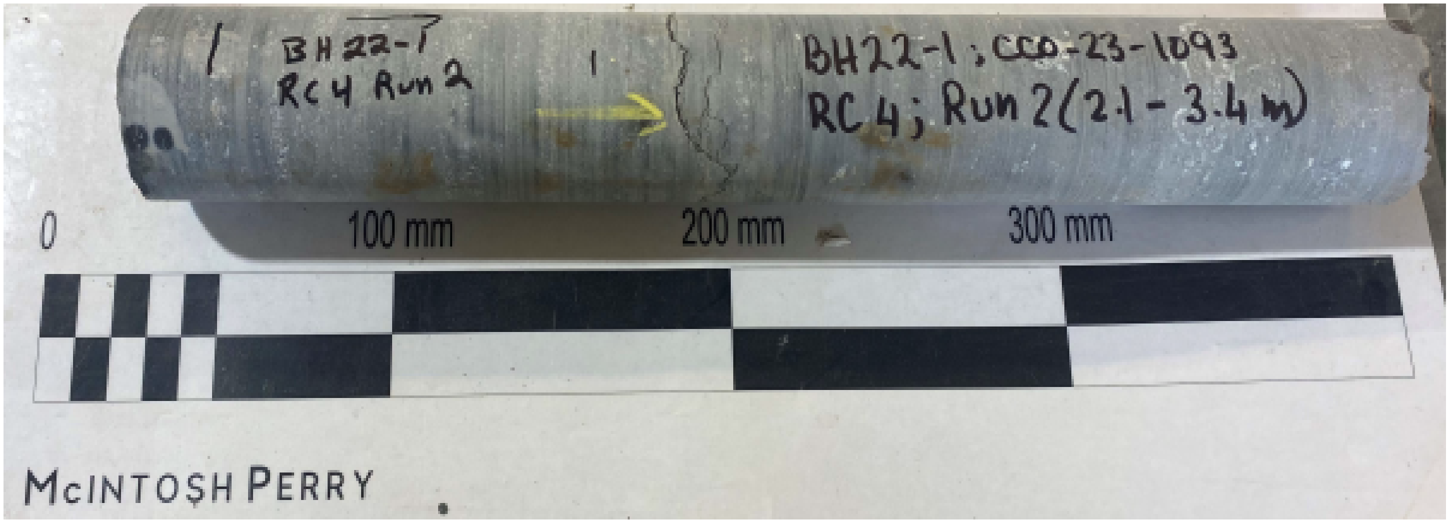
Figure No.: 4

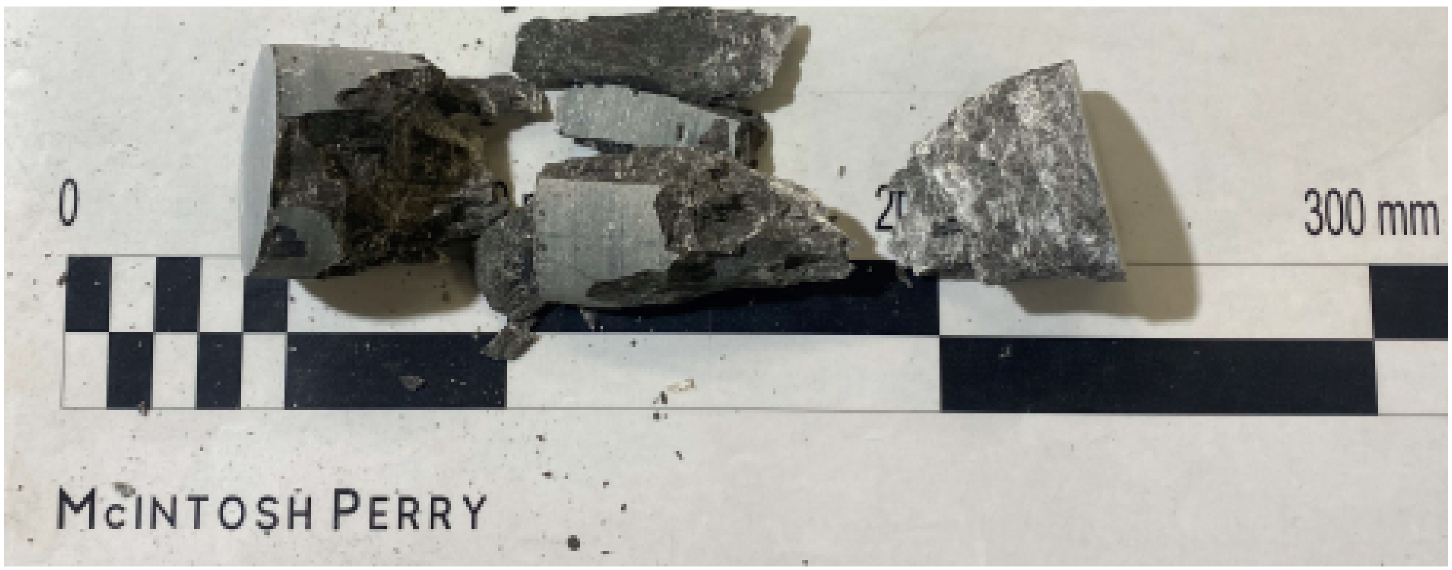
BH22-1
Run 1 (1.3 - 2.1 m)



McINTOSH PERRY 115 Walgreen Road, RR3, Carp, ON K0A 1L0 Tel: 613-836-2194 Fax: 613-836-3742 www.mcintoshperry.com	Client: Gabriela Godinez-Lavery Project Name: 1806 Scott St. Project Location: Ottawa, ON. Project No.: CCO-23-1093 Figure No.: 5

BH22-1
Run 2 (2.1 - 3.4 m)





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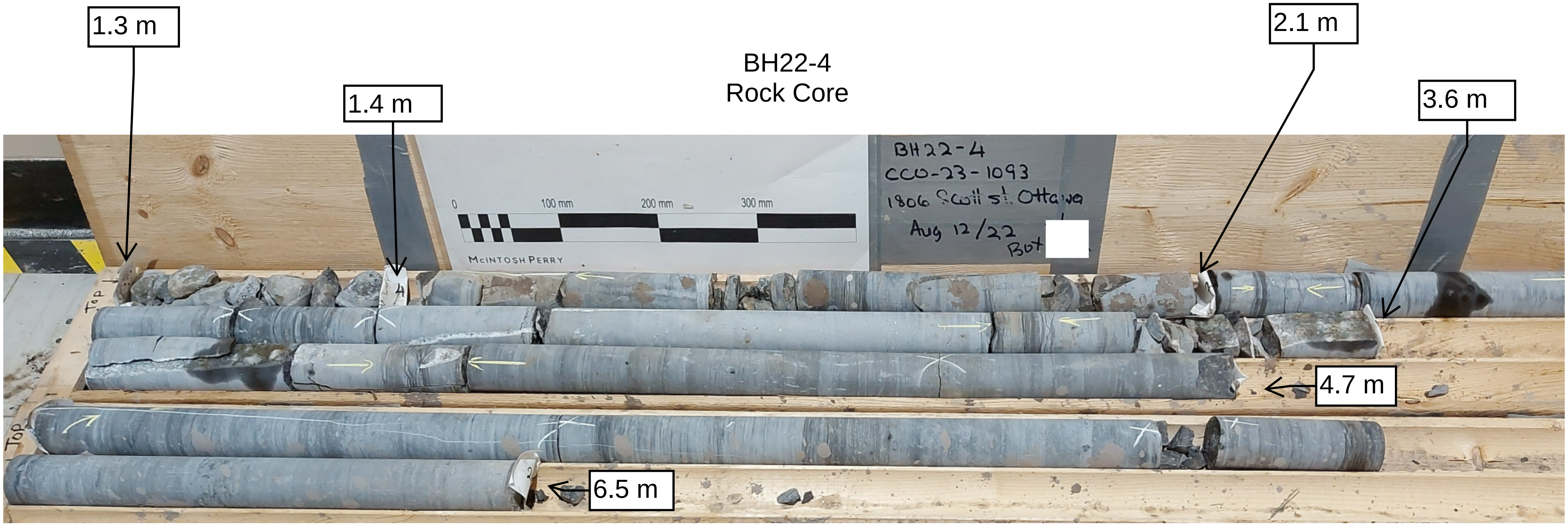
Client: Gabriela Godinez-Lavery
Project Name: 1806 Scott St.
Project Location: Ottawa, ON.

Project No.: CCO-23-1093 Figure No.: 6

BH22-4
Rock Core



Mechanical Breaks

Joints

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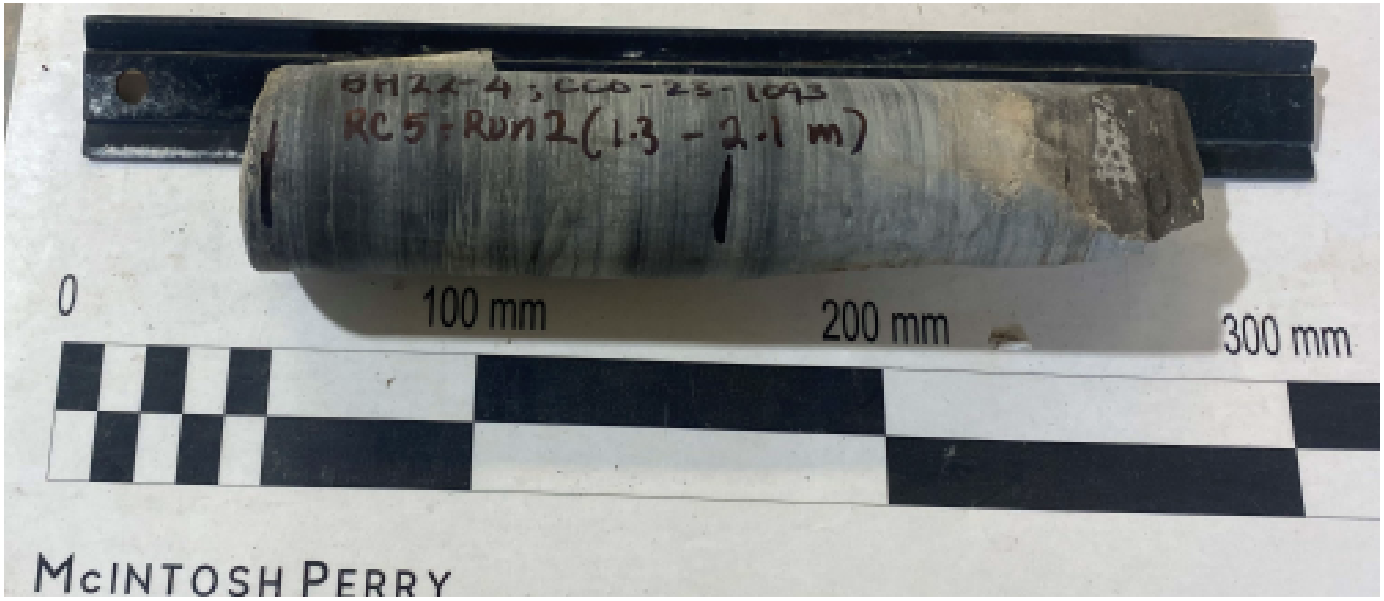
Client: Gabriela Godinez-Lavery
Project Name: 1806 Scott St.
Project Location: Ottawa, ON.

Project No.: CCO-23-1093

Figure No.: 7

BH22-4
Run 2 (1.3 - 2.1 m)



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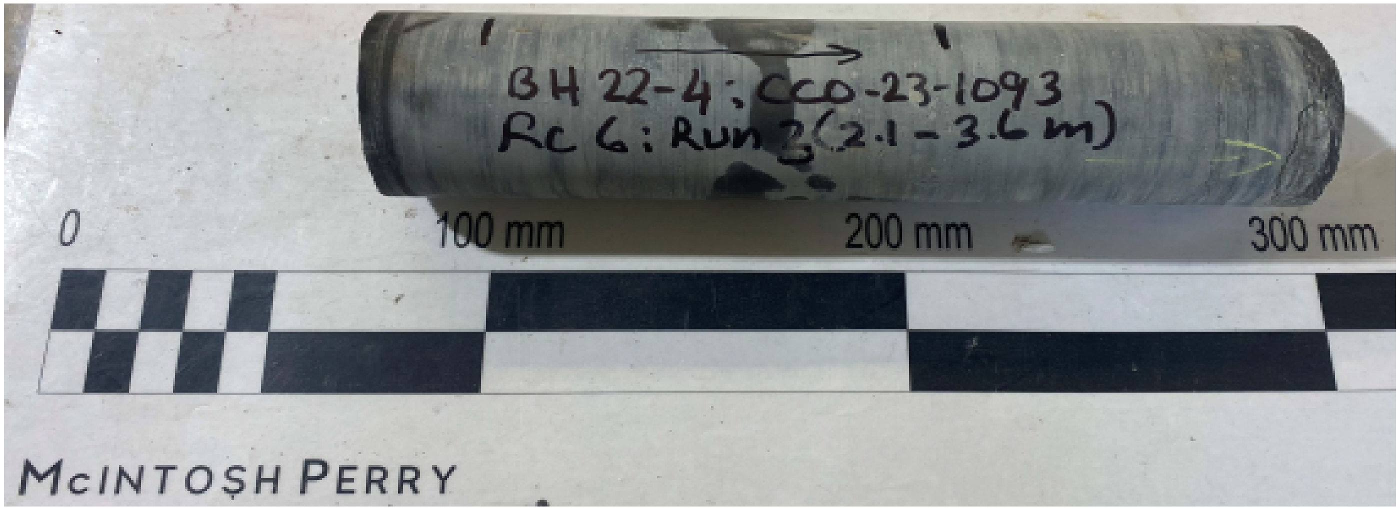
Client: Gabriela Godinez-Lavery
Project Name: 1806 Scott St.
Project Location: Ottawa, ON.

Project No.: CCO-23-1093

Figure No.: 8

BH22-4
Run 3 (2.1 - 3.6 m)



McINTOSH PERRY
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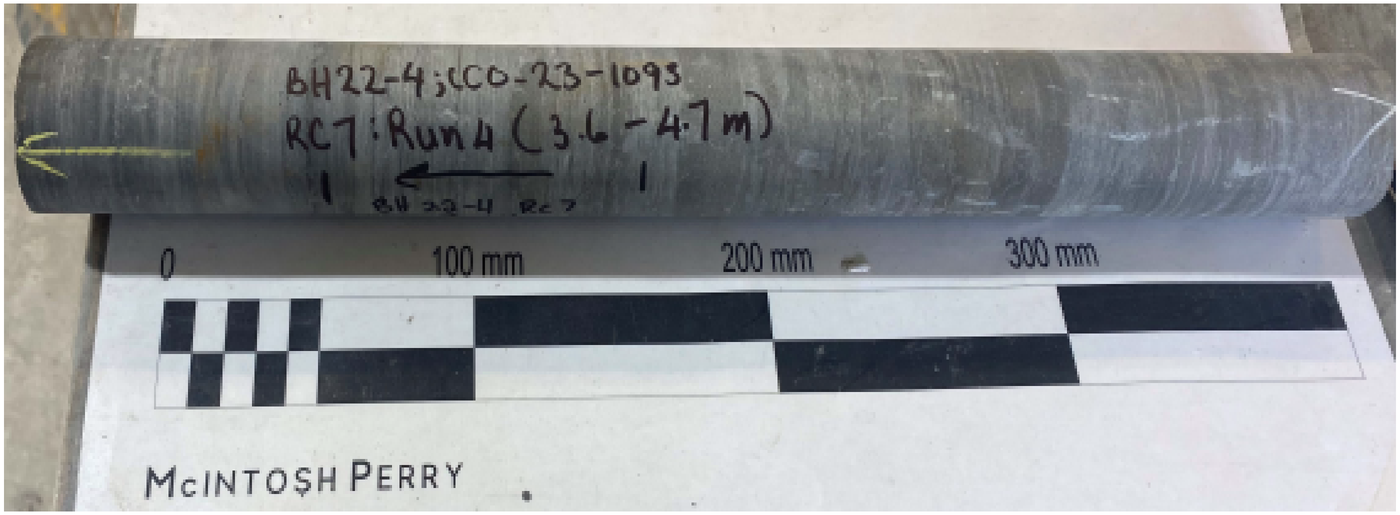
Tel: 613-836-2194 Fax: 613-836-3742
www.mcintoshperry.com

Client: Gabriela Godinez-Lavery
Project Name: 1806 Scott St.
Project Location: Ottawa, ON.

Project No.: CCO-23-1093

Figure No.: 9

BH22-4
Run 4 (3.6 - 4.7 m)



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Client: Gabriela Godinez-Lavery
Project Name: 1806 Scott St.
Project Location: Ottawa, ON.

Project No.: CCO-23-1093

Figure No.: 10

MCINTOSH PERRY

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO23-1093	**Date Issued:**	Aug 18,2022
Lab No.:	OL-22070	**Report No.:**	1 of 2
Project Name:	1806 Scott St. Ottawa		

Core No.:	1	**Moisture Condition:**	Dry as received		
Borehole Location:	BH22-1	**RC / Run:**	RC-3 / Run-1	**Depth (ft):**	1.3m-2.1m
Date Sampled:	Aug 11,2022	**Received:**	Aug 12,2022	**Tested:**	Aug 17,2022
Core No.:	2	**Moisture Condition:**	Dry as received		
Borehole Location:	BH22-1	**RC / Run:**	RC-4 / Run-2	**Depth (ft):**	2.1m-3.4m
Date Sampled:	Aug 11,2022	**Received:**	Aug 12,2022	**Tested:**	Aug 17,2022
Core No.:	3	**Moisture Condition:**	Dry as received		
Borehole Location:	BH22-4	**RC / Run:**	RC-5 / Run-2	**Depth (ft):**	1.3m-2.1m
Date Sampled:	Aug 11,2022	**Received:**	Aug 12,2022	**Tested:**	Aug 17,2022

Core No. :	1	2	3
Diameter (mm)	47.4	47.5	47.4
Thickness/Height (mm)	98.3	97.3	97.3
Density (Kg/m^3)	2752	2709	2687
Compressive Strength (Mpa)	256.9	180.6	187.3
Mass of Core (g)	477.38	467.54	465.2
Description of Failure	2 & 3	2	3

Remarks: Type 2 - Relatively well-formed cone on one end, vertical cracks running through end, no well formed cone on other end.

Type 3 - Columnar Vertical cracking through both ends, no well-formed cones.

Reviewed By: ______________________ **Date:** Aug 18,2022

Jason Hopwood-Jones
Laboratory Manager

McINTOSH PERRY

Unconfined Compressive Strength of Intact Rock Cores
ASTM D7012 Method C

Project No.:	CCO23-1093	**Date Issued:**	Aug 18,2022
Lab No.:	OL-22070	**Report No.:**	2 of 2
Project Name:	1806 Scott St. Ottawa		

Core No.:	4	**Moisture Condition:**	Dry as received		
Borehole Location:	BH22-4	**RC / Run:**	RC-6 / Run-3	**Depth (ft):**	2.1m-3.6m
Date Sampled:	Aug 11,2022	**Received:**	Aug 12,2022	**Tested:**	Aug 17,2022
Core No.:	5	**Moisture Condition:**	Dry as received		
Borehole Location:	BH22-4	**RC / Run:**	RC-7 / Run-4	**Depth (ft):**	3.6m-4.7m
Date Sampled:	Aug 11,2022	**Received:**	Aug 12,2022	**Tested:**	Aug 17,2022
Core No.:		**Moisture Condition:**			
Borehole Location:		**RC / Run:**		**Depth (ft):**	
Date Sampled:		**Received:**		**Tested:**	

Core No. :	4	5	
Diameter (mm)	47.6	47.6	
Thickness/Height (mm)	98	98.2	
Density (Kg/m^3)	2694	2738	
Compressive Strength (Mpa)	203.3	236.7	
Mass of Core (g)	469.98	477.48	
Description of Failure	3	2 & 3	

Remarks: Type 2 - Relatively well-formed cone on one end, vertical cracks running through end, no well formed cone on other end.

Type 3 - Columnar Vertical cracking through both ends, no well-formed cones.

Reviewed By: ______________ **Date:** Aug 18,2022

Jason Hopwood-Jones
Laboratory Manager

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Certificate of Analysis

McIntosh Perry Consulting Eng. (Nepean)
215 Menten Place, Unit 104
Nepean, ON K2H 9C1
Attn: Jason Hopwood-Jones

Report Date: 24-Aug-2022
Order Date: 17-Aug-2022

Client PO: CCO 23-1093
Project: 1806 Scott St.
Custody: 67079

Order #: 2234332

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

Paracel ID	Client ID
2234332-01	BH22-5 SS-2 2'-4'

Approved By:

Alex Enfield, MSc
Lab Manager

Certificate of Analysis

Report Date: 24-Aug-2022

Client: McIntosh Perry Consulting Eng. (Nepean)

Order Date: 17-Aug-2022

Client PO: CCO 23-1093

Project Description: 1806 Scott St.

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	23-Aug-22	24-Aug-22
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	23-Aug-22	23-Aug-22
Resistivity	EPA 120.1 - probe, water extraction	23-Aug-22	23-Aug-22
Solids, %	Gravimetric, calculation	22-Aug-22	22-Aug-22

Summary of Criteria Exceedances

(If this page is blank then there are no exceedances)

Only those criteria that a sample exceeds will be highlighted in red

Regulatory Comparison:

Paracel Laboratories has provided regulatory guidelines on this report for informational purposes only and makes no representations or warranties that the data is accurate or reflects the current regulatory values. The user is advised to consult with the appropriate official regulations to evaluate compliance. Sample results that are highlighted have exceeded the selected regulatory limit. Calculated uncertainty estimations have not been applied for determining regulatory exceedances.

Sample	Analyte	MDL / Units	Result	-	-

PARACEL LABORATORIES LTD.

Order #: 2234332

Certificate of Analysis

Report Date: 24-Aug-2022

Client: McIntosh Perry Consulting Eng. (Nepean)

Order Date: 17-Aug-2022

Client PO: CCO 23-1093

Project Description: 1806 Scott St.

	Client ID:	BH22-5 SS-2 2'-4'	-	-	-	-	-
	Sample Date:	11-Aug-22 09:30	-	-	-		
	Sample ID:	2234332-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	95.9	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.79	-	-	-	-	-
Resistivity	0.1 Ohm.m	71.7	-	-	-	-	-
Anions							
Chloride	5 ug/g	<5	-	-	-	-	-
Sulphate	5 ug/g	32	-	-	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions								
Chloride	ND	5	ug/g					
Sulphate	ND	5	ug/g					
General Inorganics								
Resistivity	ND	0.10	Ohm.m					

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g	ND			NC	20	
Sulphate	34.2	5	ug/g	31.5			8.2	20	
General Inorganics									
pH	7.90	0.05	pH Units	7.86			0.5	10	
Resistivity	24.6	0.10	Ohm.m	24.6			0.1	20	
Physical Characteristics									
% Solids	85.9	0.1	% by Wt.	85.7			0.2	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	106	5	ug/g	ND	106	82-118			
Sulphate	129	5	ug/g	31.5	97.8	80-120			

Qualifier Notes:

Login Qualifiers :

Received at temperature > 25C

Applies to Samples: BH22-5 SS-2 2'-4'

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 unlesss otherwise noted.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Any use of these results implies your agreement that our total liabilty in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstances be liable to you in connection with this work.

1806 SCOTT STREET, OTTAWA, ONTARIO.
GEOTECHNICAL AND FOUNDATION REPORT

APPENDIX E
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: 45.400N 75.744W **User File Reference:** 1806 Scott Street 2022-10-05 20:11 UT

Requested by: McIntosh Perry Consulting Engineering Ltd.

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)	0.632	0.384	0.247	0.089
Sa (0.5)	0.307	0.185	0.121	0.043
Sa (1.0)	0.137	0.087	0.055	0.017
Sa (2.0)	0.046	0.028	0.018	0.006
PGA (g)	0.322	0.200	0.122	0.038

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