439 CHURCHILL AVE – GEOTECHNICAL REPORT

Project No.: CCO-21-3806

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FOUNDATION DESIGN RECOMMENDATION REPORT
439 Churchill Ave, Ottawa, Ontario

1.0 INTRODUCTION

This report presents the factual findings obtained from a geotechnical investigation performed at the above-mentioned site for the proposed four-storey addition at the back of the existing two-storey building, with no basement. The fieldwork was carried out on April 14, 2021, and comprised of three boreholes advanced to a maximum depth of 5.9 m.

The purpose of the investigation was to explore the subsurface conditions at this site and to provide borehole location plans, a record of borehole logs, and laboratory test results. This report provides anticipated geotechnical conditions influencing the design and construction of the proposed four-storey addition, as well as recommendations for foundation design. 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.

The investigation was performed at the request of Grepault Developments Ltd. (the Client).

2.0 SITE DESCRIPTION

2.1 Existing Site Conditions

Currently, the site is occupied by a two-level mixed-used commercial and residential building with a detached garage at the back of the property. A driveway extends to the parking lot at the back of the property. The property is located in a densely populated mixed residential and commercial area. The site is accessible from Churchill Avenue at the west side. Churchill Public School is facing the site across Churchill Ave. To the north and south of the site, there are two-level buildings of mixed commercial and residential use. A wooden fence and mature trees separate the site from the residential dwellings at the back of the parking lot. The property limits are shown in figure 2, in Appendix B.

2.2 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 Eastern Ontario indicate the site is underlain Paleozoic bedrock terrain surrounded by areas of till deposits. The till deposits in this region are predominantly stone-poor, sandy silt to silty sand-textured till on Paleozoic terrain.

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. The sediments themselves in the valley are deep silty clay. Although the clay deposits are grey like the limestones that underlie them in part, they are only mildly calcareous and likely derived from the more acidic rock of the Canadian Shield.

3.0 PROJECT UNDERSTANDING

The proposed development is a four-storey mixed-use addition, that will contain 4 residential units and two commercial units with no basement. The proposed work consists of a third-floor addition to the existing two-story building that will contain 1 residential unit and a four-storey rear addition that will contain 3 residential units. The two commercial units in the existing building will remain. The existing detached garage will be removed and four parking spaces are proposed in the rear yard.

It is understood that design recommendations for foundation and light-duty pavement structures for the parking areas and access driveway will be required for the proposed addition.

4.0 FIELD PROCEDURES

The staff of McIntosh Perry Consulting Engineers (McIntosh Perry) visited the site before the drilling investigation to mark out the proposed borehole locations to obtain utility clearance to identify the location of underground infrastructures. 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 the drilling work.

The equipment used for drilling was owned and operated by George Downing Estate Drilling Ltd. of Hawkesbury, Ontario. Boreholes were advanced using hollow stem augers aided by a track-mounted CME 55 LC drill rig. Boreholes were advanced to a maximum depth of 5.9 m (El. 71.9 m) below the ground level. 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 bedrock was cored and sampled in all boreholes. Rock cores were obtained by diamond drilling and wireline tooling. Rock core samples were retrieved in dual wall core barrel samplers to reduce the risk of mechanical breaks. A standpipe monitoring well was installed in BH21-02 to measure the groundwater table in the site.

Field test procedures are listed below:

ASTM D1586 – Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.

Boreholes were backfilled with auger cuttings and restored to the original surface with cold patch asphalt. A summary of borehole designations, location and depths is shown in Table 4.1. Borehole locations are shown in Figure 2, included in Appendix B.

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

BH No.	Coordinates (Geodetic)		Surface El. (m)	Drilling Depth (m)	Drilling El. (m)	Coring Depth (m)	Coring El. (m)
	Latitude	Longitude					
21-01	45.390587335	-75.752625493	78.03	1.0	77.0	4.5	73.5
21-02	45.431322305	-75.689107549	58.32	1.1	76.7	10.5	47.9
21-03	45.431252427	-75.689141343	58.35	2.3	75.5	13.2	45.2

Field investigation, including drilling and sampling, were supervised on a full-time basis by McIntosh Perry engineering 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 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.

5.0 IDENTIFICATION AND TEST PROCEDURES

Laboratory testing on representative SPT and rock core samples was performed at McIntosh Perry geotechnical lab and included grain size distribution analysis and rock compressive strength test. The laboratory tests to determine index properties follow the American Society for Testing Materials (ASTM) test procedures.

Paracel Laboratories Ltd., in Ottawa, carried out chemical tests on a representative soil sample to determine the soil corrosivity characteristics. Laboratory tests are included in Appendix E.

Test procedures are listed below;

ASTM D422 – Standard Test Method for Particle-Size Analysis of Soils (LS-702);
Five (5) representative rock cores underwent uniaxial compressive strength to determine the unconfined compressive strength of intact rock cores. The tests were performed at McIntosh Perry's geotechnical lab, located in Ottawa, in accordance with American Society for Testing Materials (ASTM) test procedures.

ASTM D7012 – Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures.

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

6.0 SUBSURFACE CONDITIONS

6.1 Subsurface Conditions

In general, the site stratigraphy consists of three distinctive layers; asphalt surface over engineered fill, followed by a thin deposit of glacial till underlain by bedrock. The till layer is composed of sand and gravel with traces of silt and clay with the presence of cobbles and occasional boulders. The till layer is underlain by bedrock, which was sampled and proved in all boreholes. For classification purposes, the soils encountered at this site can be divided into three distinctive strata.

- Asphalt & Granular Fill
- Till
- Bedrock

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 unconfined compressive strength of intact rock cores are presented in Appendix D. Chemical (corrosivity) tests results are in included in Appendix E. Description of the strata encountered are given below.

6.1.1 Asphalt & Granular Fill

An asphalt layer was observed in all boreholes advanced followed by a fill layer. The asphalt thickness ranges between 20 to 50 mm. The fill layer is composed of approximately 0.7 to 1 m of sand and gravel with some silt.

The fill in general was observed to be dark brown to black, dry to damp, and loose to compact. The SPT ‘N’ values range from 6 to over 13 blows/300mm.

6.1.2 Till

The till layer is composed mainly of sand and gravel with some silt and with presence of cobbles and occasional boulders. The till was observed brown to light brown, dry, and dense, with SPT ‘N’ values ranging from 36 to 48 blows/300mm. A representative sample underwent grain size analysis testing, and the sample was observed to contain 42% gravel, 37% sand, 21% silt and clay. A summary of the grain size distribution for the tested sample from this layer is shown in Table 6-2.

Table 6-2: Grain Size Distribution of the Till Layer

Grain Size	(%)
Gravel	42
Sand	37
Silt and clay	21

6.1.3 Bedrock

Below the till layer, bedrock was encountered in all boreholes. The bedrock surface was observed at approximately the same level in BH21-01 and 21-02 (El. 77 to 76.7 m) and dropped to El. 75.5 m in BH21-03. Mud seams were observed in rock core joints in BH21-02, core "RC-04" and BH21-03, sores "RC-05 and RC-06". The mud seams ranged between 90 to 130 mm thick and was observed at depth ranges between 2.5 and 4.5 m below the ground surface.

The bedrock was cored and confirmed in all boreholes. Based on the retrieved samples and rock core quality designation (RQD), the rock was identified as slightly weathered Limestone. Five selected rock core samples were tested for uniaxial or unconfined compressive strength (UCS), and results are shown in Table 6-3. Rock core photo logs are shown in Appendix D.

Table 6-3 Rock Coring Depths and Quantities

Borehole	Borehole Surface El. (m)	Bedrock Surface Depth (m)	Rock Surface El. (m)	Total Length of Cored Rock (m)
21-01	78.03	1.0	77.0	3.5
21-01	77.83	1.1	76.7	3.4
21-03	77.77	2.3	75.5	3.6

Table 6-4: Rock Cores Unconfined Compressive Strengths

Borehole	Rock Core	Sample Depth (m)	Sample El. (m)	UCS (MPa)
21-01	RC-4	2.87	75.2	187.9
21-01	RC-5	4.44	73.6	152.6
21-02	RC-4	2.46	75.4	138.8
21-02	RC-5	4.47	73.4	131.1
21-03	RC-4	2.41	75.4	191.3

6.2 Groundwater

A standpipe monitoring well was installed in BH21-and 21-03A, and its assembly is shown on the borehole log. At the time of investigation, groundwater was not measured in open boreholes due to the added water for coring. Reading of groundwater level was taken in May 1, 2021 and the groundwater was at 2.42 m depth (EL. 75.41 m) below the existing ground level. Groundwater level may be expected to fluctuate due to seasonal changes.

6.3 Chemical Analysis

The chemical test results conducted by Paracel Laboratories in Ottawa, Ontario, to determine the resistivity, pH, sulphate and chloride content of a representative soil sample are shown in Table 6-5 below. Chemical test results are included in Appendix E.

Table 6-5: Soil Chemical Analysis Results

Borehole	Sample	Depth / El. (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (Ohm-cm)
21-03	SS-02	0.8 – 1.37	7.48	0.0082	0.0293	1420

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 and bedrock 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 2012 Ontario Building Code (OBC) extending the Limit State Design approach.

7.2 Overview

It is understood that the proposed structure is a four-storey addition with no basement. It is also understood that the finished floor elevation for the proposed development will be approximately the same as the existing building.

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

- The exact founding level of the existing mixed-use building is unknown.
- The expected foundation loads for the four (4) levels addition can be supported on the underlying shallow bedrock by spread footing founded on or within the bedrock.
- 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 surface, using caissons. Otherwise, Site Class D would be required.
- The footprint of the proposed addition is within the existing parking area. The underlain fill is approximately 0.8 to 1.1 m and was observed to be loose to compact. The existing fill is not suitable to support the spread footing system and needs to be removed from the footprint of the proposed foundation.
- If the existing fill to be reused for the slab-on-grade, parking areas, and site grading, the fill needs to be sampled and pass gradation and standard proctor compaction tests.

7.3 Site Preparation

As previously noted, the site of the proposed addition is underlain by 0.8 to 1.1 m of fill. The existing fill below the asphalt surface contains different portions of sand, gravel, silt and clay with different competence levels throughout the site. The fill is not suitable to support the structural loads of the proposed addition. If the existing fill to be reused for the slab-on-grade, parking areas, and site grading, it shall meet the material specification of OPSS 1010.

All footings are expected to be bearing on bedrock. It is recommended to use concrete grouting to improve the rock surface quality before constructing the footings.

The surface and groundwater inflow to the excavation can be handled by pumping from well-filtered sumps established on the floor of the excavation. The expected groundwater level is below the depth of excavation except in BH21-03 in which the bedrock was encountered at the same level of the groundwater. Site shall be dewatered before pouring concrete footings. The actual inflow into the excavation will depend on many factors including, but not limited to, the contractor’s schedule, the rate of excavation, the size of the excavation, and the time of the year at which the excavation is to occur.

7.4 Foundation Excavation

It is understood that no basement is provisioned. The expected foundation level will be at about an approximate elevation of 76.7 m. Excavation for the construction of the foundation will proceed through the fill and native till to the bedrock. Excavation of overburden soil shall be performed using low impact excavating equipment. The Occupational Health and Safety Act (OHSA) of Ontario indicated that side slopes in the rock fill above the water table could be classified as Type 3 soil and sloped no steeper than 1H:1V.

Before proceeding with the full excavation, the contractor shall advance test pits on the side of the existing building to ensure the existing footings will not be undermined. In such case a structural engineer shall design proper underpinning. If underpinning is required, it is prudent to evacuate the building until the underpinning is completed.

Depending on space restrictions, shoring may be required to carry out the excavations. Shoring recommendations shall be reflected in the structural drawings.

The groundwater level in the proximity of the area of the proposed development was measured in one monitoring well installed in BH21-02. The reading was taken 17 days after installation to allow the groundwater table to come to equilibrium and stabilize in the wells. The groundwater table was at El. 75.41 m, which is below the depth of excavation except in BH21-03 in which the bedrock was encountered at the same level of the groundwater. Also, depending on the construction season, surface runoff can seep into the excavation. The site shall be dewatered before pouring concrete footings.

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 and Climate Change (MOEC) if a volume of water greater than 400,000 liters per day is pumped from the excavation under normal operation. However, for more than 50,000 liters per day, the water taking will not require a PTTW, but will need to be registered in the EASR as a prescribed activity. Based on the encountered stratigraphy, the amount of groundwater intake, and excavation dimensions, a PTTW is not required.

7.5 Foundations

In general, the soil stratigraphy in the area of the proposed four-storey addition consists of a layer of fill overlying a deposit of relatively thin till. The till composes mainly of silty sand and gravel with presence of cobbles and occasional boulders, over sedimentary and metasedimentary bedrock. The bedrock surface is slightly weathered and slightly downslope (1V:4H) towards the north side of the property with level ranges between El. 77 to 75.5 m.

It is understood that the underside of the foundations will likely be at about 76.7 m; therefore, it will be supported directly on the existing bedrock using:

- Spread footings founded on or within the bedrock; or,
- Spread footings founded on mass concrete that extends to the bedrock surface.

All footings are expected to be bearing on bedrock. The existing till/fill is not a suitable subgrade stratum. Differential settlement may occur in the area where footings are founded on both bedrock and till due to the difference in material stiffness and settlement properties. It is therefore proposed that the entire structure be supported on the underlying bedrock using shallow spread footing foundations.

7.5.1 Bearing Resistance

Provided there are no continuous soil-filled seams or mud seams present at shallow depth in the bedrock below the founding level, footings on the bedrock surface, or a platform of lean concrete of compressive strength of greater than 15 MPa extending down to the bedrock surface, may be designed using an Ultimate Limit State (ULS) factored bearing resistance of 9,00 kPa.

The ULS factored bearing resistance was estimated using the Rock Mass Rating (RMR) method by Bieniawski (1989). 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:

- Average compressive strength of intact rock rating: An average uniaxial compressive strength of 130 MPa was considered, which results in rating = 12,
- RQD rating: The RQD of the rock core ranges 50 to 75, which results in rating = 13,
- Joint spacing rating: The joint spacing for the rock core samples ranges from 50 -300mm, which gives an estimated rating = 10,
- Joint condition: The joint condition was observed to be slightly rough, and the rating is estimated to be = 20,
- Groundwater rating: groundwater elevation was measured in a monitoring well installed in BH20-7 and was at level 98.3 m. Therefore, the estimated rating for water condition = 4; and
- Orientation rating: Some fractures were observed to be oriented at approximately 0° to 10° with respect to load direction; therefore, unfavorable rating was estimated = -15.

The RMR for the rock approximately equals (36) which can be classified to have fair rock quality.

Assuming the above-noted conditions are provided, the following bearing capacity can be used for structural design.

Table 7-1: Rock Bearing Capacities

Footing Type	ULS (kPa)	SLS (kPa)
square footings	900	600

The provided factored bearing resistance at ULS is based on the uniaxial compressive strength of rock. The size of the selected footing shall be determined by 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 footings sized using the above factored bearing resistance should be negligible. However, since the bedrock is sloped down at approximately 15°, the allowable bearing capacity should be reduced to account for the reduced lateral resistance provided by the smaller mass of rock on the downslope side of the footing. The allowable bearing capacity shall not exceed 600 kPa with a factor of safety of 2.5, and should govern the foundation design.

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 to confirm that the surface has been acceptably cleaned of soil, and that weathered, or excessively fractured bedrock has been removed.

7.5.1.1 Resistance to Lateral Loads

The factored ultimate resistance of the footings to lateral loading 'shear resistance for sliding' across the interface between the footing, and the bedrock may be calculated using Mohr-Coulomb criterion with load and resistance factored given in Table 7-2.

Table 7-2: f Values of Minimum Partial Factors after Meyerhof (1984) (Wyllie 2009)

Category	Item	Load Factor	Resistance factor
Loads	Dead Loads	1.25	--
	Live Loads, Wind, earthquake	1.5	--
	Water Pressure	1.25	--
Shear strength	Cohesion "c" - stability, earth pressure	--	0.65
	Cohesion "c" - Foundation	--	0.5
	Friction angle "ϕ"	--	0.8

7.5.2 Frost Protection

Frost penetration depth for this site is 1.8 m for unheated structures and 1.5 m for heated structures. Frost penetration depth is estimated based on the OPSD 3090.101, Foundation Frost Penetration Depths for Southern Ontario.

Generally, rock is not frost susceptible material. However, trapped and accumulated water on rock may damage the footing due to frost effect if adequate insulation is not provided. A proper drainage system shall be provided to mitigate water gathering/water freezing at the foundation level. The building designer shall provide the details for insolation and drainage.

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 synthetic thermal rigid insulation for frost protection purposes.

7.6 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 (2500 years return period) are as indicated in Table 7-3, shown below and in Appendix F;

Table 7-3: Selected Seismic Spectral Responses (2% in 50 Yrs) – NRCan 2010

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.634	0.307	0.137	0.046	0.323

Based on the subsurface condition and field and SPT values, the site can be classified as 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, using caissons. Otherwise, Site Class D would be required.

7.7 Engineered Fill

The proposed engineered fill, beyond the foundation influence zone, can be any material conforming to granular criteria as outlined in OPSS.MUNI 1010. Material conforming to ‘Granular’ criteria are considered free draining, compactable and can be utilized as the engineered fill. This can apply to the backfill beyond foundation walls. The engineered fill shall be compacted to a minimum of 98% Standard Proctor Maximum Dry Density SPMDD.

It is understood that grade adjustment is may be required for this site. There are no concerns with respect to the long-term settlement of the till.

All fill 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 organics or loose sand should be removed before placing engineered fill material.

7.8 Slabs-on-Grade

Slab-on-grade is 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% SPMDD. The requirements of the fill underneath slab-on-grade is noted in section 7.7 Engineered Fill.

If the slab on grade is proposed to support concentrated linear or point loads, the design loading shall be indicated in the structural specifications.

It is recommended that subgrade preparation and compaction efforts are approved under the supervision of a geotechnical representative.

Knowing the slab-on-grade cannot function as a foundation element for the proposed building, the modulus of subgrade reaction (k) might be needed for the design of the slab-on-grade to support local loads in the basement. Modulus of 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, it was assumed that 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 20,000 kN/m^2/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.

7.9 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 calculation of earth pressure on foundation walls. The following parameters are recommended for the granular backfill.

Table 7-4: Lateral Pressure parameters for Granular A and B and Horizontal Backfill

Pressure Parameter		Expected Value			
		Granular A	Granular B	Other OPSS1010 ‘Granular’	Native Till
Unit Weight (γ) kN/m^3	Above groundwater	22.5	21.7	21.7	20
	Below groundwater	12.7	11.9	11.9	10.2
Angle of Internal Friction (ϕ)		35°	32°	31°	31°
Coefficient of Active Earth Pressure (k_a)		0.27	0.31	0.32	0.32
Coefficient of Passive Earth Pressure (k_p)		3.69	3.23	3.12	3.12
Coefficient of Earth Pressure at Rest (k_o)		0.43	0.47	0.48	0.48

7.10 Pavement Structure

It is understood that the parking lot, access roadway, and the rest of the paved areas of the proposed development are to be used by residents with light to medium weight passenger vehicles on a daily basis. Pavement structure is most likely to be placed on engineered fill material overlaying native subgrade. If the existing fill to be reused for the parking areas, access roadway, and the rest of the paved areas, it shall be sampled and tested to meet the material specification of OPSS 1010. It is recommended to be replaced with compacted Granular B Type II or Granular A and compacted to 98% SPMDD. In addition, should grade raise be required, compacted Granular B Type II or Granular A should be placed as needed and compacted to 98% SPMDD prior to construction of pavement structure.

The proposed pavement structure for the parking area and the access road is included in Table 7-5:

Table 7-5: “Medium Duty” Pavement Structure

Material		Thickness (mm)
Surface	Superpave 12.5 mm, PG 58-34	40
Base	OPSS Granular A	150
Sub-base	OPSS Granular B Type II	300

The base and subbase materials, i.e., Granular A for base and Granular B Type II for subbase, shall be in accordance with OPSS 1010. Both base and sub-base should be compacted to 100% SPMDD. Asphalt layers should be compacted to comply with OPSS 310. Where the pavement structure is to be placed on engineered fill, the upper 600 mm of the fill should be compacted to 98% SPMDD to act as subbase.

7.11 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 6-6.

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 slightly to nonaggressive range.

8.0 CONSTRUCTION CONSIDERATIONS

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

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

Foundation walls should be backfilled with free-draining material with granular material conforming to OPSS 1010 Granular criteria. The native soil is not a suitable material for compaction and shall not be used for any fill work supporting structural 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 prior to pouring the concrete to ensure that strata having adequate bearing capacity have been reached, and the bearing surfaces have been properly prepared.

Vibration monitoring should be carried out during excavation and construction to ensure that the vibration levels at the existing surrounding structures and utilities are maintained below tolerable levels. A maximum peak particle velocity of 50 mm/sec is recommended.

9.0 GROUNDWATER SEEPAGE

The groundwater table is expected to be within the excavation level at the north side of the proposed development. In addition, depending on the construction season, surface runoff can flood into the excavation and surface runoff water and groundwater may present above the depth of excavation. Hydraulic conductivity value of the native till is expected to be approximately $1x10^{-3}$ m/s. This hydraulic conductivity values are estimated based on soil gradation analysis. In-situ percolation tests were not performed as part of this investigation. The suggested hydraulic conductivity value can be used for the selection of the pump capacity for dewatering. If excavation proceeds below the groundwater level, the site shall be dewatered before pouring concrete footings. Groundwater elevation is expected to fluctuate seasonally. Any surface water infiltrating into the open excavation can be removed through conventional sump and pump methods. The subgrade shall be kept dry at all times, especially before compaction and proof rolling.

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. However, for more than 50,000 liters per day, the water taking will not require a PTTW, but will need to be registered in the EASR as a prescribed activity. Based on the encountered stratigraphy, the amount of groundwater intake, and excavation dimensions, a PTTW is not required.

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 City of Ottawa Sewer Use By-law limits, and a separate sewer discharge permit or City approval is required.

10.0 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 rigid 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 pavement/fill, and till. Excavating of overburden soil shall be performed using conventional hydraulic excavating equipment. The Occupational Health and Safety Act (OHSA) of Ontario indicated that till soil is classified as Type 3 soil, and excavation side slopes must be sloped at a minimum of 1H:1V or be shored. if space restriction is encountered, the excavations can be carried out within closed sheeting, which is fully braced to resist lateral earth pressure.

The till is composed of different portions of sand and gravel with the presence of cobbles and occasional boulders. Where fill and till are encountered below the invert level, these materials should be sub-excavated from below the pipes and the site services should be founded on engineered fill to prevent concentration contact points. 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.

To extend the life of buried utilities, it is recommended utility bedding and backfill to be separated from the native soil by filter geotextile to prevent cross migration of fine materials.

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.

LICENSED PROFESSIONAL ENGINEER
N. AZIZI TAVAKKOLI
100150911
Feb. 4/2022
PROVINCE OF ONTARIO

Mohammed Al-Khazaali, Ph.D., P.Eng.
Geotechnical Engineer

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

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.

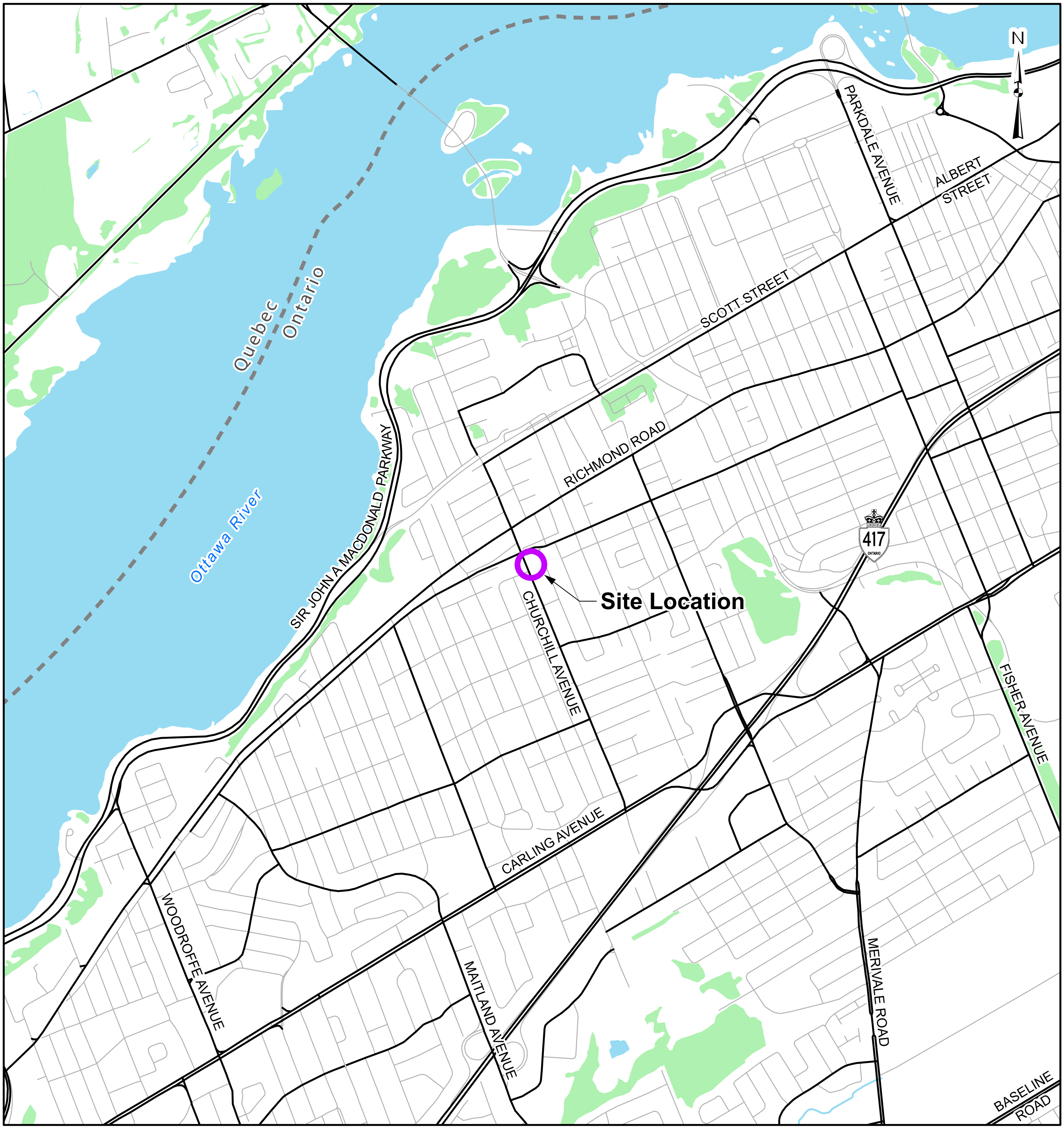
Under no circumstances shall the liability of McIntosh Perry for any claim in contract or in tort, related to the services provided and/or the content and recommendations in this report, exceed the extent that such liability is covered by such professional liability insurance from time to time in effect 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.

439 CHURCHILL AVE – GEOTECHNICAL REPORT

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:
GREPAULT DEVELOPMENTS LTD.

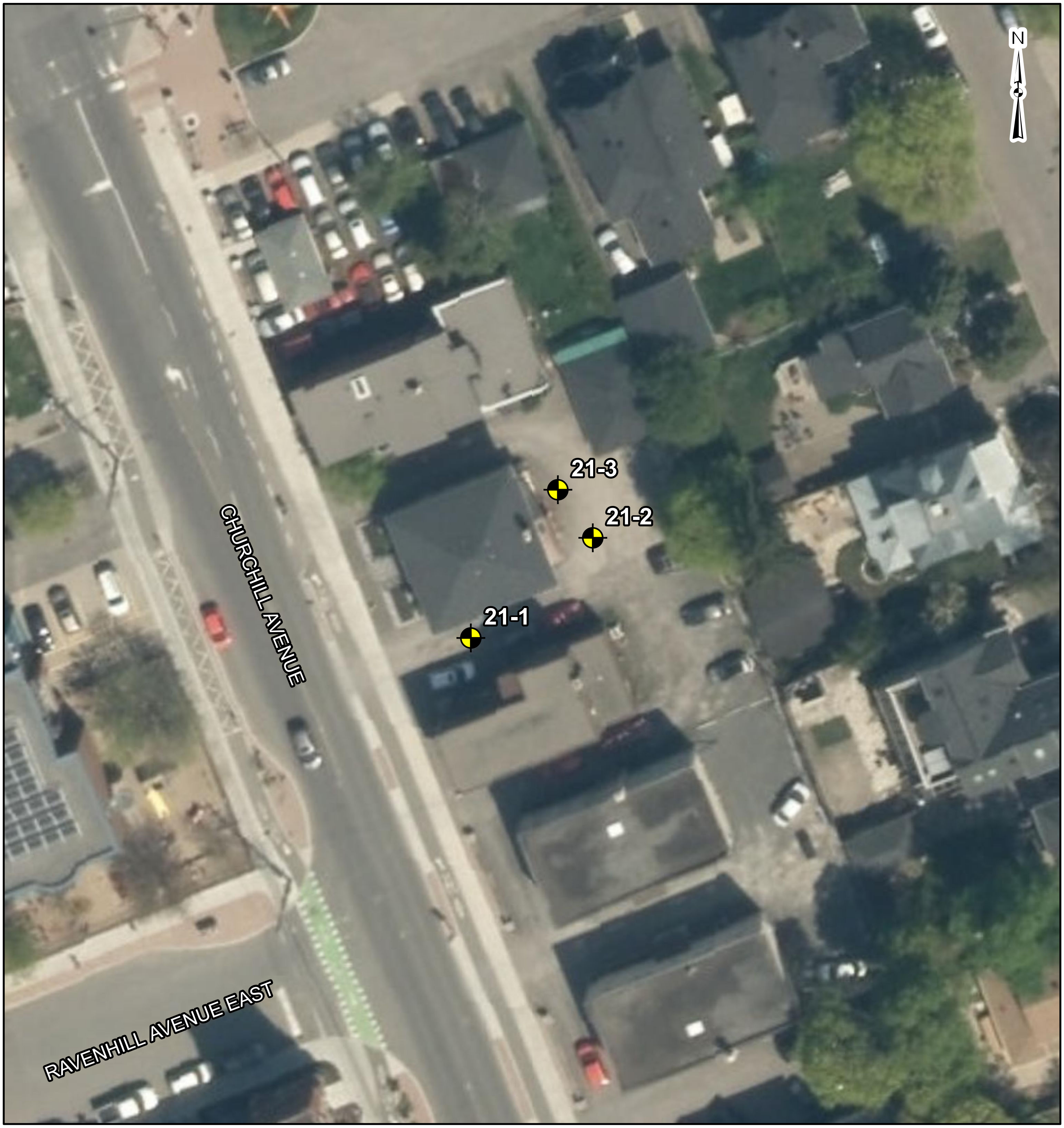
PROJECT: GEOTECHNICAL INVESTIGATION
439 CHURCHILL AVE, OTTAWA, ON

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-21-3806		FIGURE:
Date	Apr., 26, 2021	1
GIS	EU	
Checked By	MA	

C:\Users\e.ungun\McIntosh Perry\GIS - Documents\Projects\2021\CCO\CCO-21-3806 Grepault Dev - SPA - 439 Churchill Ave\aprx\Geotech\CCO-21-3806_Geotech.aprx



10 5 0 10
Scale 1:500 Metres

LEGEND

- ⊕ Borehole Location

REFERENCE

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

CLIENT:
GREPAULT DEVELOPMENTS LTD.

PROJECT: GEOTECHNICAL INVESTIGATION
439 CHURCHILL AVE, OTTAWA, ON

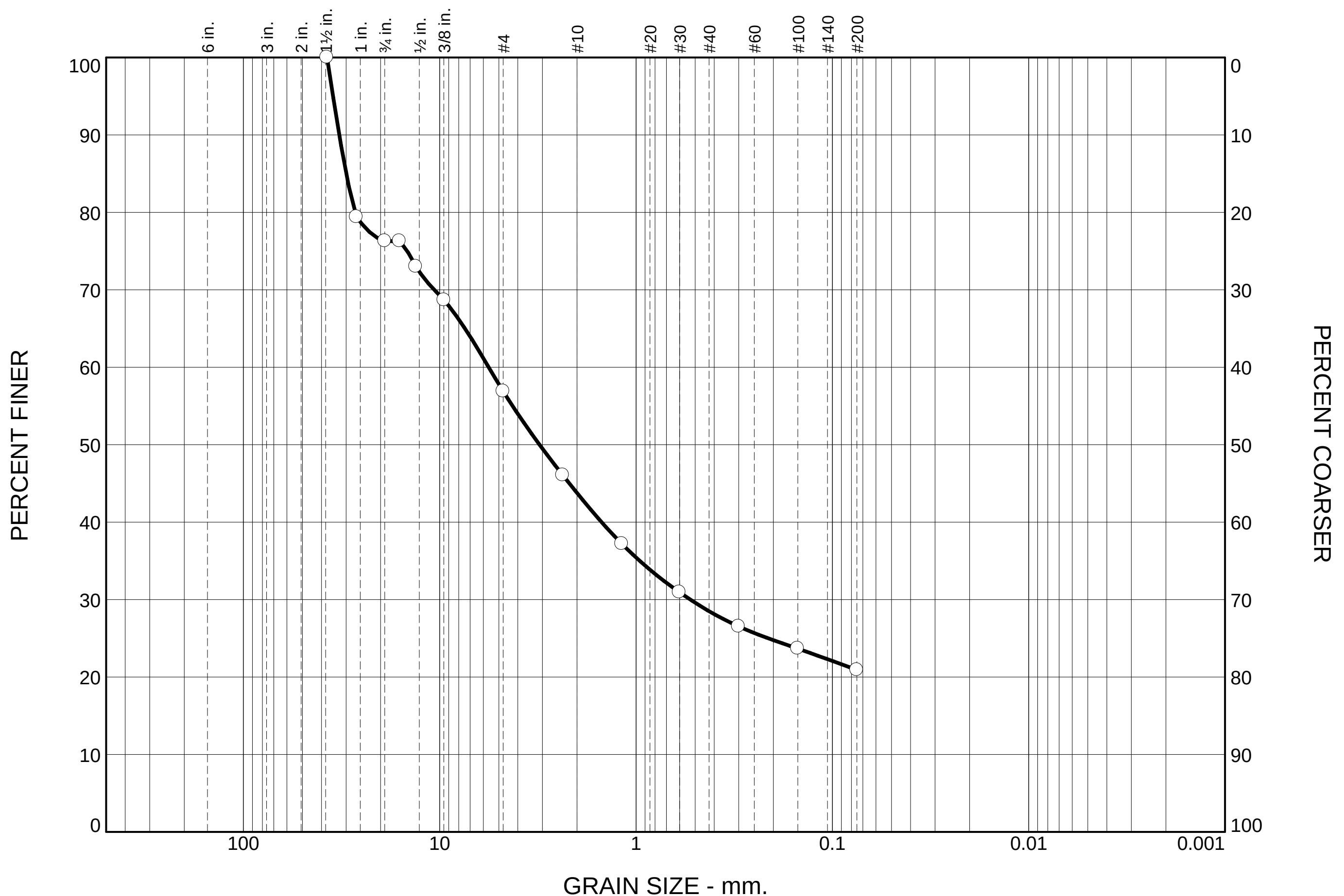
TITLE:
BOREHOLE LOCATIONS

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

PROJECT NO:CCO-21-3806		FIGURE:
Date	Apr., 26, 2021	2
GIS	EU	
Checked By	MA	

C:\Users\e.ungun\McIntosh Perry\GIS - Documents\Projects\2021\CCO\CCO-21-3806 Grepault Dev - SPA - 439 Churchill Ave\aprx\Geotech\CCO-21-3806_Geotech.aprx

Particle Size Distribution Report



	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	23.7	19.4	13.1	15.3	7.6	20.9	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	439 Churchill	21-03SS-3	5'-7'	Silty Gravel and Sand	

These results are for the exclusive use of the cliant for whom they were obtained.

McINTOSH PERRY

Client: Grepault Developments Ltd.

Project: 439 Churchill Ave.

Project No.: CCO-21-3806

Figure 3

Tested By: J.Hopwood-Jones **Checked By:** J.Hopwood-Jones

APPENDIX C
BOREHOLE LOGS

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
$\sigma_l, \sigma_2, \sigma_3$	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
$\Phi_{,}$	$-^o$	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
Υ_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
Υ_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
Υ	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
Υ_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
Υ_{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
Υ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

McINTOSH PERRY

BOREHOLE No 21-01

DATE: 14/04/2021 - 14/04/2021
PROJECT: CCO-21-3806
CLIENT: Grepault Developments Ltd.
ELEVATION: 78.03 m
LOCATION: 439 Churchill Ave, Ottawa, ON
COORDINATES: Lat: 45.390587335 , Lon: -75.752625493
DATUM: Geodetic
REMARK:
ORIGINATED BY: M.A.
COMPILED BY: M.A.
CHECKED BY: N.T.
REPORT DATE: 06/06/2021

ELEVATION - m DEPTH - m	SOIL PROFILE: DESCRIPTION	SAMPLES: TYPE AND NUMBER	RECOVERY	"N" or RQD	GROUNDWATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)
78.0	Natural ground surface					
0.0	Asphalt ~ 51 mm.					
78.0 0.1	Fill : Sand and gravel, some silt, black to dark brown, damp, compact.	SS-01	29	13 -		
77.3 0.7	Sand and gravel, some silt, light brown, compact to dense. Presence of cobbles and boulders.	SS-02	-	REF -		SPT spoon sampler refusal at (0.914 m) Auger refusal at (1.04 m)
77.0 1.0	Limestone	RC-03	100	- *41*		
		RC-04	91	- *60*		Rock core UCS = 187.9 MPa.
		RC-05	100	- *83*		Rock core UCS = 152.6 MPa.
73.5 4.5	**END OF BOREHOLE**					

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

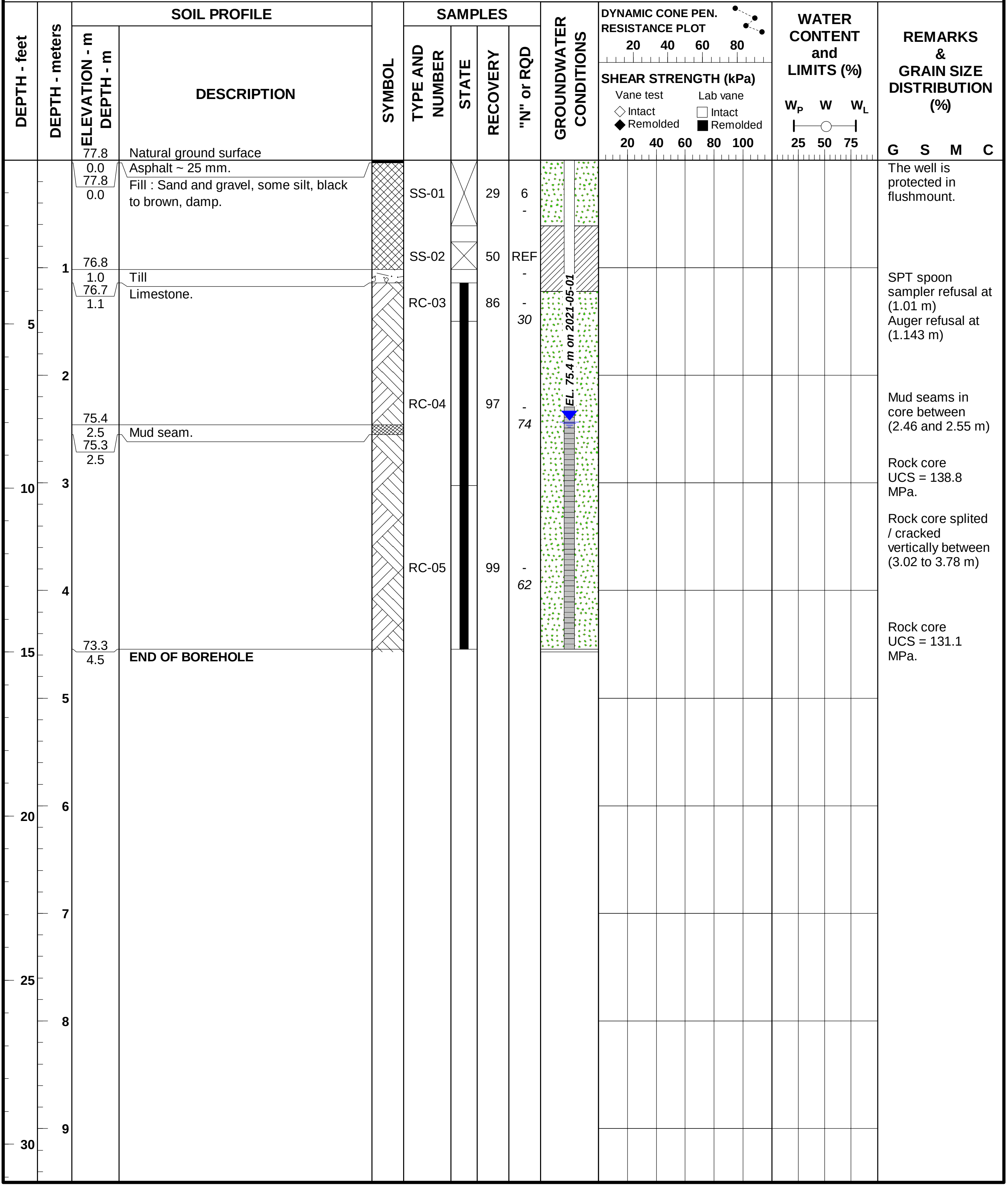
GRAIN SIZE: G S M C

McINTOSH PERRY

BOREHOLE No 21-02

Page 1 of 1

DATE:	14/04/2021 - 14/04/2021	**LOCATION:**	439 Churchill Ave, Ottawa, ON	**ORIGINATED BY:**	M.A.
PROJECT:	CCO-21-3806	**COORDINATES:**	Lat: 45.39067118 , Lon: -75.752483364	**COMPILED BY:**	M.A.
CLIENT:	Grepault Developments Ltd.	**DATUM:**	Geodetic	**CHECKED BY:**	N.T.
ELEVATION:	77.83 m	**REMARK:**		**REPORT DATE:**	06/06/2021



ELEVATION - m DEPTH - m	DESCRIPTION	TYPE AND NUMBER	RECOVERY	"N" or RQD	REMARKS
77.8	Natural ground surface				
0.0 77.8	Asphalt ~ 25 mm.				The well is protected in flushmount.
0.0	Fill : Sand and gravel, some silt, black to brown, damp.	SS-01	29	6 -	
		SS-02	50	REF -	
76.8 1.0	Till				SPT spoon sampler refusal at (1.01 m)
76.7 1.1	Limestone.	RC-03	86	- 30	Auger refusal at (1.143 m)
		RC-04	97	- 74	Mud seams in core between (2.46 and 2.55 m)
75.4 2.5	Mud seam.				
75.3 2.5					Rock core UCS = 138.8 MPa.
					Rock core splited / cracked vertically between (3.02 to 3.78 m)
		RC-05	99	- 62	Rock core UCS = 131.1 MPa.
73.3 4.5	**END OF BOREHOLE**				

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McINTOSH PERRY

BOREHOLE No 21-03

Page 1 of 1

DATE: 14/04/2021 - 14/04/2021
PROJECT: CCO-21-3806
CLIENT: Grepault Developments Ltd.
ELEVATION: 77.77 m

LOCATION: 439 Churchill Ave, Ottawa, ON
COORDINATES: Lat: 45.390710711 , Lon: -75.752524905
DATUM: Geodetic
REMARK:

ORIGINATED BY: M.A.
COMPILED BY: M.A.
CHECKED BY: N.T.
REPORT DATE: 06/06/2021

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

SOIL PROFILE: ELEVATION - m / DEPTH - m	SOIL PROFILE: DESCRIPTION	SAMPLES: TYPE AND NUMBER	SAMPLES: RECOVERY	SAMPLES: "N" or RQD	GROUNDWATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C
77.8	Natural ground surface					
0.0	Asphalt ~ 25 mm.					
77.7 0.0	Fill : Sand and gravel, some silt, black to brown, damp, compact.	SS-01	29	11 -		
76.9 0.8	Silty sand and gravel, brown to light brown, dry, compact to dense. Presence of cobbles and boulders.	SS-02	-	48 -		
		SS-03	29	36 -		42 37 - 21 -
75.5 2.3	Limestone with mud seams	RC-04	100	- *91*		Auger refusal at (2.29 m) Rock core UCS = 191.3 MPa.
74.9 2.9	Mud seam.					Mud seams in core between (2.88 and 2.97 m)
74.8 3.0		RC-05	100	- *87*		
73.3 4.5	Mud seam					A thick continuous mud seam between (4.47 and 4.56 m)
73.2 4.6	Limestone	RC-06	98	- *70*		
71.9 5.9	**END OF BOREHOLE**					

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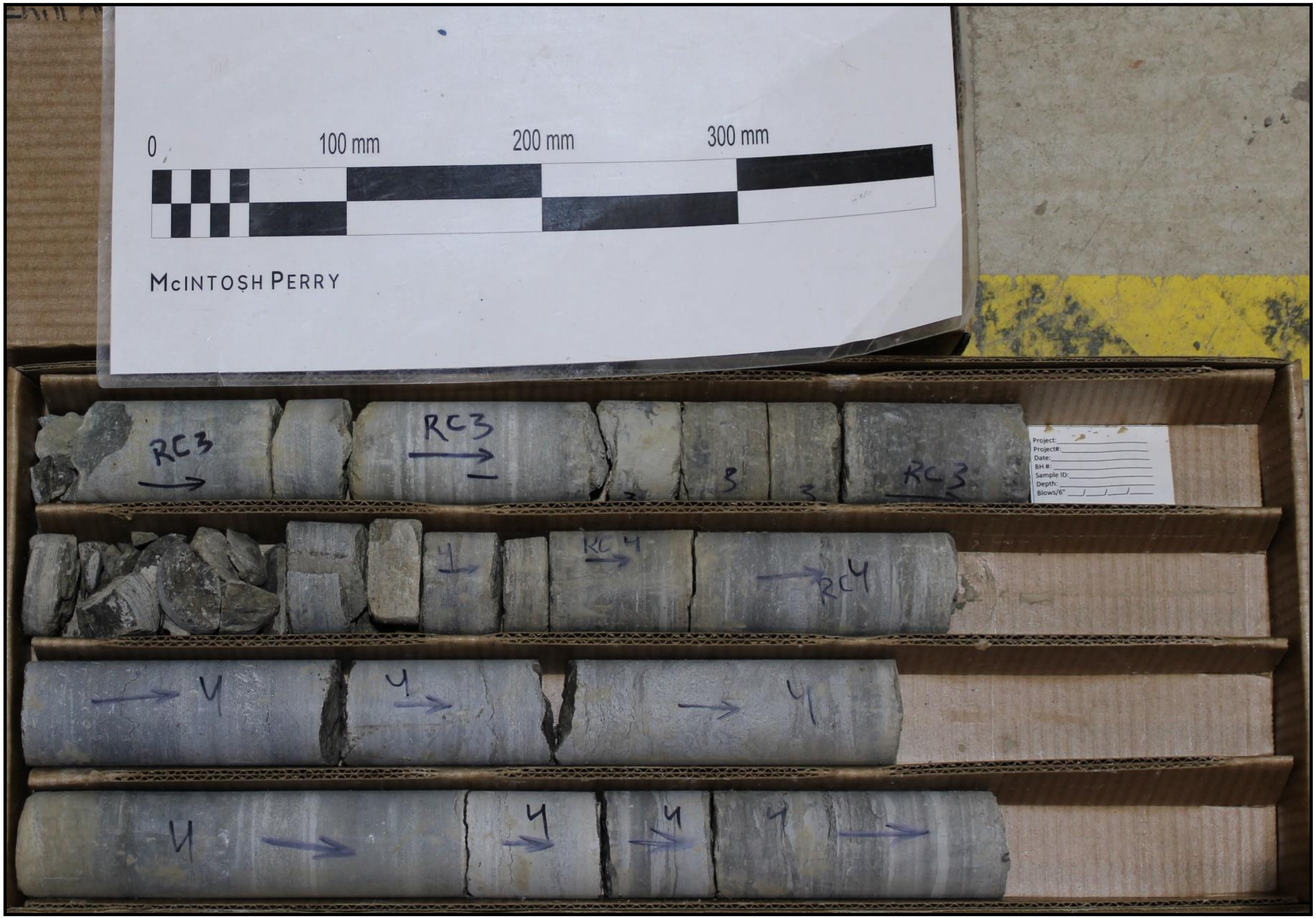
439 CHURCHILL AVE – GEOTECHNICAL REPORT

APPENDIX D
ROCK CORES

McINTOSH PERRY



McINTOSH PERRY

Client: Grepault Developments Ltd.

Project: Geotech. Investigation – Four-Storey Addition
439 Churchill Ave., Ottawa, ON.

Borehole: 21-01 **Sample:** RC – 03 and RC-04 **Depth:** 1.04 – 3.01 m

Project No.: CCO-21-3806

McINTOSH PERRY

Client: Grepault Developments Ltd.
Project: Geotech. Investigation – Four-Storey Addition
439 Churchill Ave., Ottawa, ON.
Borehole: 21-01 **Sample:** RC – 05 **Depth:** 3.01 – 4.5 m
Project No.: CCO-21-3806

McINTOSH PERRY

Client: Grepault Developments Ltd.

Project: Geotech. Investigation – Four-Storey Addition
439 Churchill Ave., Ottawa, ON.

Borehole: 21-02 **Sample:** RC – 03 and RC-04 **Depth:** 1.14 – 3.02 m

Project No.: CCO-21-3806

McINTOSH PERRY

Client: Grepault Developments Ltd.
Project: Geotech. Investigation – Four-Storey Addition
439 Churchill Ave., Ottawa, ON.
Borehole: 21-02 **Sample:** RC – 05 **Depth:** 3.02 – 4.54 m
Project No.: CCO-21-3806

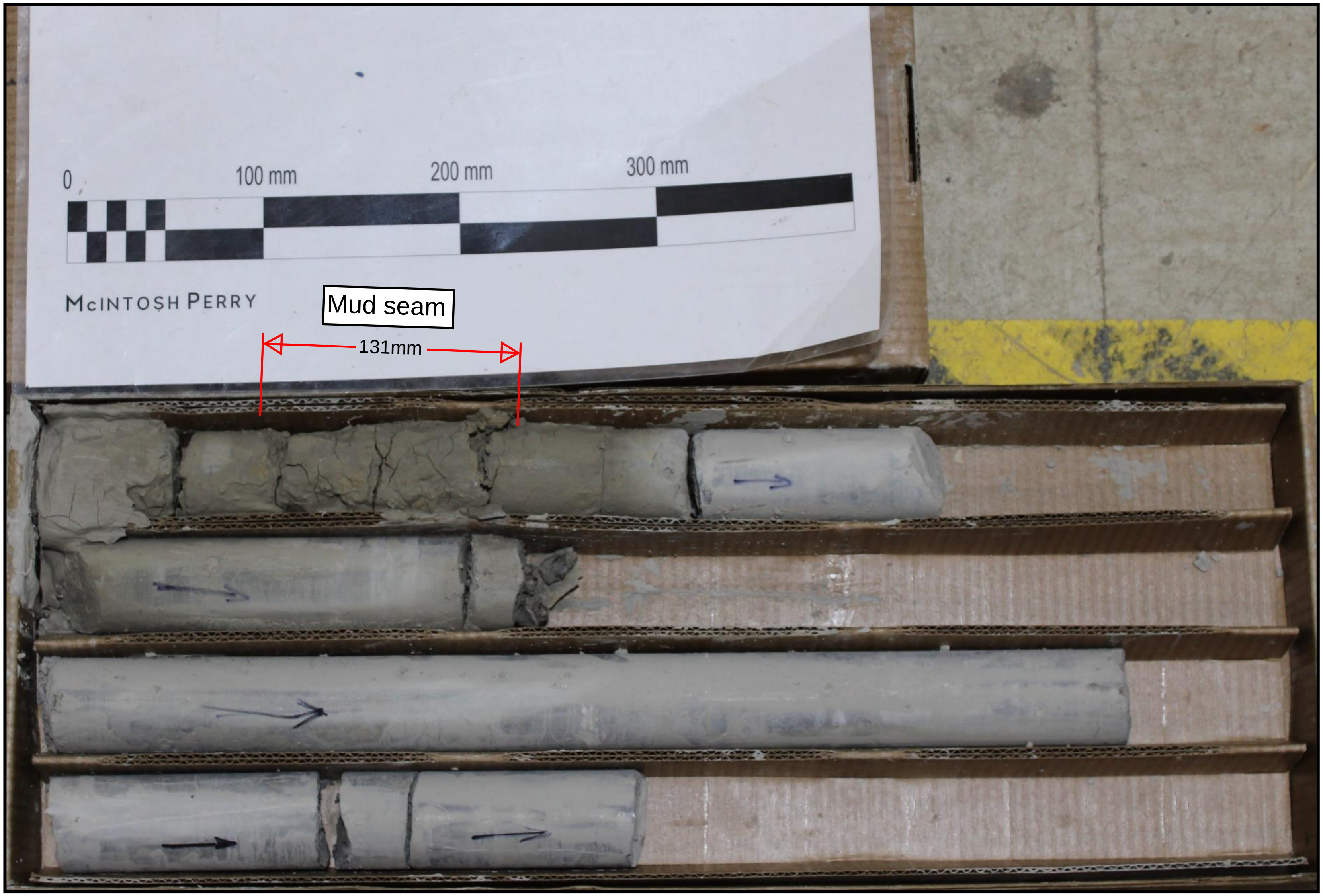
McINTOSH PERRY

Client: Grepault Developments Ltd.

Project: Geotech. Investigation – Four-Storey Addition
439 Churchill Ave., Ottawa, ON.

Borehole: 21-03 **Sample:** RC – 04 and RC-05 **Depth:** 2.29 – 4.36 m

Project No.: CCO-21-3806



McINTOSH PERRY

Client: Grepault Developments Ltd.

Project: Geotech. Investigation – Four-Storey Addition
439 Churchill Ave., Ottawa, ON.

Borehole: 21-03 **Sample:** RC – 06 **Depth:** 4.36 – 5.9 m

Project No.: CCO-21-3806

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO-21-3806	**Date Issued:**	2021-04-26
Lab No.:	OL-21023	**Report No.:**	OL-1
Project Name:	439 Churchill Ave.		

Core No.:	1	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-01	**Run:**	RC-4	**Depth (ft):**	9'2"-9'7"
Date Sampled:	2021-04-14	**Received:**	2021-04-14	**Tested:**	2021-04-26
Core No.:	2	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-01	**Run:**	RC-5	**Depth (ft):**	14'4"-14'9"
Date Sampled:	2021-04-14	**Received:**	2021-04-14	**Tested:**	2021-04-26
Core No.:		**Moisture Condition:**			
Borehole Location:		**Run:**		**Depth (ft):**	
Date Sampled:		**Received:**		**Tested:**	

Core No. :	1	2	
Diameter (mm)	47.3	47.2	
Thickness/Height (mm)	112.1	111.2	
Density (Kg/m^3)	2726	2639	
Compressive Strength (Mpa)	187.9	152.6	
Corr. Compressive Strength (Mpa)	N/A	N/A	
Description of Failure	Type 1	Type 1	

Remarks:

Reviewed By: **Date:** April 27,2021

Jason Hopwood-Jones
Laboratory Manager

MCINTOSH PERRY

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO-21-3806	**Date Issued:**	2021-04-26
Lab No.:	OL-21023	**Report No.:**	OL-2
Project Name:	439 Churchill Ave.		

Core No.:	3	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-02	**Run:**	RC-4	**Depth (ft):**	8'10"-9'3"
Date Sampled:	2021-04-14	**Received:**	2021-04-14	**Tested:**	2021-04-26
Core No.:	4	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-02	**Run:**	RC-5	**Depth (ft):**	14'5"-14'10"
Date Sampled:	2021-04-14	**Received:**	2021-04-14	**Tested:**	2021-04-26
Core No.:		**Moisture Condition:**			
Borehole Location:		**Run:**		**Depth (ft):**	
Date Sampled:		**Received:**		**Tested:**	

Core No. :	3	4	
Diameter (mm)	47.3	47.2	
Thickness/Height (mm)	111.4	112.8	
Density (Kg/m^3)	2721	2626	
Compressive Strength (Mpa)	138.8	131.1	
Corr. Compressive Strength (Mpa)	N/A	N/A	
Description of Failure	Type 3	Type 2	

Remarks:

Reviewed By: **Date:** April 27,2021

Jason Hopwood-Jones
Laboratory Manager

Unconfined Compressive Strength of Intact Rock Cores

ASTM D7012 Method C

Project No.:	CCO-21-3806	**Date Issued:**	2021-04-26
Lab No.:	OL-21023	**Report No.:**	OL-3
Project Name:	439 Churchill Ave.		

Core No.:	5	**Moisture Condition:**	As Received		
Borehole Location:	BH 21-03	**Run:**	RC-4	**Depth (ft):**	7'8"-8'1"
Date Sampled:	2021-04-14	**Received:**	2021-04-14	**Tested:**	2021-04-26
Core No.:		**Moisture Condition:**			
Borehole Location:		**Run:**		**Depth (ft):**	
Date Sampled:		**Received:**		**Tested:**	
Core No.:		**Moisture Condition:**			
Borehole Location:		**Run:**		**Depth (ft):**	
Date Sampled:		**Received:**		**Tested:**	

Core No. :	5		
Diameter (mm)	47.2		
Thickness/Height (mm)	110		
Density (Kg/m^3)	2722		
Compressive Strength (Mpa)	191.3		
Corr. Compressive Strength (Mpa)	N/A		
Description of Failure	Type 1		

Remarks:

Reviewed By: **Date:** April 27,2021

Jason Hopwood-Jones
Laboratory Manager

439 CHURCHILL AVE – GEOTECHNICAL REPORT

APPENDIX E
LAB RESULTS

Only selected pages from the third-party lab are included in this appendix

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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

Client PO:
Project: CCO 21-3806
Custody: 128751

Report Date: 30-Apr-2021
Order Date: 27-Apr-2021

Order #: 2118210

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

Paracel ID	Client ID
2118210-01	BH21-03 SS-2A

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis
Client: McIntosh Perry Consulting Eng. (Nepean)
Client PO:

Report Date: 30-Apr-2021
Order Date: 27-Apr-2021
Project Description: CCO 21-3806

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	29-Apr-21	29-Apr-21
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	27-Apr-21	28-Apr-21
Resistivity	EPA 120.1 - probe, water extraction	28-Apr-21	30-Apr-21
Solids, %	Gravimetric, calculation	28-Apr-21	28-Apr-21

Certificate of Analysis

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

Client PO:

Report Date: 30-Apr-2021

Order Date: 27-Apr-2021

Project Description: CCO 21-3806

	Client ID:	BH21-03 SS-2A	-	-	-
	Sample Date:	14-Apr-21 09:00	-	-	-
	Sample ID:	2118210-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.48	-	-	-
Resistivity	0.10 Ohm.m	14.2	-	-	-
Anions					
Chloride	5 ug/g dry	293	-	-	-
Sulphate	5 ug/g dry	82	-	-	-

Qualifier Notes:

None

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.

439 CHURCHILL AVE – GEOTECHNICAL REPORT

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: 45.391N 75.752W **User File Reference:** 439 Churchill Ave. 2021-04-29 19:40 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)	0.634	0.385	0.248	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)	0.323	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