200 MAPLE CREEK COURT – GEOTECHNICAL REPORT

Project No.: CP-18-0512

Prepared for:

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REVISED
January 2020

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GEOTECHNICAL INVESTIGATION and
FOUNDATION DESIGN RECOMMENDATION REPORT
200 Maple Creek Court, Carp, Ontario

1.0 INTRODUCTION

This report presents the factual findings obtained from a geotechnical investigation performed at the above-mentioned site, for the proposed construction of a commercial storage building Ottawa, Ontario. The field work was carried out on January 11, 2019 and comprised of five boreholes advanced to a maximum depth of 5.2 m below existing ground surface.

The purpose of the investigation was to explore the subsurface conditions at this site and to provide anticipated geotechnical conditions influencing the design and construction of the proposed building.

McIntosh Perry Consulting Engineers Ltd (McIntosh Perry) carried out the investigation at the request of NCM Hydrovac Services.

2.0 SITE DESCRIPTION

The property under considerations for proposed development is located at 200 Maple Creek Road, north of highway 417 in an industrial park in Carp, Ontario. Access to the site is via an existing gravel driveway located to the south of the property. At the time of investigation, the site was observed to be relatively flat, overlain by a layer of fill with debris scattered in various locations with some trees and vegetation.

It is understood that the proposed development will comprise of a 1012 m^2 commercial storage building with a small mezzanine office space and site development will include a septic bed, fueling station, parking lot and gravel driveway.

Site location is shown on Figure 1, included in Appendix B.

3.0 FIELD PROCEDURES

Staff of McIntosh Perry Consulting Engineers (McIntosh Perry) visited the site before the drilling investigation to mark out the proposed borehole locations and assess drill rig access. Utility clearance was carried out by USL-1 on behalf of McIntosh Perry. Public and private utility authorities were informed and all utility clearance documents were obtained before the commencement of drilling work.

The equipment used for drilling was owned and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario. Boreholes were advanced using hollow stem augers aided by a truck-mounted CME-850 drilling rig. Boreholes were advanced to a maximum depth of 5.2 m below the ground level. Soil samples were obtained at 0.75 m intervals of depth in boreholes using a 51 mm outside diameter split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. Boreholes were backfilled with auger cuttings.

All boreholes were restored to match the original surface. Borehole locations are shown on Figure 2, included in Appendix B.

4.0 LABORATORY TEST PROCEDURES

Laboratory tests were carried out on representative SPT samples recovered during the site investigation. Soil testing was carried out by McIntosh Perry Consulting Engineers. The laboratory tests to determine index properties were performed in accordance with American Society for Testing Materials (ASTM) test procedures. The following standards were followed to complete the laboratory testing;

ASTM C136 – Sieve analysis of Fine and Coarse Aggregates (LS-602)
ASTM D422 – Standard Test Method for Particle-Size Analysis of Soils (LS-702)
ASTM D2216 – Standard Test Method for Laboratory Determination of Water Content of Soil and Rock by Mass

Paracel Laboratories Ltd., in Ottawa carried out chemical tests on one representative soil sample to determine the soil and water corrosivity characteristics.

Laboratory test results are included in Appendix D.

The remaining soil samples recovered will be stored in McIntosh Perry's storage facility for a period of one month after submission of the final report. Samples will be disposed after this period of time unless otherwise requested in writing by the owners' representative.

5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

Based on published physiography maps of the area (Ontario Geological Survey) the site is located within the Ottawa Valley Clay Plains. Surficial geology maps of southern Ontario identify the property as on fine to coarse-textured deposits.

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 so that some of the uplifted blocks appear above the clay beds. The sediments themselves in the valley are deep silty clay. Although the clay deposits are grey in color like the limestones that underlies them in part, they are only mildly calcareous and likely derived from the more acidic rock of the Canadian Shield.

5.2 Subsurface Conditions

In general, the site stratigraphy encountered during the investigation consists of a relatively thin layer of fill overlaying till over probable bedrock, containing sand and varying portions of silt, gravel and clay. The soils encountered at this site can be summarized by the following two zones.

a) Fill
b) Till

The soils encountered during the course of the investigation, together with the field and laboratory test results are shown on the Record of Borehole sheets included in Appendix C. Description of the strata encountered are given below.

5.2.1 Fill

A 0.5 m to 0.8 m layer of fill was present at the top of all boreholes. The fill in boreholes BH19-01 through BH19-04 consisted of silty sand and some gravel with trace clay present in BH19-04. This fill was described as being brown, moist and compact. The fill in BH19-05 consisted of sand and gravel with trace silt described as grey and moist.

5.2.2 Till

Below the fill layer was a silty sand some gravel, to silty gravelly sand till, which was observed to be light brown to brown, moist to wet, compact to very dense. SPT 'N' values within this layer ranged from 21 blows/300 mm to refusal. Six representative samples of the till underwent sieve or hydrometer grain size analysis and were found to range from 11% gravel, 56% sand, 31% fines to 22% silt and 17% clay. Moisture contents within this layer were on average 9%.

5.3 Chemical Analysis

The chemical test results conducted by Paracel Laboratories in Ottawa, Ontario, to determine the resistivity, pH, sulphate and chloride content of representative soil sample are shown in Table 5-1 below:

Table 5-1: Soil Chemical Analysis Results

Borehole	Sample	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (Ohm-cm)
BH19-01	SS-02	1.5-2.1	7.94	0.0005	0.0023	7,610

5.4 Groundwater

At the time of drilling, groundwater was observed in open boreholes BH19-01, BH19-02 and BH19-05. A well was installed in BH19-02. Water level readings were taken on January 11, 2019 after drilling. Borehole/Well locations can be seen in Figure 2 in appendix B.

Table 5-2: Groundwater Levels

Borehole	BH Elev. (m)	Water Level Reading (m)	Groundwater Elev. (m)	Reading Method
BH19-01	113.90	1.90	112.0	Measured in open borehole following drilling
MW19-02	114.10	0.68	113.4	Measured in MW at end of day

BH19-05	114.10	0.91	113.2	Measured in open borehole following drilling

Groundwater levels may be expected to fluctuate due to seasonal changes.

6.0 DISCUSSIONS AND RECOMMENDATIONS

6.1 General

This section of the report provides recommendations for the design of the proposed storage building located at 200 Maple Creek Court in Carp, Ontario. The recommendations are based on interpretation of the factual information obtained from the boreholes advanced during the subsurface investigation. The discussions and recommendations presented are intended to provide sufficient information to the designer of the proposed building to select the suitable types of foundation to support the structure.

The comments made on the construction are intended to highlight aspects which could have impact or affect the detailed design of the building, for which special provisions may be required in the Contract Documents. Those who requiring information on construction aspects should make their own interpretation of the factual data presented in the report. Interpretation of the data presented may affect equipment selection, proposed construction methods, and scheduling of construction activities.

6.2 Project Design

6.2.1 Existing Site Condition

Detailed site condition is provided in Section 2. The topography was observed to be relatively flat, and covered in grass. The surrounding area consisted of industrial warehouses. The location of the site is shown on Figure 1 included in Appendix B.

6.2.2 Proposed Development

It is understood that the proposed development will be a single-story and will likely to be supported on conventional shallow footings.

6.3 Frost Protection

Based on applicable building codes, a minimum earth cover of 1.8 m, or the equivalent of thermal insulation, should be provided for all exterior footings to reduce the effects of frost action. Frost penetration depth is assumed reduced to 1.5 m for heated buildings if constructed properly, consistently heated during the cold season, and the heat flux is not blocked.

If adequate frost cover is not provided, the deficit of earth cover should be compensated by application of synthetic insulation material adequately projecting beyond foundation walls.

6.4 Site Classification for Seismic Site Response

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 6-1, shown below and in Appendix E;

Table 6-1: Selected Seismic Spectral Responses (10% in 50 Yrs)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.157	0.298	0.133	0.045	0.315

The site can be classified as a Site Class "D" based on the till consistency for the purposes of site-specific seismic response to earthquakes based on Table 4.1.8.4.A OBC 2012. A copy of this table is included in Appendix E.

6.5 Non-structural Slabs-on-Grade

All fill should be removed from the footprint of the building. Free-floating Slabs-on-grade (to form the finished floor) should be supported on minimum 200 mm of Granular A compacted to 100% SPMDD. The existing fill shall be removed and replaced with engineered fill up to underneath the slab bedding. The subgrade underneath the slab can be raised using Granular B type II or granular A needs to be compacted to minimum 96% SPMDD. The slab is expected to be subject to heavy industrial loads, proper construction of fill and bedding is vital for the slab to function under expected loads.

All subgrades should be approved and proof-rolled under the supervision of a geotechnical representative prior to placement of the engineered fill and Granular "A" bedding.

6.6 Shallow Foundations

Considering the order of structural loads expected at the foundation level, provision of conventional strip footings will be adequate. Footings are expected to be buried to resist overturning and sliding and also to provide protection against frost action.

If subgrade needs to be raised to underside of the footings, all granular material should be placed in horizontal lifts of uniform thickness of no more than 300 mm before compaction. It should be placed at appropriate moisture content and compacted to a 100% standard Proctor density. The requirements for fill material and compaction may be addressed with a note on the structural drawing for foundation or grading drawing and/or with a Non-Standard Special Provision (NSSP).

6.6.1 Bearing Capacity

Assuming spread and strip footings are constructed through excavating any existing fill and exposing the native subgrade, the following bearing capacity values can be used for structural design;

Factored beading pressure at Ultimate Limit State (ULS): 275 kPa

Serviceability Limit State (SLS): 100 kPa

It is expected the strip footing will be between 0.6 m and 2.0 m, if strip footings outside these dimensions are required, the authors of this report should be informed to verify the compatibility of the design.

6.7 Grade Raise

Since there are no 'fine-grain' soils observed on site, all settlements induced by grade raise is considered immediate or to be matured in a relatively short period of time. Therefore, there are no concerns with grade raise at this site. For grade raises thicker than 1 m, it is recommended to complete most of the grade raise, as much as practically possible, before construction of the footings.

6.8 Lateral Earth Pressure

Free draining material should be used as backfill material for foundation walls. If the 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 6-1: Backfill Material Properties

Borehole	Granular "A"	Granular "B"
Effective Internal Friction Angle, ϕ'	35°	30°
Unit Weight, $\gamma\ (kN/m^3)$	22.8	22.8

7.0 PAVEMENT STRUCTURE

No details are provided on the traffic loads but it is understood that the parking lot and surrounding paved area is to be used frequently by light to heavy weight vehicles on a daily basis. Pavement structure most likely to be placed on fill material overlaying till.

Subgrade should be proof rolled prior to placing sub-base. All top soil and soft and deleterious material shall be removed and subgrade should be proof rolled under the supervision of a geotechnical engineer or technician, prior to placing the pavement structure. Should grade raise be required, compacted Granular B Type II or Granular A should be placed as needed and compacted to 95% SPMDD prior to construction of pavement structure.

The proposed pavement structure for light vehicles parking area and access road is included in Table 7-1;

Table 7-1: "Light Duty" Pavement Structure

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

The light duty pavement structure is suitable for the pavement on parking stalls for ordinary passenger cars and small pick-up trucks.

A high-durability pavement structure is required for areas with heavy truck traffic, such as access roads or loading bays if required. The recommended high-durability structure is included in Table 7-2;

Table 7-2: Proposed "High-Durability" Pavement Structure

Material		Thickness (mm)
Surface	Superpave 12.5 mm, PG 58-34	50
Binder	Superpave 19.0 mm, PG 58-34	60
Base	OPSS Granular A	150
Sub-base	OPSS Granular B Type II	450

Should a section of the property remain unpaved as a gravel lot, the cross section in Table 7-3 can be used;

Table 7-3: Proposed Gravel Lot Structure

Material		Thickness (mm)
Base	OPSS Granular A	150
Sub-base	OPSS Granular B or SSM	450

The base and sub base materials, i.e., Granular A and Granular Type B II, shall be in accordance with OPSS 1010. All granular compaction to minimum 100% Standard Proctor Maximum Dry Density (SPMDD). Superpave grade to be 58-34 PGAC. Asphalt layers should be compacted to comply with OPSS 310. Traffic category shall be provided to the contractor prior to preparation of mix designs.

If parking spaces and access roads are to be used unpaved, there might be a need for periodic maintenance depending on the usage level.

8.0 CONSTRUCTION CONSIDERATIONS

Any topsoil and existing fill material of any kind, should be removed from the entire footprint of the building. If grade raise is required suitable fill material to conform to specifications of OPSS Granular criteria shall be used. The Structural Fill, if directly supporting the load of the structure, should be free from any recycled or deleterious material, it should not be placed in lifts thicker than 300 mm and should be compacted to 100% Standard Proctor Maximum Dry Density (SPMDD). Exiting fill, if clean and free from deleterious material, can be sampled in large quantities for gradation. If passes the requirements of OPSS 1010 for 'Granular' criteria, it can be used as a grade raise under the slab-on-grade only.

Based on water elevation readings, founding level may be below groundwater level. The till is expected to have a high percentage of sand, some gravel, as a result the permeability is expected to be high. Depending on the staging of the excavation, dewatering of open excavation may be in excess of 50,000 L/day. As a result, application for a Permit to Take Water (PTTW) is required. Groundwater elevation is expected to fluctuate seasonally.

The excavations are expected to be advanced through either fill and till deposits. The overburden excavation should be completed in accordance with Ontario Regulation (O.Reg.) 213/91 under the Occupational Health and Safety Act (OHSA) with specific reference to acceptable side slopes and stabilization requirements. The general stratigraphy outlined herein can be considered an OHSA Type 3 Soil. As per the regulation the excavation walls are exempted from requiring protection systems for 1H:1V side slopes excavated in a Type 3 soil. For excavations through multiple soil types, the side slope geometry is governed by the soil with the highest number designation.

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

Foundation walls should be backfilled with free-draining material such as OPSS Granular types A or B. The native till is not a suitable material for backfilling unless otherwise proved conforming to OPSS 1010 Granular criteria through bulk sampling. There is no basement proposed for this site, however it is a good practice to provide sub-drains with positive of drainage to the City sewer, ditches, or storm water collection.

Septic sampling and design were provided by others and this report does not intend to provide any factual information for septic design. Groundwater and hydraulic conductivity recommendations provided in this report, may be used by other engineering teams at their own discretion.

9.0 HYDRAULIC CONDUCTIVITY

It is understood the servicing report provides a percolation rate of 75 mm/hr. Based on our field investigation and laboratory testing the native soil is identified as Silty Sand. Portions of clay and gravel also encountered within the soil matrix. Six grain-size analysis were completed for this project. Sand percentage ranged between 51% and 60% with and average value of 56%. The range of expected values for hydraulic conductivity of a silty sand deposit like those encountered on site is 10^{-5} m/s to 10^{-6} m/s or as commonly accepted in the industry 50 to 350 mm/hr. The percolation rate used in the servicing report is closer to the lower boundary value and considered in conformance with the findings of the geotechnical investigation if this value is to be used for design of the proposed infiltration.

10.0 SITE SERVICES

At the subject site, the burial depth of water-bearing utility lines is typically 2.4 m below ground surface. If this depth is not achievable due to design restrictions, 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.

Utilities should be supported on minimum of 150 mm bedding of Granular A compacted to minimum 96% of SPMDD. Since the native subgrade contains fine grained soils, it is recommended to separate the subgrade from the bedding material by a layer of geotextile to prevent cross migration of materials. 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 intersecting structural elements. The engineer designing utilities shall ensure the proposed utility pipes can tolerate compaction loads.

11.0 CEMENT TYPE AND CORROSION POTENTIAL

A 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 element. Test results are presented in Tables 5-1.

The potential for sulphate attack on concrete structures is moderate. Type GU Portland cement is expected to be adequate to protect buried concrete elements in the subsurface conditions encountered.

The corrosion potential for buried steel elements was determined as ‘non-aggressive’.

12.0 CLOSURE

We trust this geotechnical investigation and foundation design report meets requirements of your project. The “Limitations of Report” presented in Appendix A are an integral part of this report. Please do not hesitate to contact the undersigned should you have any questions or concerns.

McIntosh Perry Consulting Engineers Ltd.

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

LICENSED PROFESSIONAL ENGINEER
N. AZIZI TAVAKKOLI
100150911
Jan 31/20
PROVINCE OF ONTARIO

13.0 REFERENCES

Canadian Geotechnical Society, “Canadian Foundation Engineering Manual”, 4th Edition, 2006.

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

Google Earth, Google, 2015.

NRCan 2015 Seismic Hazard Calculator

200 MAPLE CREEK

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.

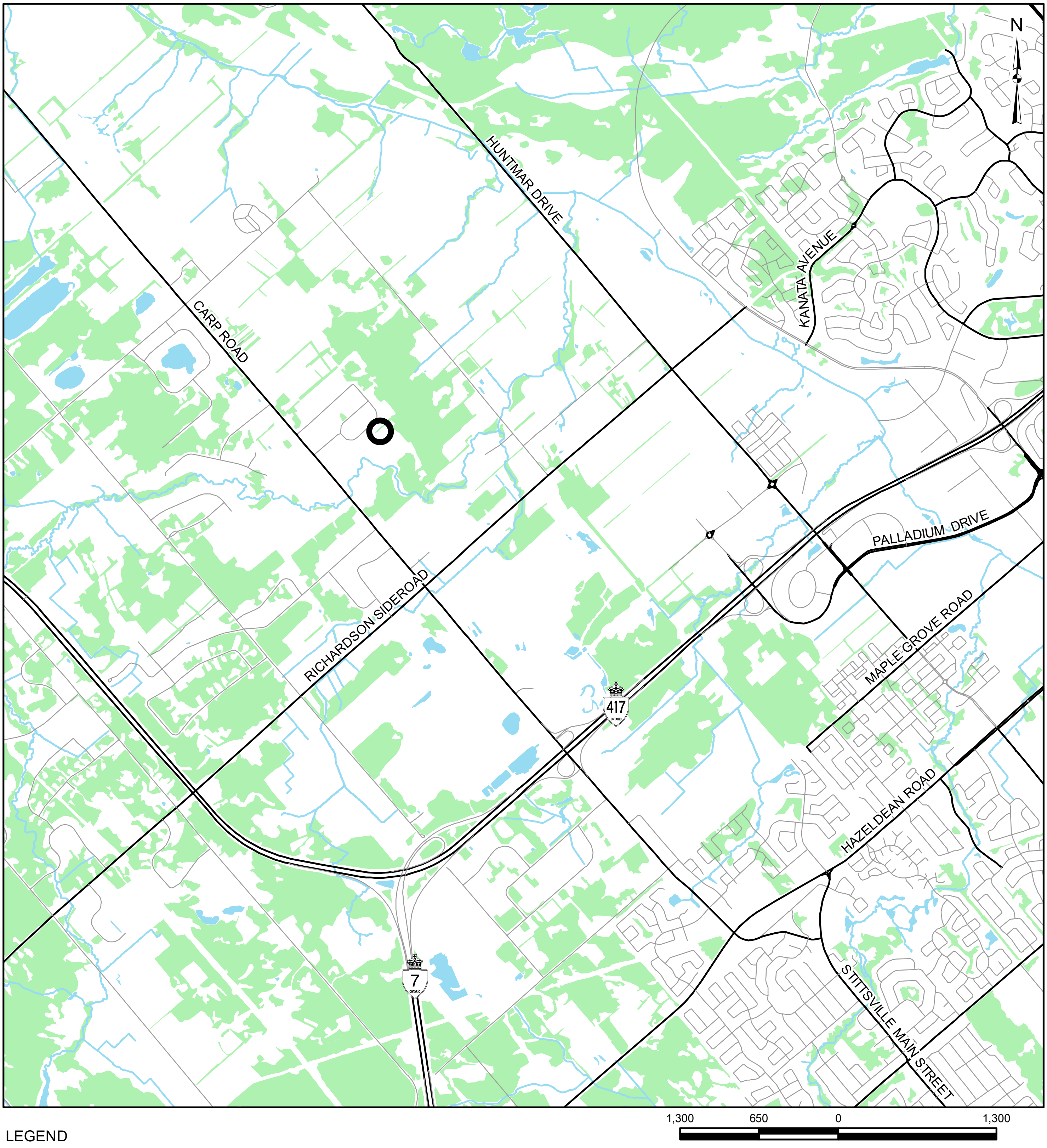
Under no circumstances shall the liability of McIntosh Perry for any claim in contract or in tort, related to the services provided and/or the content and recommendations in this report, exceed the extent that such liability is covered by such professional liability insurance from time to time in effect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

200 MAPLE CREEK

APPENDIX B
FIGURES

McINTOSH PERRY



LEGEND

● Site Location
🟩 Wooded Area
— Local Road
🟦 Waterbody
— Major Road
— Watercourse

CLIENT:
NCM HYDROVAC SERVI CES

PROJECT: GEOTECHNICAL INVESTIGATION
200 MAPLE CREEK COURT

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:CP-18-0512		FIGURE:
Date	Jan., 14, 2019	1
GIS	LC	
Checked By	MG	

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2019 and the City of Ottawa, 2019.

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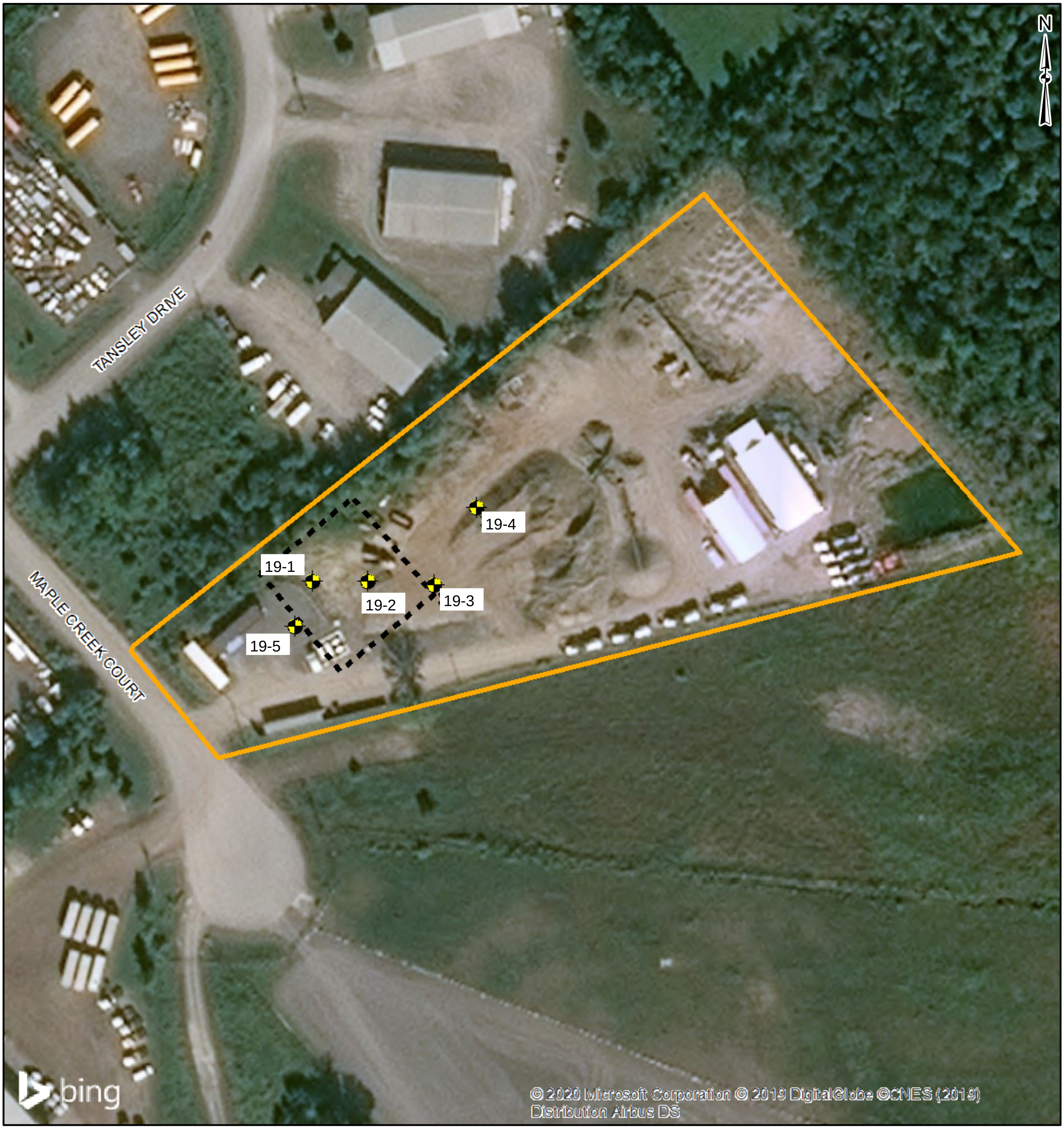

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0 10 20 40

Scale 1:1,500

Metres

LEGEND

- ⊕ Borehole Location
- ⬚ Proposed Building
- ▭ Site Location

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2020.
Note: Property and severance boundaries are approximate.

CLIENT: NCM HYDROVAC SERVICES

PROJECT: GEOTECHNICAL INVESTIGATION
200 MAPLE CREEK COURT

TITLE: BOREHOLE LOCATION

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PROJECT NO: CP-18-0512		FIGURE:
Date	Jan., 07, 2020	2
GIS	LC	
Checked By	DA	

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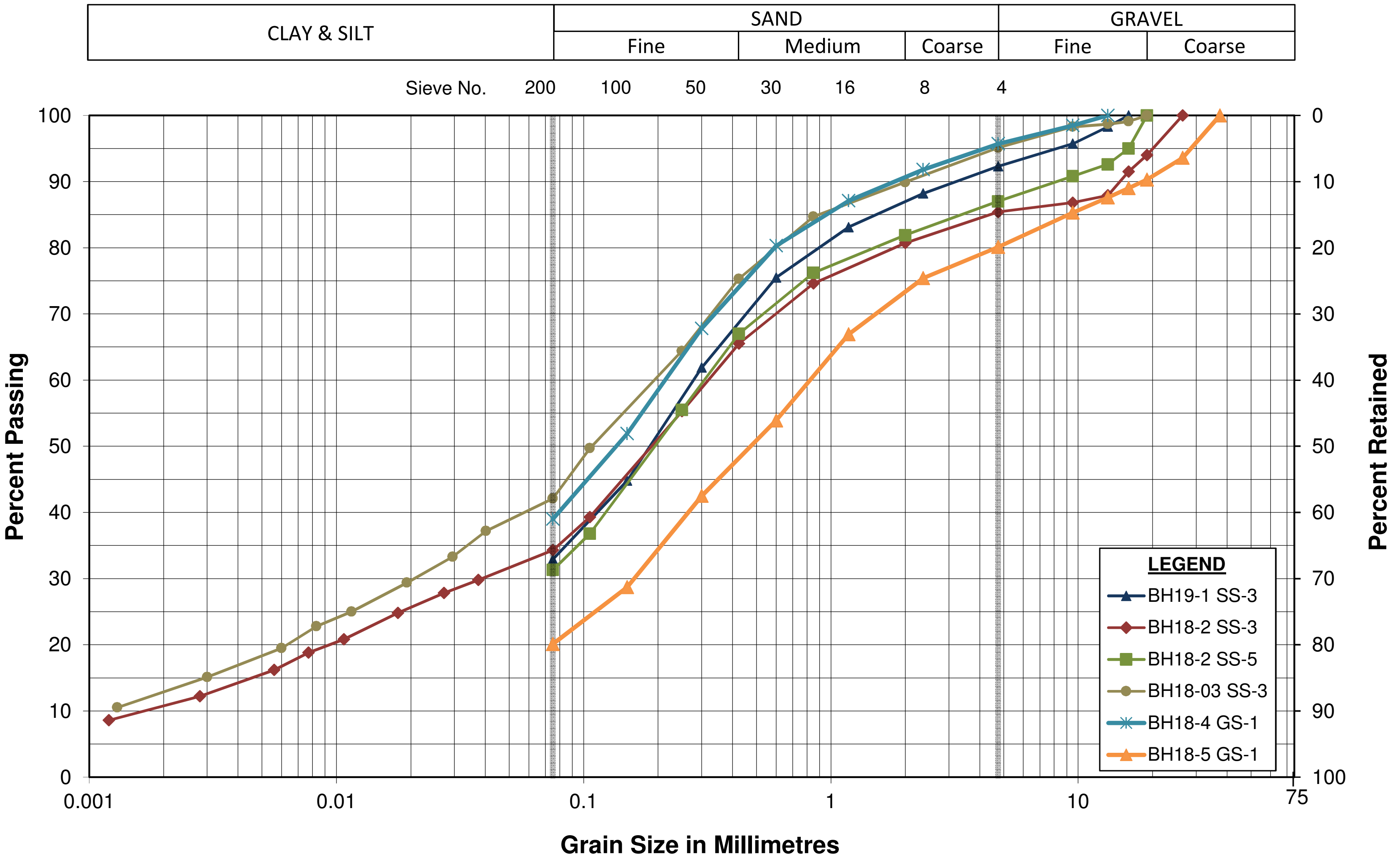
Unified Soil Classification System



McINTOSH PERRY

Grain Size Distribution

TILL

200 Maple Creek Ct

Figure No. 3

CM-18-0512

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

RECORD OF BOREHOLE No 19-1

DATE: 11/01/2019 -
PROJECT: CP-18-0512-MAPLECREE
CLIENT: NCM Services
ELEVATION: 113.9 m

LOCATION: 200 Maple Creek ()
COORDINATES: Lat: 45.304629479 , Lon: -75.977061366
DATUM: Geodetic
REMARK:

ORIGINATED BY: PH
COMPILED BY: MG
CHECKED BY: NT
REPORT DATE: 04/02/2019

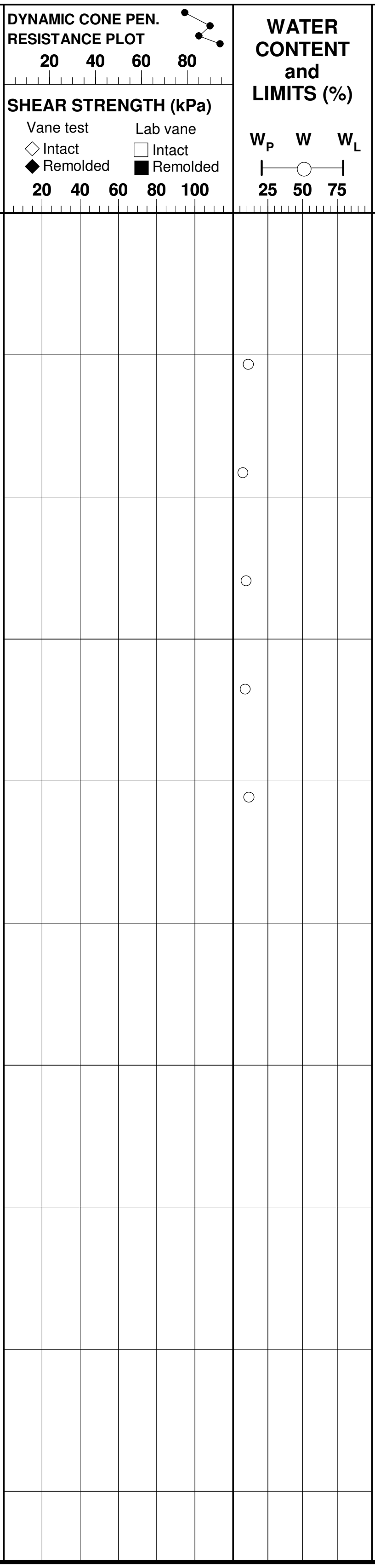
SOIL PROFILE

ELEVATION - m DEPTH - m	DESCRIPTION
113.9	Natural ground surface
0.0	Fill. Silty sand, some gravel, brown, moist.
113.2 0.8	Silty and gravelly sand, traces of clay, brown, moist to wet, dense. Presence of cobbles and boulders. (TILL)
110.9 3.0	Silty sand, traces of clay and gravel, grey, wet, compact. Presence of cobbles and boulders. (TILL)
110.1 3.8	Silty sand, traces of clay and gravel, grey, moist, dense. Presence of cobbles and boulders. (TILL)
109.2 4.7	**END OF BOREHOLE** **Auger refusal on probable bedrock.**

SAMPLES

TYPE AND NUMBER	RECOVERY	"N" or RQD	REMARKS & GRAIN SIZE DISTRIBUTION (%) G	S	M	C
SS-01	83	36				
SS-02	67	29				
SS-03	58	25	8	59		33
SS-04	67	21				
SS-05	100	38				
SS-06	17	REF				

GROUNDWATER CONDITIONS: 1.9 m



McINTOSH PERRY

RECORD OF BOREHOLE No 19-2

DATE: 11/01/2019 -
PROJECT: CP-18-0512-MAPLECREE
CLIENT: NCM Services
ELEVATION: 114.1 m

LOCATION: 200 Maple Creek ()
COORDINATES: Lat: 45.304619564 , Lon: -75.976863068
DATUM: Geodetic
REMARK:

ORIGINATED BY: PH
COMPILED BY: MG
CHECKED BY: NT
REPORT DATE: 04/02/2019

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	DESCRIPTION
114.1	Natural ground surface
0.0	Fill. Silty sand, some gravel, brown, moist, compact.
113.4 0.8	Silty and gravelly sand, traces of clay, grey, moist to wet, compact. Presence of cobbles and boulders. (TILL)
112.6 1.5	Silty sand, some gravel, traces of clay, grey, moist, compact. Presence of cobbles and boulders. (TILL)
111.1 3.0	Silty sand, some gravel, traces of clay, grey, moist, very dense. Presence of cobbles and boulders. (TILL) - Silty sand to sandy silt
109.0 5.2	**END OF BOREHOLE** **Auger refusal on probable bedrock.**

SAMPLES

TYPE AND NUMBER	RECOVERY	"N" or RQD	G	S	M	C
SS-01	58	21				
SS-02	71	18				
SS-03	79	27	15	51	19	16
SS-04	42	REF				
SS-05	92	67	13	56	31	
SS-06	92	53				
SS-07	100	REF				

GROUNDWATER CONDITIONS: 0.68 m

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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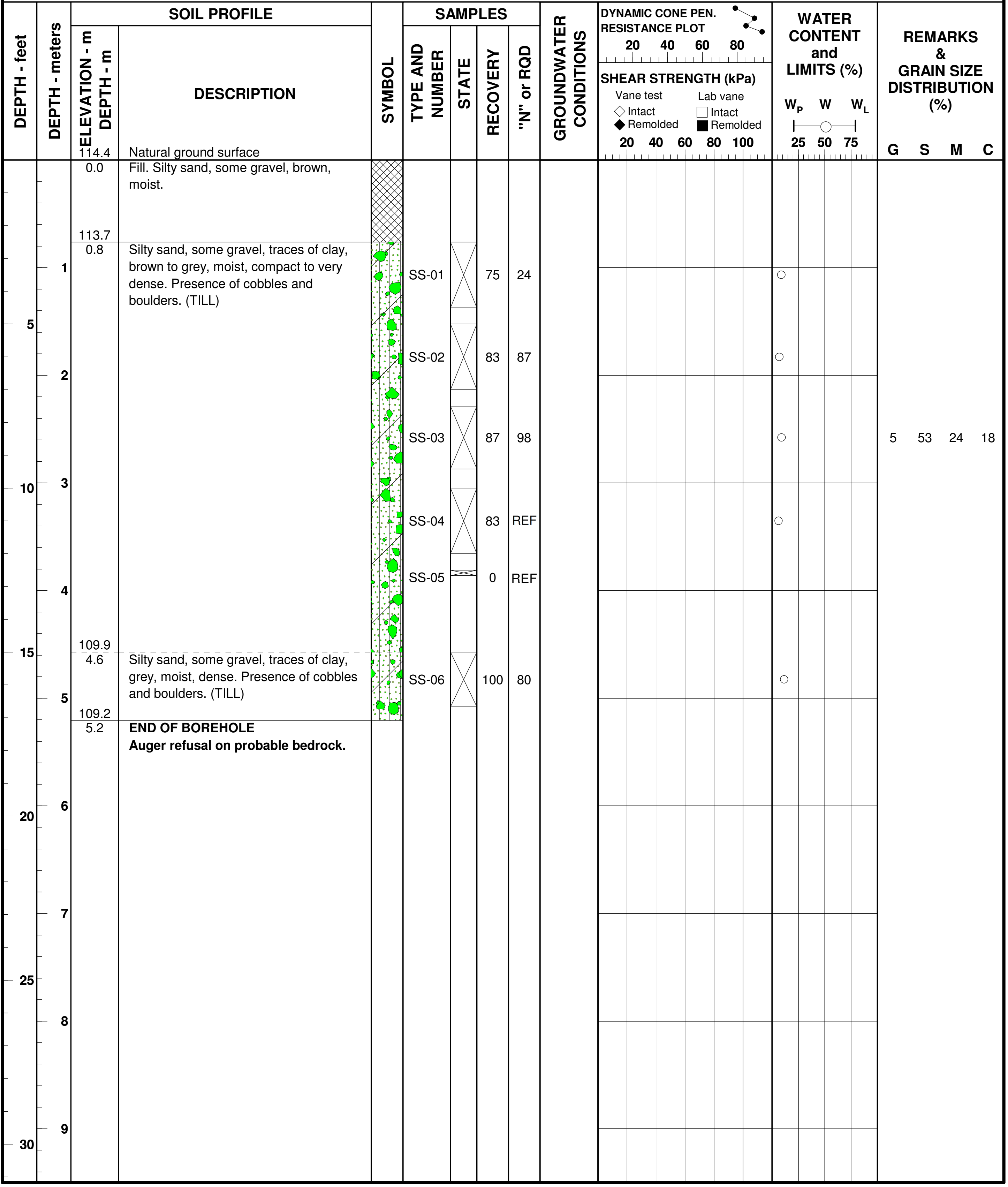
RECORD OF BOREHOLE No 19-3

DATE: 11/01/2019 -
LOCATION: 200 Maple Creek ()
ORIGINATED BY: PH
PROJECT: CP-18-0512-MAPLECREE
COORDINATES: Lat: 45.30460204 , Lon: -75.976633979
COMPILED BY: MG
CLIENT: NCM Services
DATUM: Geodetic
CHECKED BY: NT
ELEVATION: 114.4 m
REMARK: No water observed in open borehole.
REPORT DATE: 04/02/2019



SOIL PROFILE

ELEVATION - m DEPTH - m	DESCRIPTION
114.4	Natural ground surface
0.0	Fill. Silty sand, some gravel, brown, moist.
113.7 0.8	Silty sand, some gravel, traces of clay, brown to grey, moist, compact to very dense. Presence of cobbles and boulders. (TILL)
109.9 4.6	Silty sand, some gravel, traces of clay, grey, moist, dense. Presence of cobbles and boulders. (TILL)
109.2 5.2	**END OF BOREHOLE** **Auger refusal on probable bedrock.**

SAMPLES

TYPE AND NUMBER	RECOVERY	"N" or RQD	REMARKS & GRAIN SIZE DISTRIBUTION (%) G	S	M	C
SS-01	75	24				
SS-02	83	87				
SS-03	87	98	5	53	24	18
SS-04	83	REF				
SS-05	0	REF				
SS-06	100	80				

McINTOSH PERRY

RECORD OF BOREHOLE No 19-4

DATE: 11/01/2019 -
PROJECT: CP-18-0512-MAPLECREE
CLIENT: NCM Services
ELEVATION: 114.9 m

LOCATION: 200 Maple Creek ()
COORDINATES: Lat: 45.304795722 , Lon: -75.976491676
DATUM: Geodetic
REMARK: No water observed in open borehole.

ORIGINATED BY: PH
COMPILED BY: MG
CHECKED BY: NT
REPORT DATE: 04/02/2019

DEPTH - feet	DEPTH - meters	SOIL PROFILE ELEVATION - m DEPTH - m	SOIL PROFILE DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	SAMPLES STATE	SAMPLES RECOVERY	SAMPLES "N" or RQD	GROUNDWATER CONDITIONS	DYNAMIC CONE PEN. RESISTANCE PLOT 20 40 60 80 SHEAR STRENGTH (kPa) Vane test ◇ Intact ◆ Remolded Lab vane ☐ Intact ■ Remolded 20 40 60 80 100	WATER CONTENT and LIMITS (%) W_P W W_L 25 50 75	REMARKS & GRAIN SIZE DISTRIBUTION (%) G	S	M	C
		114.9	Natural ground surface												
		0.0	Fill. Silty sand, some gravel, trace clay, brown, moist.	▨											
		114.5 0.5	Silty sand, traces of clay and gravel, grey, moist. Presence of cobbles. (TILL)												
	1				GS-01	☒					○	4	57	39	
		113.4 1.5	**END OF BOREHOLE**												
5															
	2														
	3														
10															
	4														
15															
	5														
	6														
20															
	7														
25															
	8														
	9														
30															

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McINTOSH PERRY

RECORD OF BOREHOLE No 19-5

DATE: 11/01/2019 -
PROJECT: CP-18-0512-MAPLECREE
CLIENT: NCM Services
ELEVATION: 114.1 m

LOCATION: 200 Maple Creek ()
COORDINATES: Lat: 45.304494619 , Lon: -75.977138925
DATUM: Geodetic
REMARK:

ORIGINATED BY: PH
COMPILED BY: MG
CHECKED BY: NT
REPORT DATE: 12/02/2019

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

DEPTH - feet	DEPTH - meters	ELEVATION - m DEPTH - m	SOIL PROFILE DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	STATE	RECOVERY	"N" or RQD	GROUNDWATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) G	S	M	C
		114.1	Natural ground surface										
		0.0	Fill. Sand and gravel, trace silt, grey, moist.	▨									
		113.6 0.5	Silty sand, some gravel, brown, moist to wet.						0.91 m				
	1				GS-01	☒				20	60	20	
5		112.6 1.5	**END OF BOREHOLE**										
	2												
	3												
10													
	4												
15													
	5												
	6												
20													
	7												
25													
	8												
	9												
30													

APPENDIX D
LAB RESULTS

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Certificate of Analysis

McIntosh Perry Consulting Eng. (Carp)

215 Menton Place
Nepean, ON K2H 9C1
Attn: Mary Ellen Gleeson

Client PO: Maple Creek
Project: CP-18-0512
Custody: 120053

Report Date: 21-Jan-2019
Order Date: 15-Jan-2019

Order #: 1903252

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

Paracel ID	Client ID
1903252-01	BH18-1 SS-2

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	21-Jan-19	21-Jan-19
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	17-Jan-19	18-Jan-19
Resistivity	EPA 120.1 - probe, water extraction	21-Jan-19	21-Jan-19
Solids, %	Gravimetric, calculation	17-Jan-19	17-Jan-19

Certificate of Analysis

Client: McIntosh Perry Consulting Eng. (Carp)

Client PO: Maple Creek

Report Date: 21-Jan-2019

Order Date: 15-Jan-2019

Project Description: CP-18-0512

	Client ID:	BH18-1 SS-2	-	-	-
	Sample Date:	01/11/2019 09:00	-	-	-
	Sample ID:	1903252-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.94	-	-	-
Resistivity	0.10 Ohm.m	76.1	-	-	-
Anions					
Chloride	5 ug/g dry	23	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

APPENDIX E
SEISMIC HAZARD CALCULATION

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

Requested by: , January 22, 2019

Site Coordinates: 45.3046 North 75.9768 West

User File Reference:

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

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.616	**0.298**	**0.133**	**0.045**	**0.315**

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. 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 calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.085	0.239	0.375
Sa(0.5)	0.041	0.118	0.180
Sa(1.0)	0.017	0.054	0.085
Sa(2.0)	0.0060	0.017	0.027
PGA	0.036	0.118	0.194

References

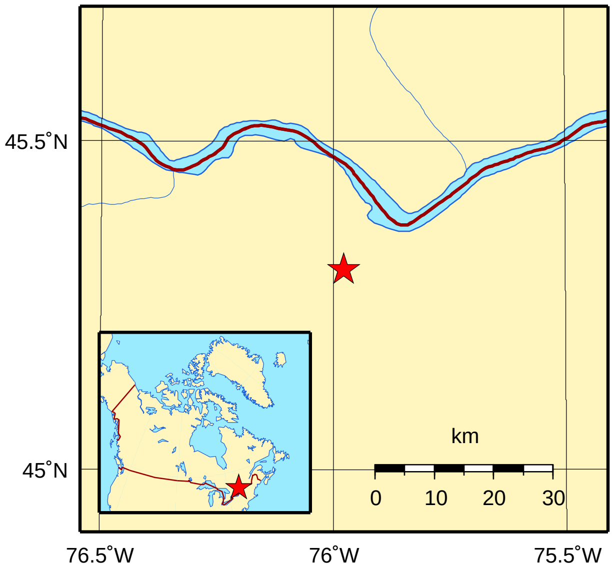
National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

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

Aussi disponible en français



Natural Resources Canada

Ressources naturelles Canada

Canada

(3) If average shear wave velocity, $\overline{V}_S$, is not known, Site Class shall be determined from energy-corrected Average Standard Penetration Resistance, $\overline{N}_{60}$, or from *soil* average undrained shear strength, s_u, as noted in Table 4.1.8.4.A., $\overline{N}_{60}$ and s_u being calculated based on rational analysis.

(4) Acceleration- and velocity-based site coefficients, F_a and F_v, shall conform to Tables 4.1.8.4.B. and 4.1.8.4.C. using linear interpolation for intermediate values of $S_a(0.2)$ and $S_a(1.0)$.

(5) Site-specific evaluation is required to determine F_a and F_v for Site Class F.

(6) For structures with a fundamental period of vibration equal to or less than 0.5 s that are built on liquefiable *soils,* Site Class and the corresponding values of F_a and F_v may be determined as described in Tables 4.1.8.4.A., 4.1.8.4.B., and 4.1.8.4.C. by assuming that the *soils* are not liquefiable.

(7) The design spectral acceleration values of S(T) shall be determined as follows, using linear interpolation for intermediate values of T:

$$
\begin{aligned}
S(T) &= FaSa(0.2) \text{ for } T \le 0.2 \text{ s} \\
&= F_v S_a(0.5) \text{ or } F_a S_a(0.2)\text{, whichever is smaller for } T = 0.5 \text{ s} \\
&= F_v S_a(1.0) \text{ for } T = 1.0 \text{ s} \\
&= F_v S_a(2.0) \text{ for } T = 2.0 \text{ s} \\
&= F_v S_a(2.0)/2 \text{ for } T \ge 4.0 \text{ s}
\end{aligned}
$$

Table 4.1.8.4.A.
Site Classification for Seismic Site Response

Forming Part of Sentences 4.1.8.4.(1) to (3)

Item	**Column 1**	**Column 2**	**Column 3**	**Column 4**	**Column 5**
	Site Class	**Ground Profile Name**	**Average Properties in Top 30 m**		
			Average Shear Wave Velocity, $\overline{V}_S$ (m/s)	**Average Standard Penetration Resistance $\overline{N}_{60}$**	***Soil* Undrained Shear Strength, s_u**
1.	A	Hard *rock*[1] [2]	$\overline{V}_S > 1500$	N/A	N/A
2.	B	*Rock*[1]	$760 < \overline{V}_S \le 1500$	N/A	N/A
3.	C	Very dense *soil* and soft *rock*	$360 < \overline{V}_S < 760$	$\overline{N}_{60} > 50$	$s_u > 100$kPa
4.	D	Stiff *soil*	$180 < \overline{V}_S < 360$	$15 \le \overline{N}_{60} \le 50$	50 kPa $< s_u \le$ 100 kPa
5.	E	Soft *soil*	$\overline{V}_S < 180$	$\overline{N}_{60} < 15$	$s_u <$ 50 kPa
			Any profile with more than 3 m of *soil* with the following characteristics: • plasticity index: PI>20 • moisture content w ≥40%, and • undrained shear strength: $s_u <$ 25 kPa		
6.	F	Other *soils*[3]	Site-specific evaluation required		

Notes to Table 4.1.8.4.A.:

[1] Site Classes A and B, hard *rock* and *rock*, are not to be used if there is more than 3 m of softer materials between the *rock* and the underside of footing or mat *foundations*. The appropriate Site Class for such cases is determined on the basis of the average properties of the total thickness of the softer materials.

[2] If $\overline{V}_S$ has been measured in-situ, the F_a and F_v values derived from Tables 4.1.8.4.B. and 4.1.8.4.C. may be multiplied by $(1500/\overline{V}_S)^{1/2}$.

[3] Other *soils* include:

(a) liquefiable *soils*, quick and highly sensitive clays, collapsible weakly cemented *soils*, and other *soils* susceptible to failure or collapse under seismic loading,

(b) peat and/or highly organic clays greater than 3 m in thickness,

(c) highly plastic clays (PI > 75) more than 8 m thick, and

(d) soft to medium stiff clays more than 30 m thick.