6688 FRANKTOWN – GEOTECHNICAL REPORT

Project No.: CP-17-0503

Prepared for:

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July 2018

McINTOSH PERRY

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APPENDICES

GEOTECHNICAL INVESTIGATION and
FOUNDATION DESIGN RECOMMENDATION REPORT
6688 Franktown Road, 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 construction of a prayer facility complex in Ottawa, Ontario. The field work was carried out on May 25, 2018 and comprised of three boreholes advanced to a maximum depth of 7.9 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 BING Professional Engineering Inc.

2.0 SITE DESCRIPTION

The property under considerations for proposed development is located at 6688 Franktown Road, southwest of the Village of Richmond located within Ottawa, Ontario. The property is located in a rural area with heavy vegetation prior to site clearing. Access to the site is granted via a gravel access road leading from the South side of Franktown Road extending approximately 200 m into the property. At the time of the investigation, the site was observed to be relatively flat, overlain by black peat with brush piles in various locations. Ponding to the northeast of the gravel access road, as well as ponding in logger skidder wheel ruts were indicative of a shallow water table.

It is understood that the proposed development will comprise of the following;

- The main prayer facility building will be one storey above ground level. One portion of the building at the south side is proposed to have a basement. The other 3 sides around the courtyard at the north portion are proposed as one storey building without basement. This building is designed for total area of approximately 2665 m²;
- A two-storey hexagonal building beside the main prayer facility. This building is approximately 635 m² at the base. The main building and the hexagonal building will be connected by an elevated covered link;
- A one storey building without basement proposed at the northeast of the property with approximately 350 m² footprint.

Location of the property 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-55 drilling rig. Boreholes were advanced to a maximum depth of 7.9 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 and rock cores recovered during the site investigation. Soil testing was carried out by McIntosh Perry Consulting Engineers and Rock Core testing was carried out by LRL Associates Ltd., on behalf of McIntosh Perry. The laboratory tests to determine index properties were performed in accordance with American Society for Testing Materials (ASTM) test procedures. 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 coarse-textured glaciomarine 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 peat, sand with trace clay and silt, sand containing trace amounts of silt, clay and gravel and limestone bedrock. The soils encountered at this site can be summarized by the following four zones.

a) Peat
b) Loose to compact sand trace clay and silt
c) Compact to dense sand, trace clay, silt and gravel
d) Limestone bedrock

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 Peat

A 0.2 m to 0.3 m layer of peat was present at the top of all boreholes. Silty sand was present in this layer in boreholes BH18-01 and BH18-02.

5.2.2 Loose to Compact Sand, Trace Clay and Silt

From a depth of approximately 0.2 m to 0.3 m there was a layer of sand containing clay and silt. This layer, extending to a depth ranging from 3.4 m to 5.0 m below ground surface, was described as light brown to brown, moist to wet, very loose to compact. SPT 'N' values within this layer ranged from 0 to 15 blows/ 300 mm. Two representative samples of the sand underwent 'hydrometer grain size analysis' and were found to contain on average 0 % gravel, 94 % sand, 4 % silt and 1 % clay. Moisture contents within this layer were on average 25 %.

5.2.1 Compact to Dense Sand, Trace Clay, Silt and Gravel

Underlying the above mentioned layer was a layer of Sand, containing trace amount of silt, clay and gravel. This material was generally described as light grey to grey wet, and compact to dense. The material extended to depths between 4.6 m and 5.7 m below ground surface. SPT 'N' values within this layer ranged from 15 to 58 blows/ 300 mm. A representative sample of this material underwent 'hydrometer grain size analysis' and was found to contain 5 % gravel, 86 % sand, 8 % sand and 1 % clay. A representative sample tested for natural water content indicated moisture content to be approximately 14 %.

5.2.2 Limestone Bedrock

Found at the bottom of all boreholes was limestone bedrock. This rock was cored in boreholes BH18-02 and BH18-01. A representative sample underwent Uniaxial Compressive Strength testing, resulting in a strength of 143 MPa with a predominantly columnar failure with a well formed cone on one end.

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)
BH18-02	SS-02	0.8 – 1.4	5.88	0.0020	0.0006	2,850

5.4 Groundwater

At the time of drilling groundwater was observed in all open boreholes at a depth of 0.3 m below ground surface. Water level readings of the wells were taken on June 15, 2018, water levels were as shown below in Table 5-2.

Table 5-2: Groundwater Levels

Borehole	BH Elev. (m)	Water Level Reading (m)	Groundwater Elev. (m)
BH18-01	100.900	1.532	99.368
BH18-02	100.680	1.327	99.353
BH18-03	100.960	1.115	99.845

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 three proposed buildings. Detailed description of structures is provided in Section 2.

The recommendations herein provided 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 property is predominately flat and it was recently cleared of heavy brush and is overlain by peat. The surrounding area consisted of heavy bush and farm land. The location of the site is shown on Figure 1 included in Appendix B.

6.2.2 Proposed Foundation Systems

Based on previous discussions, it is understood the following foundations systems will be used for the footing design;

- Conventional spread and strip footings for the southern portion of the main prayer facility. A basement is proposed for this portion of the building. The other 3 sides around the courtyard at the north portion are proposed as one storey building without basement. This portion is proposed on raft footings to the same width of the building and to be installed at the surface;
- A two-storey hexagonal building beside the main prayer facility which the two buildings will be connected by an elevated hallway. This building will be installed on piers or caissons bearing on rock;
- A one storey building without basement proposed at the northeast of the property is proposed to be supported on isolated spread or strip footings.

6.3 Frost Protection

Based on applicable building codes, frost penetration depth is approximated to 1.8 m for the geographical region of this site. A minimum earth cover of 1.8 m for unheated buildings (or 1.5 m for heated buildings), or the thermal equivalent of insulation, should be provided for all exterior footings to reduce the effects of frost action. Manufacturers' specifications shall be consulted for insulation properties and thicknesses.

6.4 Site Classification for Seismic Site Response

Selected spectral responses in the general vicinity of the site for 10% chance of exceedance in 50 years (475 years return period) are as indicated in Table 6-1, shown below and in Appendix D;

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

Sa(0.2)	Sa(0.5)	Sa(2.0)	PGA	PGV
0.142	0.080	0.019	0.088	0.062

The PGA for 2500 years return is 0.249 according to NRCan hazard maps 2015.

For the purposes of site-specific seismic response to earthquakes based on Table 4.1.8.4.A OBC 2012, the site can be classified as a Site Class "C", if building loads are transferred to the bedrock. For footings borne on sand a Site Class "D" shall be considered for the design.

6.5 Liquefaction Potential

Sudden loss in stiffness and strength of the subgrade due to cyclic loading, or seismic liquefaction, was considered for this site. The reason for liquefaction study was the presence of poorly graded sand with percentage fines less than 10% and relatively very high groundwater table. The analytical approach to assess liquefaction potential involves calculation of cyclic stress ratio (CSR) and comparing that value with cyclic resistance ratio (CRR).

It is understood there will be a basement proposed to the building. As the borehole investigation logs indicate, sand compactness increases around 2.5 m and deeper below existing ground surface. Therefore, liquefaction calculations were considered for the 4 m depth. Calculation assumptions included a PGA of 0.249, overburden total stress of 80 kPa, effective stress of 45 kPa and stress reduction value of 0.97 and resulted in CSR value of 0.03 which indicates the subgrade at the assumed depth is not liquefiable. When groundwater is relatively high, the CSR value is usually higher for internal columns due to less overburden pressure. Nevertheless, the subject building has to be designed for controlling hydrostatic uplift pressure if a basement is included in the design. Authors of this report recommend to revisit these calculations once the preliminary structural design is completed.

6.6 Foundation Design Options

6.6.1 Spread and Strip Footings

These footings are primarily proposed for the rectangular building at the northeast of the property. All boreholes indicated the lowest SPT 'N' value around 1.7 m to 2 m below existing surface. It is recommended to place the proposed shallow spread and strip footings at approximately 2.5 m below surface or lower as the ground demonstrates higher resistance at depths lower than 2.5 m. For calculation purposes it was assumed these footings will be 1.5 m to 3 m in shorter dimension.

The Serviceability Limit State for conventional sizes of shallow footings, usually less than 3 m in shorter dimension, can be calculated using Burland and Burbidge method. Also, a deduction factor equal to 0.55 was considered to reflect the submerge state of the footings and the depth of groundwater table below existing ground.

Table 6-2: SLS Values for Shallow Footings at 2.5 m Depth

Footing's shorter dimension (m)	$\overline{N}_{60}$	Founding Depth (m)	Allowable Settlement (mm)	SLS (kPa)
1.5	7	2.5	25	85
3.0	7	2.5	25	55

The Ultimate Limit State (ULS) for spread footings placed below 2.5 m (approximate El. 97) can be calculated using Terzaghi bearing capacity correlations;

Table 6-3: ULS Values for Shallow Footings at 2.5 m Depth

Footing's shorter dimension (m)	Groundwater Depth (m)	Founding Depth (m)	Friction angle	Unit Weight (kN/m³)	ULS (kPa)
1.5	0.3	2.5	30	17	150
3.0	0.3	2.5	30	17	180

6.6.2 Structural Slabs-on-Grade (Raft)

These design recommendations assume the floating slab is designed as a reinforced concrete structure or raft footing. It is understood this spread footing will be designed with adequate structural (i.e. flexural) strength so the design will compensate for the lack of stiffness of the subgrade. In this case the structural slab will be directly supported on the native subgrade. It is understood the slab on grade will be 7.5 m at its narrowest section. In general, for granular soil and under drained condition, shallower the footings (less over burden pressure) lower the ULS bearing capacity values. Therefore, the bearing capacity of the floating slab on grade constructed close to the surface was calculated relatively low. The low SPT values of sand close to the surface were also brought into account. Terzaghi bearing capacity correlations were used for calculation of ultimate bearing capacity. However due to the large width of the proposed footing, conventional empirical serviceability correlations based on SPT 'N' values appeared irrelevant. Therefore, the serviceability was calculated using finite element analysis.

Ultimate bearing capacities was calculated considering 0.5 m of surcharge and internal friction angle of 30 degrees for a 7.5 m wide footing. A factored ULS value of 300 kPa can be considered for the design. It is understood a spring constant is needed for the finite element design of the slab on grade.

The spring constant for the structural design of the slab can be taken as 20x10^6 N/m³. This value is not derived by direct calculation of deformation vs. factored ULS since the deformation of subgrade under governing load combinations, which most possibly includes dynamic loads, is expected to be less than serviceability settlement. The spring constant here in provided are based on the Young's modulus considered for this sand.

The Serviceability Limit State is controlled by the spring modulus provided for the ultimate capacity design (i.e. 25 mm settlement under expected loads). However, to be consistent with Canadian Foundation Engineering Manual practice, the SLS value for footings wider than 3 m can be taken assuming 3 m width. Therefore, for design check purposes and SLS value of 55 kPa can be used. If required by the structural engineer, a more realistic SLS value can be calculated through elastic Mohr-Coulomb finite element analysis.

Existing ground shall be excavated to the native sand subgrade. Load bearing insulation shall be provided for underneath the raft footing, projecting beyond the slab equal to the difference of frost penetration depth and the proposed soil cover.

If the site has to be over excavated due to presence of unsuitable material, the fill should be placed in horizontal lifts of uniform thickness of no more than 300 mm before compaction and it should be placed at appropriate moisture content. 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).

All non-structural slab-on-grade units shall float independently from all load-bearing structural elements. These slabs can be supported on minimum 200 mm granular A compacted to 100% SPMDD on native subgrade and separated from the subgrade by a layer of geotextile to provide both filtering function and resisting compaction puncture. These non-structural slabs shall be also protected from frost effects on subgrade.

6.6.3 Footings and Caissons on Rock

It is understood the detached hexagonal building is proposed to be supported on caissons or piers bearing on rock. This is the preferred approach since the two-storey hexagonal building and the southern portion of the main prayer facility will be connected through a hallway at second level. Therefore, it is important to control the differential settlement of the two buildings within a defined tolerable range and founding both structures on rock is a reasonable approach.

For footings bearing on rock, an Ultimate Limit State of 500 kPa can be assumed for the rock considering surficial fractures. Serviceability Limit State is not applicable for footings placed on rock and considering expected conventional loads.

Buoyancy forces shall be considered for the design once footings are founded on rock and a basement is included in the design.

Soil improvement options such as rammed aggregate piers supporting strip footings and spread footings are not discussed in this report.

6.7 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 6-4: Backfill Material Properties

Borehole	Granular “A”	Granular “B”
Effective Internal Friction Angle, ϕ'	35°	30°
Unit Weight, γ (kN/m^3)	22.8	22.8

7.0 CONSTRUCTION CONSIDERATIONS

Any organic material and existing fill material of any kind, shall be removed from the footprint of the footings and all structurally load bearing elements. If grade raise above the native subgrade is required suitable fill material to conform to specifications of OPSS Granular criteria shall be used. The Structural Fill should be free from any recycled or deleterious material, it should not be placed in lifts thicker than 300 mm and should be compacted as specified.

Given the encountered groundwater level and the overburden grain size distribution which implies high hydraulic conductivity, a relatively large flow of groundwater is expected in the excavation. A Permit to Take Water may be necessary to obtain. The groundwater elevation is expected to fluctuate seasonally which can change the amount of groundwater discharge. The founding level shall be kept dry at all time to minimize disturbance.

A dewatering program may become necessary to temporarily lower the groundwater table before start of the construction/excavation.

If construction is going to be conducted in multiple stages, care must be taken dewatering of any current construction phase shall not affect established buildings.

Soil type shall be considered as Type 4 for dewatered sand according to Ontario Health and Safety manual. Therefore, an excavation slope of 3H:1V or flatter is needed. If sand is not dewatered or it remains overly wet, temporary sheet piles or trench boxes may need to be driven to the rock to facilitate excavation.

For placement of any engineered fill, a geotechnical staff 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 due to its poor gradation, unless otherwise proven suitable by laboratory testing on bulk samples obtained during construction.

8.0 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. 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.

Since the native sand is expected to be of high permeability, installation of cut-off walls for utility trenches does not seem necessary.

9.0 PAVEMENT RECOMMENDATIONS

It is understood as part of this construction, a final total of approximately 206 parking spots will be constructed on the property. It is expected the pavement structure will likely to be placed on existing sandy material. The topsoil and any soft materials should be removed and the top of the sand should be compacted (proof rolled) under the supervision of a geotechnical staff. If parking areas contain organics or a higher thickness of topsoil/soft material, this material should be excavated prior to the parking lot construction. 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 the pavement structure. The proposed pavement structure is included in below tables.

Table 9-1: Proposed Light Use Pavement Structure – Passenger Vehicles

Material		Thickness (mm)
Surface	Superpave 12 mm, Design Category B (or HL 3), PG 58-34	50
Base	OPSS Granular A	150
Sub-base	OPSS Granular B Type II	450

Table 9-2: Proposed Heavy Use Pavement Structure (e.g. Fire Truck Route)

Material		Thickness (mm)
Surface	Superpave 12.5 mm, Design Category B (or HL 3), PG 58-34	50
Binder	Superpave 19 mm, Design Category B (or HL 8), PG 58-34	50
Base	OPSS Granular A	150
Sub-base	OPSS Granular B Type II	550

Table 9-3: Proposed Gravel Surface Heavy Use Pavement Structure (e.g. Fire Truck Route)

Material		Thickness (mm)
Base	OPSS Granular A	200
Sub-base	OPSS Granular B Type II	600

Both base and sub-base should be compacted to 100% standard Proctor maximum dry density (SPMDD). Existing sandy material is not suitable to be used for pavement structure. Asphalt layers should be compacted to comply with OPSS 310.

Due to the large size of the parking lot adequate drainage structures will be required.

From pavement strength design standpoint asphalt is the preferred option, however, it is understood the designers are also considering use of gravel surface pavements. There might be also an advantage with using gravel surfaced pavements due to high groundwater at this site. One of the factors which negatively impacts pavement longevity is presence of undrained water within the frost penetration depth. For this site, since the groundwater table is relatively high, either the pavement structure has to be built up, or the groundwater has to be drained to a lower elevation. If neither is considered in the design, then a gravel surfaced pavement maybe less expensive to maintain after each freeze-thaw cycle. Whereas an asphalt paved surface at the presence of high groundwater table may experience severe frost heave distress and cracking after each seasonal cycle. To emphasize, the pavement structures shown in above tables are adequate to tolerate intended loads, but the high groundwater table can reduce the pavement life, unless the site is built up or the water is lowered.

10.0 CEMENT TYPE AND CORROSION POTENTIAL

Samples from subgrade soil were submitted to Paracel Laboratories for testing of chemical properties relevant to exposure of concrete elements to sulfate attack, as well as potential soil corrosivity effects on the buried metallic structural elements. Test results are presented in Table 5-1.

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

The soil pH is quite acidic, which indicates the environment for buried steel element is within the aggressive range. In general, all steel components of the building buried in within a material with relatively high hydraulic conductivity, such as the native sand of this site, and being exposed to wetting drying cycles due to fluctuation of the groundwater table, are prone to corrosion.

11.0 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.

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12.0 REFERENCES

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

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6688 FRANKTOWN ROAD

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.

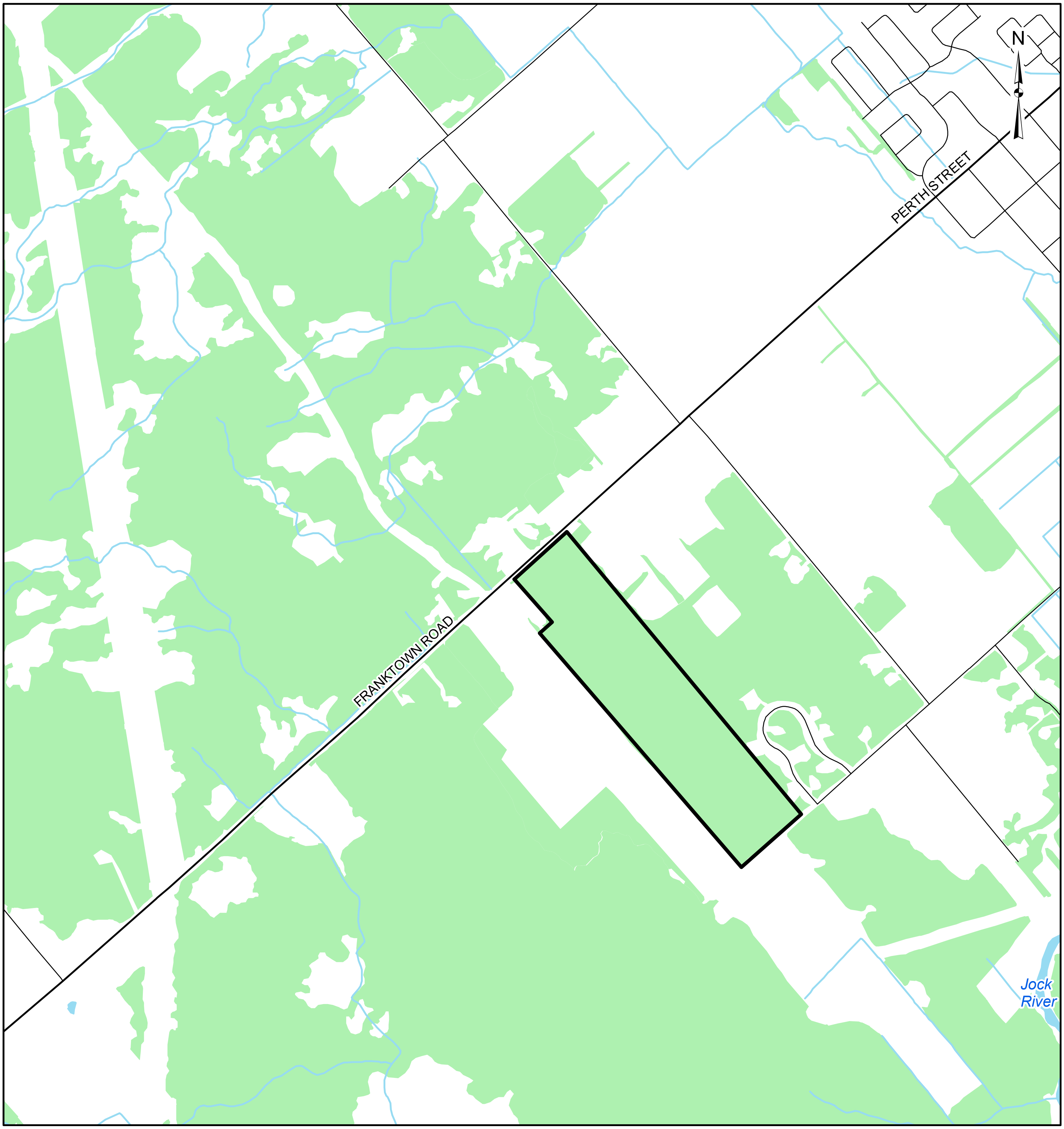
Under no circumstances shall the liability of McIntosh Perry for any claim in contract or in tort, related to the services provided and/or the content and recommendations in this report, exceed the extent that such liability is covered by such professional liability insurance from time to time in effect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

6688 FRANKTOWN ROAD

APPENDIX B
FIGURES

McINTOSH PERRY



LEGEND

Local Road	Watercourse
Major Road	Waterbody
Wooded Area	

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2018.

CLIENT: BING PROFESSIONAL ENGINEERING

PROJECT: GEOTECHNICAL INVESTIGATION
6688 FRANKTOWN ROAD

TITLE: SITE LOCATION

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LEGEND

Borehole Location

Approximate Property Boundary

Proposed Development

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2018.

CLIENT: BING PROFESSIONAL ENGINEERING

PROJECT: GEOTECHNICAL INVESTIGATION
6688 FRANKTOWN ROAD

TITLE: BOREHOLE LOCATIONS

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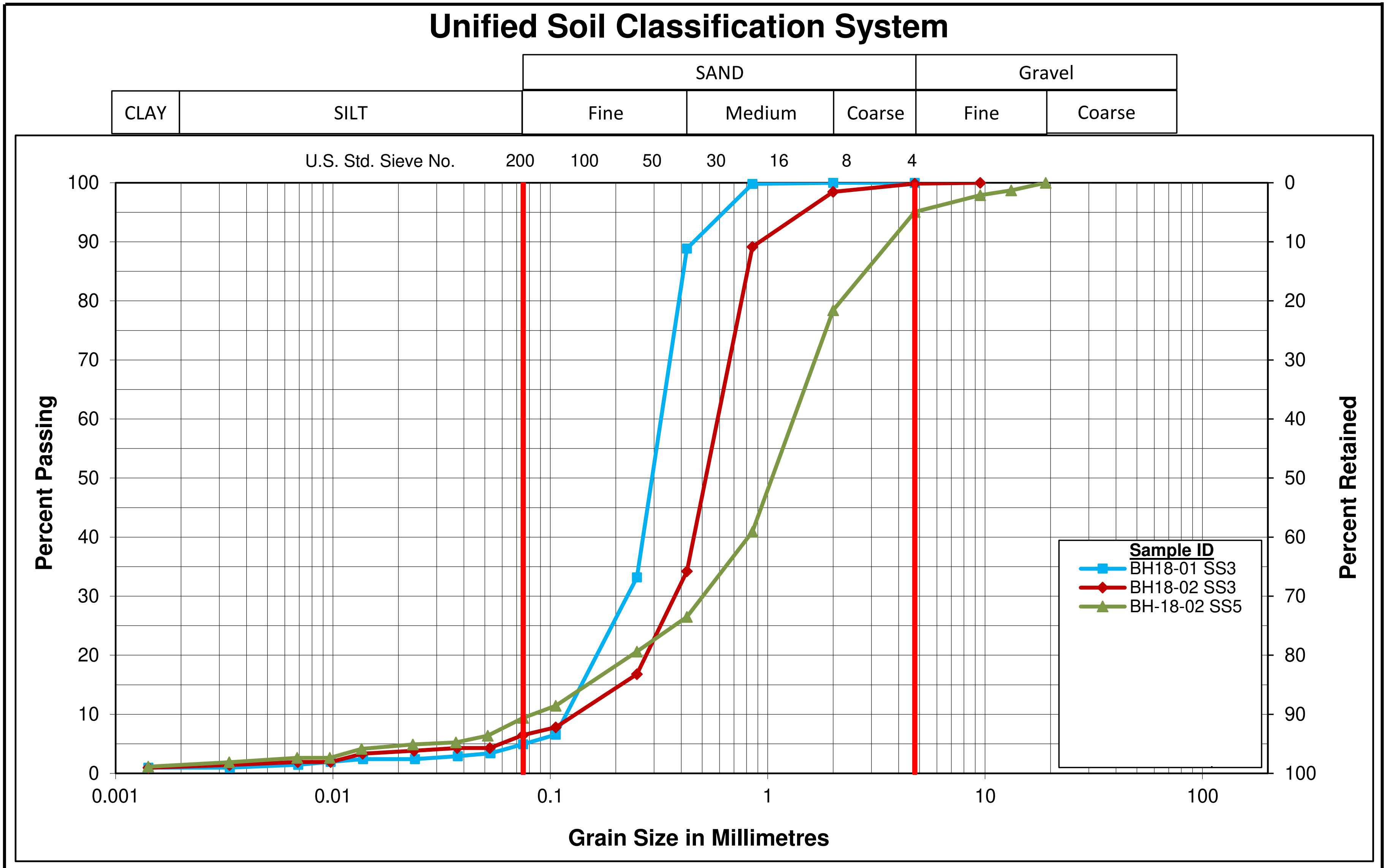

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GRAIN SIZE DISTRIBUTION

SAND

Figure No. 3

Project No. CP-17-0503

6688 FRANKTOWN ROAD

APPENDIX C
BOREHOLE LOGS

McINTOSH PERRY

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	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
Φ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
s_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $(W_L - W_L)$
I_L	1	LIQUIDITY INDEX = $(W - W_P)/ I_P$
I_C	1	CONSISTENCY INDEX = $(W_L - W) / 1_P$
$e_{,max}$	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	N PERCENT – DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

MCINTOSH PERRY

RECORD OF BOREHOLE No 18-01

DATE:	23/05/2018 - 24/05/2018	**LOCATION:**	6688 Franktown Road ()	**ORIGINATED BY:**	PH
PROJECT:	CP-17-0503BING	**COORDINATES:**	Lat: 45.17634877 , Lon: -75.86253728	**COMPILED BY:**	MG
CLIENT:	Bing Professional Engineering Inc.	**DATUM:**	Geodetic	**CHECKED BY:**	NT
ELEVATION:	100.0 m	**REMARK:**		**REPORT DATE:**	29/06/2018

ELEVATION - m / DEPTH - m	DESCRIPTION (SOIL PROFILE)	TYPE AND NUMBER	STATE	RECOVERY	"N" or RQD	GROUNDWATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C
100.0	Natural ground surface							
0.0	Silty sand, traces of gravel, brown, moist to wet, very loose. Presence of organic matter.	SS-01		75	5	0.3 m		
99.8 / 0.2	Sand, traces of clay and silt, brown to light grey, wet, loose.	SS-02		67	7			
		SS-03		88	2			0 95 4 1
		SS-04		100	10			
		SS-05		100	27			
96.5 / 3.5	Sand, traces of clay, silt and gravel, light grey, wet, compact to dense.	SS-06		44	49			
95.5 / 4.6	Limestone Bedrock, good quality, slightly weathered, grey.							UCS = 143 MPa
		RC-1		100	*76*			
		RC-2		100	*76*			
92.1 / 7.9	**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

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

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

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McINTOSH PERRY

RECORD OF BOREHOLE No 18-02

DATE:	23/05/2018 - 24/05/2018	**LOCATION:**	6688 Franktown Road ()	**ORIGINATED BY:**	PH
PROJECT:	CP-17-0503BING	**COORDINATES:**	Lat: 45.17596968 , Lon: -75.86225266	**COMPILED BY:**	MG
CLIENT:	Bing Professional Engineering Inc.	**DATUM:**	Geodetic	**CHECKED BY:**	NT
ELEVATION:	100.9 m	**REMARK:**		**REPORT DATE:**	29/06/2018

ELEVATION - m / DEPTH - m	SOIL PROFILE – DESCRIPTION	SAMPLE TYPE AND NUMBER	RECOVERY	"N" or RQD	GROUNDWATER CONDITIONS	WATER CONTENT (%)	G	S	M	C
100.9	Natural ground surface									
0.0	Peat, trace silty sand, light grey, wet									
100.6 / 0.3	Sand, traces of clay and silt, brown, moist to wet, very loose to loose.	SS-01	58	2	0.3 m					
		SS-02	75	4						
		SS-03	100	0		○	0	94	5	1
97.5 / 3.4	Sand, traces of clay, silt and gravel, grey, wet, loose to compact.	SS-04	42	6						
		SS-05	57	20		○	5	86	8	1
		SS-06	65	15						
95.6 / 5.2	Limestone Bedrock, good quality, slightly weathered, grey.	RC-1	100	*86*						
		RC-2	100	*85*						
93.9 / 6.9	**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
REMARKS & GRAIN SIZE DISTRIBUTION (%): G S M C

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

\\LICENSES7\Sobek\Geotec80\Style\Log_Borehole_v5.sty

McINTOSH PERRY

RECORD OF BOREHOLE No 18-03

DATE:	23/05/2018 - 23/05/2018	**LOCATION:**	6688 Franktown Road ()	**ORIGINATED BY:**	PH
PROJECT:	CP-17-0503BING	**COORDINATES:**	Lat: 45.17596968 , Lon: -75.86225266	**COMPILED BY:**	MG
CLIENT:	Bing Professional Engineering Inc.	**DATUM:**	Geodetic	**CHECKED BY:**	NT
ELEVATION:	99.8 m	**REMARK:**		**REPORT DATE:**	21/06/2018

ELEVATION - m / DEPTH - m	DESCRIPTION (SOIL PROFILE)	TYPE AND NUMBER	RECOVERY	"N" or RQD	GROUNDWATER CONDITIONS
99.8	Natural ground surface				
0.0	Peat, trace organics (roots)				
99.5 / 0.3	Sand, traces of clay and silt, light brown to grey, wet, loose to compact.	SS-01	67	2	0.3 m
		SS-02		4	
		SS-03		3	
		SS-04	75	10	
		SS-05	83	12	
		SS-06	92	15	
		SS-07	100	33	
94.8 / 5.0	Sand, traces of clay, silt and gravel, grey, wet, dense.	SS-08	80	58	
94.1 / 5.7	**END OF BOREHOLE** **Auger refusal on probable bedrock.**				

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

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

CP-17-0503 6688 Franktown Road
BH18-01
RC-01: 4.78 m - 6.43 m

CP-17-0503 6688 Franktown Road
BH18-01
RC-02: 6.43 m - 7.92 m

CP-17-0503 6688 Franktown Road
BH18-02
RC-01: 5.16 m - 6.43 m
RC-02: 6.43 m - 6.93 m

6688 FRANKTOWN ROAD

APPENDIX D
LAB RESULTS

McINTOSH PERRY

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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: 6688 Franktown Rd CP-17-0503
Project: CP-17-0503
Custody: 40897

Report Date: 8-Jun-2018
Order Date: 4-Jun-2018

Order #: 1823084

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

Paracel ID	Client ID
1823084-01	CP-17-0503 BH18-02 SS-02

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis
Client: McIntosh Perry Consulting Eng. (Carp)
Client PO: 6688 Franktown Rd CP-17-0503

Report Date: 08-Jun-2018
Order Date: 4-Jun-2018
Project Description: CP-17-0503

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	6-Jun-18	7-Jun-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	5-Jun-18	6-Jun-18
Resistivity	EPA 120.1 - probe, water extraction	7-Jun-18	7-Jun-18
Solids, %	Gravimetric, calculation	8-Jun-18	8-Jun-18

PARACEL
LABORATORIES LTD.

Order #: 1823084

Certificate of Analysis

Client: McIntosh Perry Consulting Eng. (Carp)

Client PO: 6688 Franktown Rd CP-17-0503

Report Date: 08-Jun-2018

Order Date: 4-Jun-2018

Project Description: CP-17-0503

Client ID:		CP-17-0503 BH18-02 SS-02	-	-	-
Sample Date:		05/23/2018 09:00	-	-	-
Sample ID:		1823084-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	79.9	-	-	-
General Inorganics					
pH	0.05 pH Units	5.88	-	-	-
Resistivity	0.10 Ohm.m	285	-	-	-
Anions					
Chloride	5 ug/g dry	6	-	-	-
Sulphate	5 ug/g dry	20	-	-	-

Certificate of Analysis
Client: McIntosh Perry Consulting Eng. (Carp)
Client PO: 6688 Franktown Rd CP-17-0503

Report Date: 08-Jun-2018
Order Date: 4-Jun-2018
Project Description: CP-17-0503

Method Quality Control: Blank

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

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	7.8	5	ug/g dry	8.0			2.2	20	
Sulphate	57.3	5	ug/g dry	53.6			6.6	20	
General Inorganics									
pH	7.57	0.05	pH Units	7.65			1.1	10	
Resistivity	52.5	0.10	Ohm.m	49.5			5.9	20	
Physical Characteristics									
% Solids	97.8	0.1	% by Wt.	97.6			0.2	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	99.6	5	ug/g	8.0	91.7	78-113			
Sulphate	147	5	ug/g	53.6	93.3	78-111			

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.

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.

LRL Associates Ltd.

Unconfined Compressive Strength of Intact Rock Core

ASTM D 7012: Method C

LRL ENGINEERING | INGÉNIERIE

Client: McIntosh Perry Consulting Engineers
Project: Materials Testing
Location: Franktown Road

Reference No.: CP-17-0503
File No.: 170496-32
Report No.: 1

Drill Core Information

Date(s) Sampled: May 24, 2018
Sampled By: McIntosh Perry Consulting Engineers
Date Received: June 1, 2018

Laboratory Identification	Core No.	Field Identification	Borehole	Run	Depth	Location / Description
C0710	1		18-1	RC-1	4.82 m - 5.24 m	Franktown Road

Rock Core Unconfined Compressive Strength Test Data

Laboratory Identification	Core No.	Conditioning	Length, mm	Diameter, mm		MPa	Description of Failure
C0710	1	As received	97.1	47.2		142.7	Predominantly columnar, relatively well formed cone on one end

Comments:

Date Issued: June 7, 2018

Reviewed By: W.A. McLaughlin

W.A.M^cLaughlin, Geo.Tech., C.Tech.

6688 FRANKTOWN ROAD

APPENDIX E
SEISMIC HAZARD CALCULATION

McINTOSH PERRY

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

June 29, 2018

Site: 45.178 N, 75.864 W   User File Reference:

Requested by: ,

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

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.391	0.462	**0.389**	0.297	**0.213**	**0.108**	**0.052**	**0.014**	**0.0051**	**0.249**	**0.177**

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. ***These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.037	0.125	0.211
Sa(0.1)	0.053	0.161	0.260
Sa(0.2)	0.049	0.142	0.224
Sa(0.3)	0.039	0.111	0.174
Sa(0.5)	0.029	0.080	0.125
Sa(1.0)	0.014	0.042	0.064
Sa(2.0)	0.0057	0.019	0.031
Sa(5.0)	0.0012	0.0045	0.0076
Sa(10.0)	0.0006	0.0018	0.0030
PGA	0.028	0.088	0.142
PGV	0.019	0.062	0.100

References

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

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

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

Aussi disponible en français