Addendum No. 4

To: International Buddhist Progress Society of Ottawa - Carleton

From: Egis Group Ltd. (formerly McIntosh Perry Consulting Engineers Ltd.)

Date: January 21, 2025

Re: Additional Technical Information to the Revised Geotechnical Report Issued January 2019– IBPS - 6688 Franktown Rd Richmond

This is an addendum to the revised Geotechnical Report, dated January 2019, which is included in Appendix E. It provides updated recommendations for the foundation design of the proposed temple building, based on the new architectural and civil drawings provided by the International Buddhist Progress Society of Ottawa (the client), which reflect the building's relocation. This addendum must be read in conjunction with the revised report. The additional information and recommendations are presented below. The included information does not replace that which already exists in the revised January 2019 report but serves to augment or update as necessary.

1.0 BACKGROUND

Subsequent to the issuance of the updated site plan, the client has requested an update to the foundation design recommendations to reflect the relocation of the temple structure. The revised geotechnical report issued by McIntosh Perry in January 2019 had recommended either raft footing or deep foundations, such as footings and caissons on rock, or helical piles for the temple building.

2.0 SITE DESCRIPTION

In Section 2.0 Site Description, the proposed development description and composition are amended by the replacement with the following:

The revised proposed building will be a single-storey structure with no basement, designed with an approximate footprint of 1,398 m².

Furthermore, Figure 2, Borehole Locations, included in Appendix B of the revised report, has been updated to reflect the borehole locations corresponding to the revised location of the proposed temple building, as outlined in Appendix A of this addendum.

3.0 CHEMICAL ANALYSIS

Section 5.3 Chemical Analysis, Table 5-1: Soil Chemical Analysis Results, is amended by the replacement with the following table:

Borehole ID	Sample	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm.cm)
BH18-02	SS-02	0.8 – 1.4	5.88	0.0020	0.0006	28,500

Based on electrical resistivity results and chloride content, the corrosion potential for buried steel elements is within the nonaggressive range. However, all steel components of the building buried 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 fluctuations of the groundwater table, are susceptible to corrosion.

4.0 GROUNDWATER

Section 5.4 Groundwater, Table 5-2: Groundwater Levels, is amended by the replacement with the following table:

Borehole	BH Elev. (m)	Date	Water Level Reading (m)	Groundwater Elev. (m)
MW18-1	100.900	June 15, 2018	1.532	99.368
MW18-2	100.680	June 15, 2018	1.327	99.353
MW18-3	100.960	June 15, 2018	1.115	99.845
MW18-1	100.900	December 6, 2024	1.77	99.13
MW18-2	100.680	December 6, 2024	1.72	98.96
MW18-3	100.960	December 6, 2024	0.58	100.38

5.0 RECOMMENDATION FOR FOUNDATION DESIGN

- The revised report recommends deep foundation options as the preferred solution for the temple structure, given that the shallow foundation alternative is considered impractical due to the substantial dewatering efforts required for open excavation and the construction of shallow footings.
- A technical memorandum (Appendix B) was sent in April 2019 to respond to the structural engineer's inquiry after reviewing the structural drawings of the existing temple building. The memorandum confirmed that the pile cap supported by helical piles is a practical and suitable option. It was also noted that the helical piles are designed by the specialty contractor to accommodate the reactions specified in

the structural drawings, and the installation procedure ensures that the required capacity is achieved for each pile.

- Based on the architectural site plan A002 Rev. 1 (included in Appendix C), prepared by GRC Architects, dated December 18, 2024, provided by the client, the temple building will be a single-storey structure with no basement, with an approximate footprint of 1,398 m². The new location of the building, as indicated in the site plan, is supported by the borehole data shown in Appendix A. Specifically, boreholes BH18-1, BH18-2, and MW18-1, located within the building footprint, remain valid and provide the necessary geotechnical data for the foundation design.

- Section 6.4, Table 6-1: Selected Seismic Spectral Responses (2% in 50 Years), is amended by replacing it with the following table, in accordance with the National Building Code of Canada 2020 instead of 2010 NBC Seismic Hazard calculation in the Appendix E. Additionally, APPENDIX E, SEISMIC HAZARD CALCULATION, of the revised report is amended by replacing it with APPENDIX D, SEISMIC HAZARD CALCULATION, from this addendum.

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.575	0.491	0.293	0.141	0.338

- According to the site grading plan titled C200 Rev. 04 (included in Appendix C), prepared by EXP Services Inc., dated December 18, 2024, provided by the client, the Finish Floor Elevation is set at 102 m asl. The existing site grades beneath the building footprint range from 99.82 m to 100.02 m asl, indicating a grade raise of approximately 2.0 m.

- The following key factors should be considered when selecting the foundation types:

  1. Loose sand was encountered to a depth ranging from 2.5 to 3.5 m bgs and is not suitable for supporting shallow foundations.
  2. The existing high groundwater table was encountered at depth ranged from 1.33 to 1.53 m bgs in June 2018, and from 1.72 to 1.77 m bgs in December 2024.
  3. High hydraulic conductivity of the sand strata beneath the groundwater table.
  4. A grade raise of up to 2 meters has been identified.

- Based on the above factors, the preferred foundation option is deep foundations such as caissons (as mentioned in Section 6.5.3) or helical piles (Section 6.2.2), with a suspended floor slab instead of a slab-on-grade. This approach offers the following benefits, in addition to those mentioned in Section 6.5:

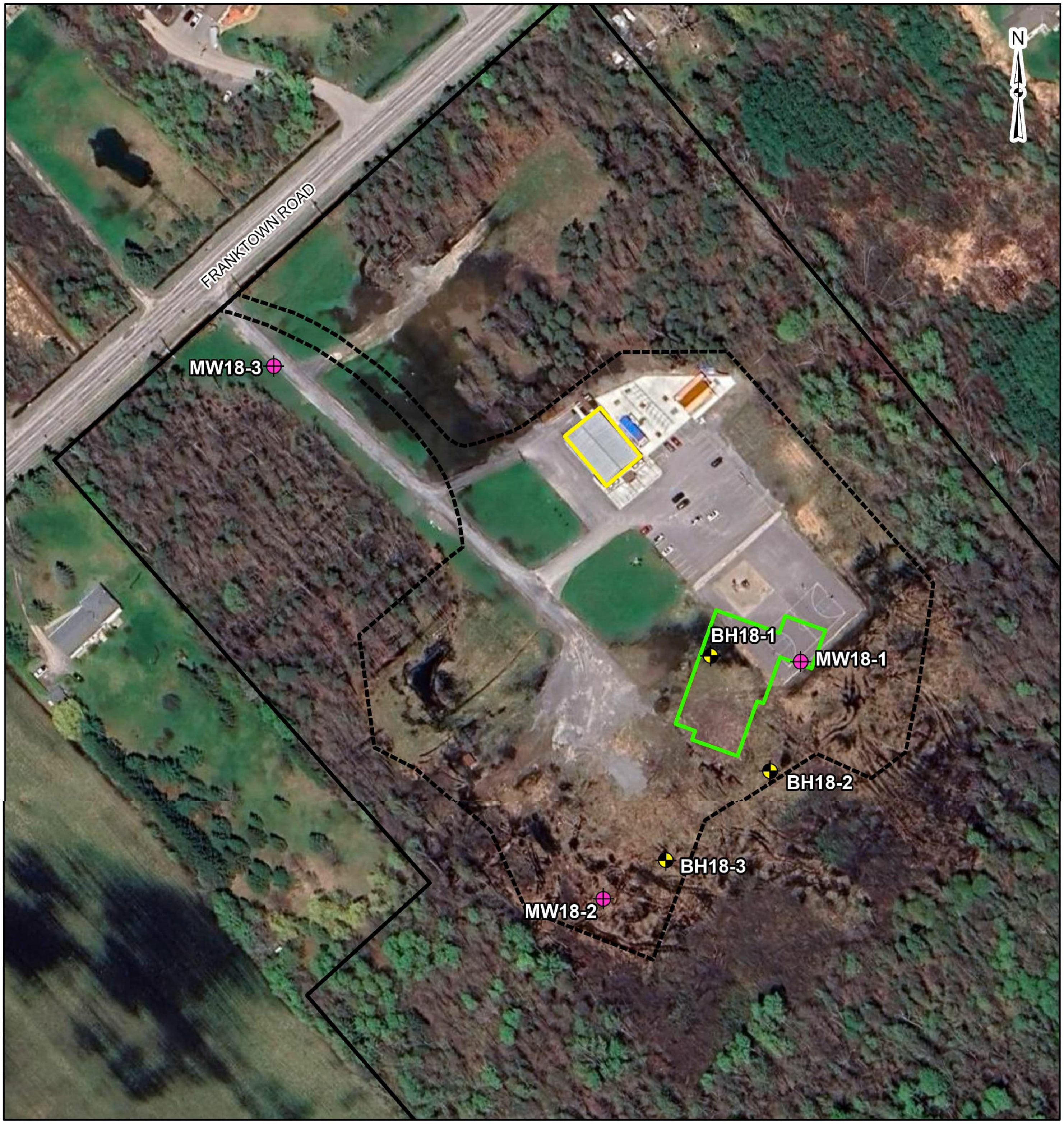
  - Avoid the costs associated with improving the subgrade for the slab-on-grade.
  - Eliminate the need for approximately 2,000 m³ of engineered fill for backfilling up to 2 meters for the grade raise.

6.0 CLOSURE

As noted, information provided in this addendum shall be read in conjunction with the revised geotechnical report and Addenda No. 1 to 3. It supersedes information provided in these documents in case of any contradiction.

Prepared By:	Reviewed By:
Nader Girgis, M.Sc., P.Eng., PMP Geotechnical Engineer nader.girgis@egis-group.com	Michelle Wang, M.Sc., P.Eng. Geotechnical Engineer michelle.wang@egis-group.com

APPENDIX A: BOREHOLE LOCATIONS



LEGEND

- Approximate Property Boundary
- Existing Building Footprint
- Proposed Building Footprint
- Proposed Development
- Monitoring Well Location
- Borehole Location

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2025.

CLIENT: INTERNATIONAL BUDDHIST PROGRESS SOCIETY OF OTTAWA

PROJECT: GEOTECHNICAL INVESTIGATION 6688 FRANKTOWN ROAD

TITLE: BOREHOLE LOCATIONS

egis
115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742

PROJECT NO:	CP-17-0503
Date	Jan. 14, 2025
GIS	LC
Checked By	MG

FIGURE: 2 Rev.1

C:\Users\c.zander\EGIS Group\GIS - Documents\Projects\2017\CP\0CP-17-0503 6688Franktown\aprx\Geotech\CP-17-0503_Geotech.aprx

APPENDIX B: TECHNICAL MEMORANDUM

McINTOSH PERRY

TECHNICAL MEMORANDUM

To: Bingfeng Li, P.Eng.

From: McIntosh Perry Consulting Engineers

Date: April 5, 2019

Re: Foundation Design Option for the 'Interim' Building, 6688 Franktown Rd, Ottawa, Ontario

The following structural drawings are reviewed for the project title "Proposed Foguangshan Temple Interim Building";

- Drawing S1, Foundation Plan, issued Feb. 20, 2019;
- Drawing S2, Foundation and Pier Details, issued Feb. 20, 2019.

It is understood that the selected foundation system is pile cap supported on helical piles. Provision of helical pile foundation system is noted in McIntosh Perry Geotechnical Report – Revised (the Geotechnical Report), Section 6.2.2, submitted January 9, 2019.

The foundation design concept and methodology as reviewed in the above-noted drawings, DWGs S1 and S2, are in conformance with the contents of the Geotechnical Report. It is practically feasible and considered a suitable option for the proposed Interim Building. Description of this building is included in the Geotechnical Report, Section 2.

A more competent sand deposit is expected at approximately 3.5 m below the existing surface and bedrock is expected at 4.5 m to 6 m below current grade. Helical piles are designed by the specialty contractor for the reactions listed in the structural drawings. Installation procedure warrants the required capacity is reached for each pile.

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

McIntosh Perry Consulting Engineers

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

n.tavakkoli@mcintoshperry.com

613.223.9207

APPENDIX C: ADDITIONAL DRAWINGS

3 SITE PLAN AREA OF DEVELOPMENT
A002 1 : 400

PROJECT NAME: **IBPS TEMPLE**

OWNER: INTERNATIONAL BUDDHIST PROGRESS SOCIETY OF OTTAWA-CARLETON
6688 FRANKTOWN ROAD,
RICHMOND, ON K0A 2Z0
T: 613 759 8111 F: 613 759 8110

ARCHITECT: GRC ARCHITECTS
401-47 CLARENCE STREET,
OTTAWA, ON K1N 9K1
T: 613 241 8203 F: 613 241 4180

LANDSCAPE ARCHITECT: JAMES B. LENNOX & ASSOCIATES INC.
3332 CARLING AVENUE,
OTTAWA, ON K2H 5A8
T: 613 722-5168

STRUCTURAL: CUNLIFFE & ASSOCIATES INC.
102 - 1737 WOODWARD DRIVE,
OTTAWA, ON K2C 0P9
T: 613 729-7242 F: 613 728-1461

CIVIL: EXP
2650 QUEENSVIEW DR SUITE 100,
OTTAWA, ON K2B 8H6
T: (613) 688-1899

ELECTRICAL & MECHANICAL: SMITH + ANDERSEN
1600 CARLING AVE #530
OTTAWA, ON K1Z 1G3
T: (613) 230-1186

SURVEYOR: (CONTRACTED BY OWNER) EGIS
3240 DRUMMOND CON. 5A, R.R. #7
PERTH, ON K7H 3C9
T: 613-267-6524 F 613-267-7992

ADDRESS 6688 FRANKTOWN RD, RICHMOND, ON K0A 2Z0

ZONING PROVISION	REQUIRED	PROVIDED
LEGAL DESCRIPTION	PART OF LOT 19 CONCESSION 3	
ZONE PROVISIONS (TABLE 223-RI ZONE PROVISIONS)	RI[643R] RURAL INSTITUTIONAL ZONE	
(a) Minimum lot width	30m min lot width	> 30m min lot width
(b) Minimum lot area	2000m² min lot area	> 2000m² min lot area
(c) Minimum front yard setback	6m front yard setback	> 6m setback
(d) Minimum rear yard setback	i) 7.5m rear yard setback	> 7.5m setback
(e) Minimum interior side yard setback	6m interior side yard setback	> 7.5m setback
(f) Minimum corner side yard setback	6m interior corner side yard setback	> 6m setback
(g) Maximum principal building height	10m maximum building height	10m maximum building height
COVERAGE (TABLE 223-RI ZONE PROVISIONS) (h) Maximum lot coverage (%)	50%	**Site Area:** 224031m² (plot RI[643R]) **Building Area:** Existing Building footprint: 348m² New Building footprint: 1398m² 1746m² = 0.8% (1746/224031) **Asphalt Area:** 9707m² = 4% (9707/224031)
LANDSCAPE (TABLE 223-RI ZONE PROVISIONS) (i) Minimum landscaped area (%)	20%	**Landscaped Area:** 212857m² = 95% (212857/224031)
GROSS FLOOR AREA (CALCULATION PER ZONING BY-LAW 2008-250 "DEFINITIONS (SEC. 54)")		
Existing Building		348m²
New Building		955m²
PARKING PROVISIONS (AREA "D" SCHEDULE 1A - N66 APPLIES TO ASSEMBLY AREA ONLY, ROW R22 6 ROOMING UNITS) ACCESSIBLE PARKING AS PER CITY OF OTTAWA ADS TABLE 3. PARKING RATES	**N66 Assembly Area:** = 10 per 100 m2 of Gross floor area of assembly area. = 73 parking spaces required (Shrine hall 342m2 + Dining room 192m2 + existing building workship area 195m2) **R22 Rooming Units:** 6 rooming units = 0.5 per rooming unit = 3 parking spaces required **ADS Table 3:** = 5 accessible spaces required	98 Standard Spaces (Assembly) 3 Standards Spaces (Rooming) 7 'Type A' Accessible Spaces **= 108 Total Spaces**
BICYCLE PARKING RATES (CALCULATION PER ZONING BY-LAW 2008-250 "DEFINITIONS (SEC. 111))	**111A (a) 0.25 per rooming unit** = 2 required	2 spaces 0.6m x 1.8m

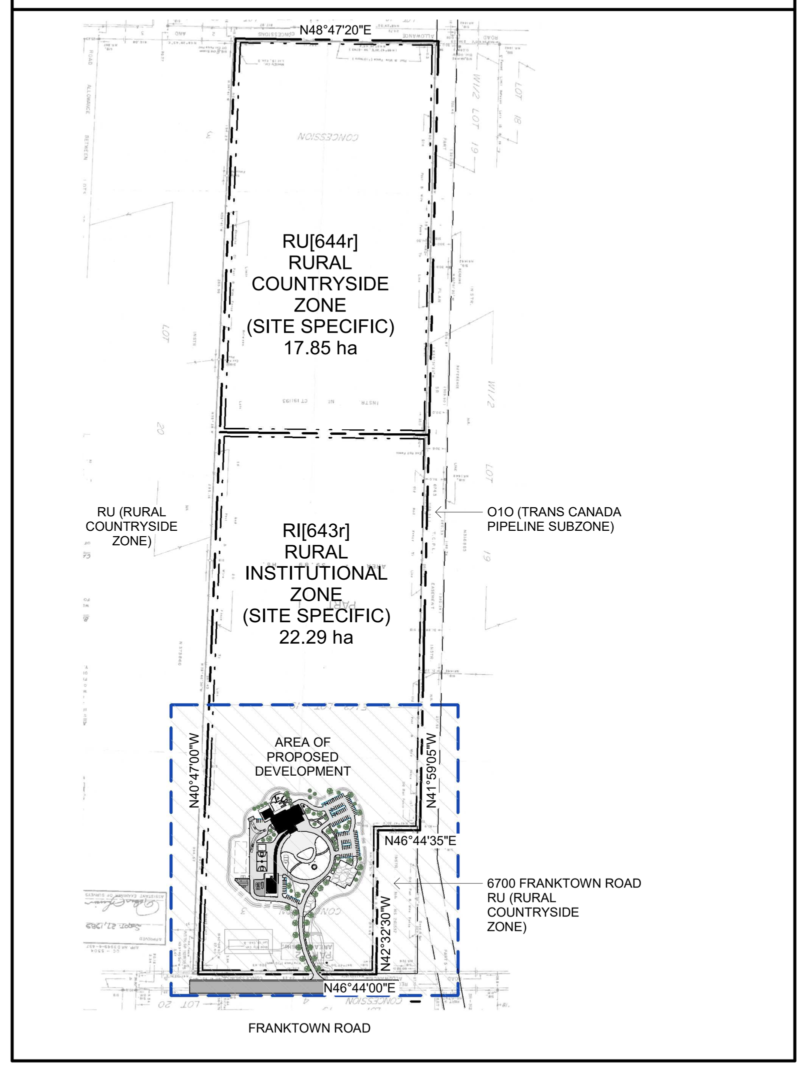
GENERAL NOTES

1. ALL DIMENSIONS ARE IN MILLIMETERS EXCEPT GRADE, FLOOR ELEVATIONS AND PROPERTY LINE DIMENSIONS (SHOWN IN METERS).
2. ALL LEGAL SURVEY INFORMATION OBTAINED FROM SURVEY PLANS PREPARED BY : MCINTOSH PERRY SURVEYING INC, DATED AUGUST 29, 2023.
3. REFER TO LANDSCAPE DRAWINGS FOR SOFT/HARD LANDSCAPING, AND PAVEMENT MATERIAL PATTERN LAYOUT.
4. REFER TO CIVIL DRAWING FOR SITE SERVICES, SITE GRADING, SURFACE DRAINAGE, ROAD WORK, PAVEMENT, SIDEWALK AND CONCRETE CURBS.
5. REFER TO ELECTRICAL DRAWING FOR LIGHT FIXTURES & ELECTRICAL SCOPE OF WORK.
6. ELEVATIONS SHOWN ARE REFERRED TO GEODETIC DATUM.
7. ALL PARKING SPACES TO HAVE 100mm WIDE PAINTED LINES ON ASPHALT, INCLUDING ACCESSIBLE PARKING SPACE PICTOGRAM.

FULL COURT BASKETBALL
- ALL GAME LINES FOR FULL COURT BASKETBALL TO BE PAINTED WHITE, UNLESS NOTED OTHERWISE
- LANE SPACE MARKS & NEUTRAL ZONE MARKS TO BE PAINTED A CONTRASTING COLOR TO THE BOUNDING LINES
- ALL GAME LINES FOR BASKETBALL TO BE 50mm WIDE, UNLESS NOTED OTHERWISE.

SITE PLAN LEGEND

PROPERTY LINE
BUILDING SETBACK
NEW CHAIN LINK FENCE
MAIN ENTRANCE
SECONDARY ENTRANCE
BARRIER FREE PARKING STALL
VEHICULAR DIRECTION
CONCRETE PAVERS, REFER TO LANDSCAPE DRAWINGS
EXISTING ASPHALT, SEE CIVIL
ASPHALT, SEE CIVIL
GRAVEL, SEE CIVIL
FH FIRE HYDRANT, SEE CIVIL
G 4 CUBIC YARD GARBAGE CONTAINER
R RECYCLE AREA
B NEW GALVANIZED STEEL BOLLARDS, PAINTED.
LP NEW LIGHT POST C/W CONCRETE BASE, REFER TO ELECTRICAL
ES EXTERIOR SIGNAGE. REFER TO ELECTRICAL
EXISTING CONCRETE, SEE CIVIL
CONCRETE, SEE CIVIL
EXISTING TREE CANOPY OUTLINE AS PER MCINTOSH PERRY SURVEY DATED AUGUST 29, 2023
PROPOSED TREE, SEE LANDSCAPE



LOCATION PLAN

no.	date	revision	by
1	18/12/2024	ISSUED FOR SITE PLAN CONTROL	AL

grc architects
A PROVENCHER_ROY COMPANY
47 Clarence Street, Suite 401
Ottawa, Ontario K1N 9K1
t:613-241-8203 f: 613-241-41-80
info@grcarchitects.com
www.grcarchitects.com

consultant

northpoint

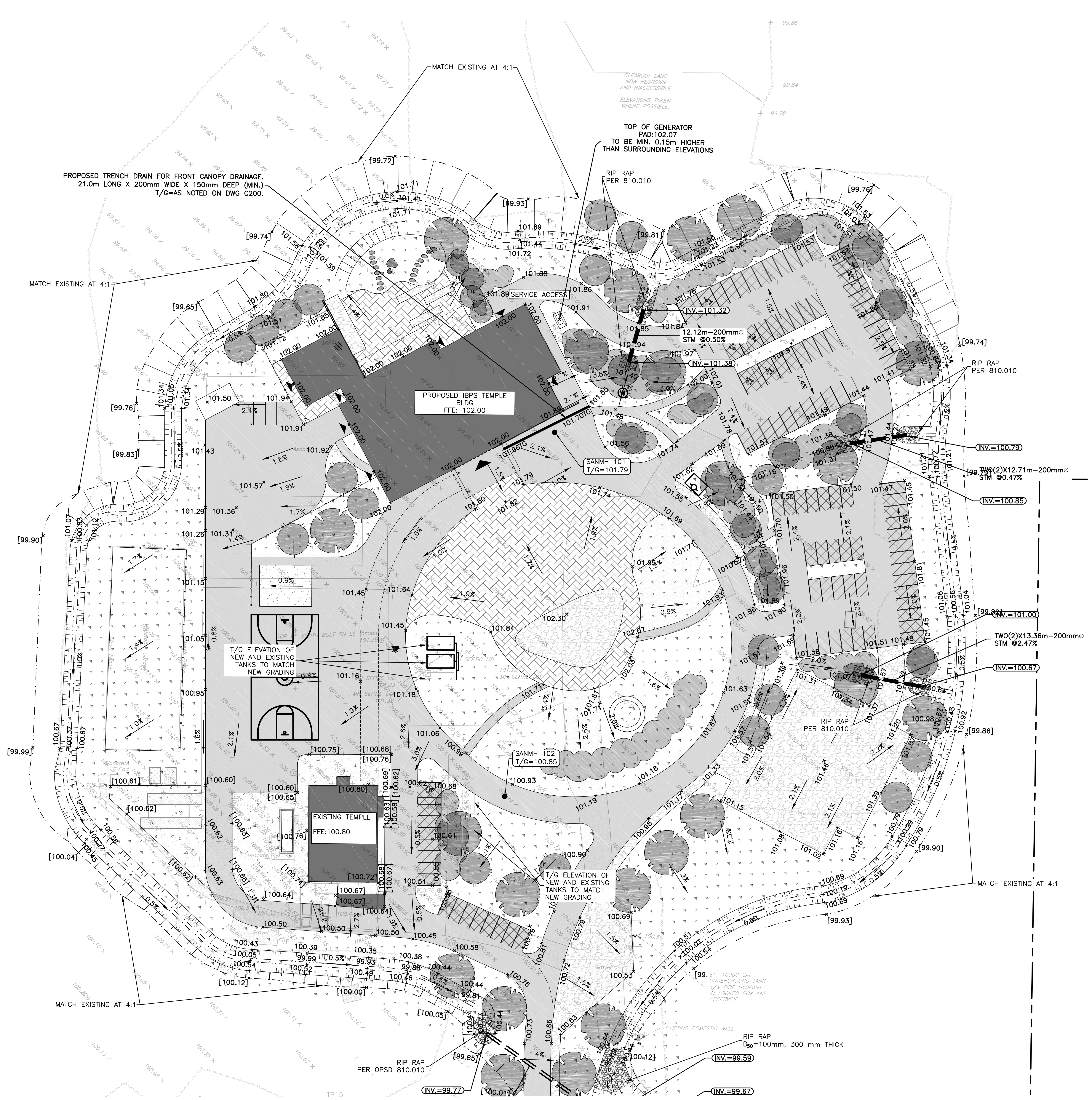
professional stamp
ONTARIO ASSOCIATION OF ARCHITECTS
ALEX LEUNG
LICENCE
5370

project title
IBPS TEMPLE
address Ottawa, ON

drawing title
SITE PLAN

date	DECEMBER 18, 2024	job. no.	**0623**
scale	As indicated		
drawn	CM	drawing no.	**A002**
approved	AL		
plot date	12/18/24		

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS AND NOTIFY THE ARCHITECT OF ANY DISCREPANCIES BEFORE WORK COMMENCES
3. THIS DRAWING TO BE READ IN CONJUNCTION WITH THE FOLLOWING DRAWINGS: STRUCTURAL, MECHANICAL, ELECTRICAL



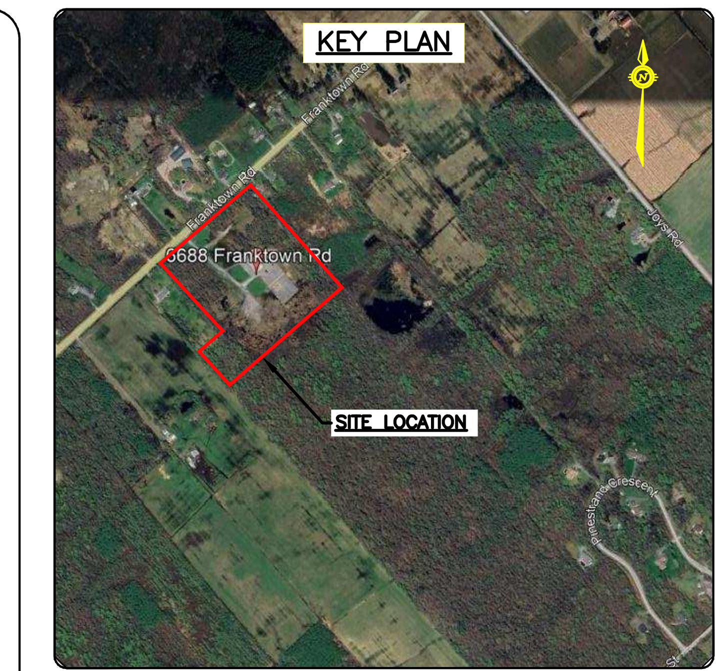
SITE GRADING PLAN (SOUTH)

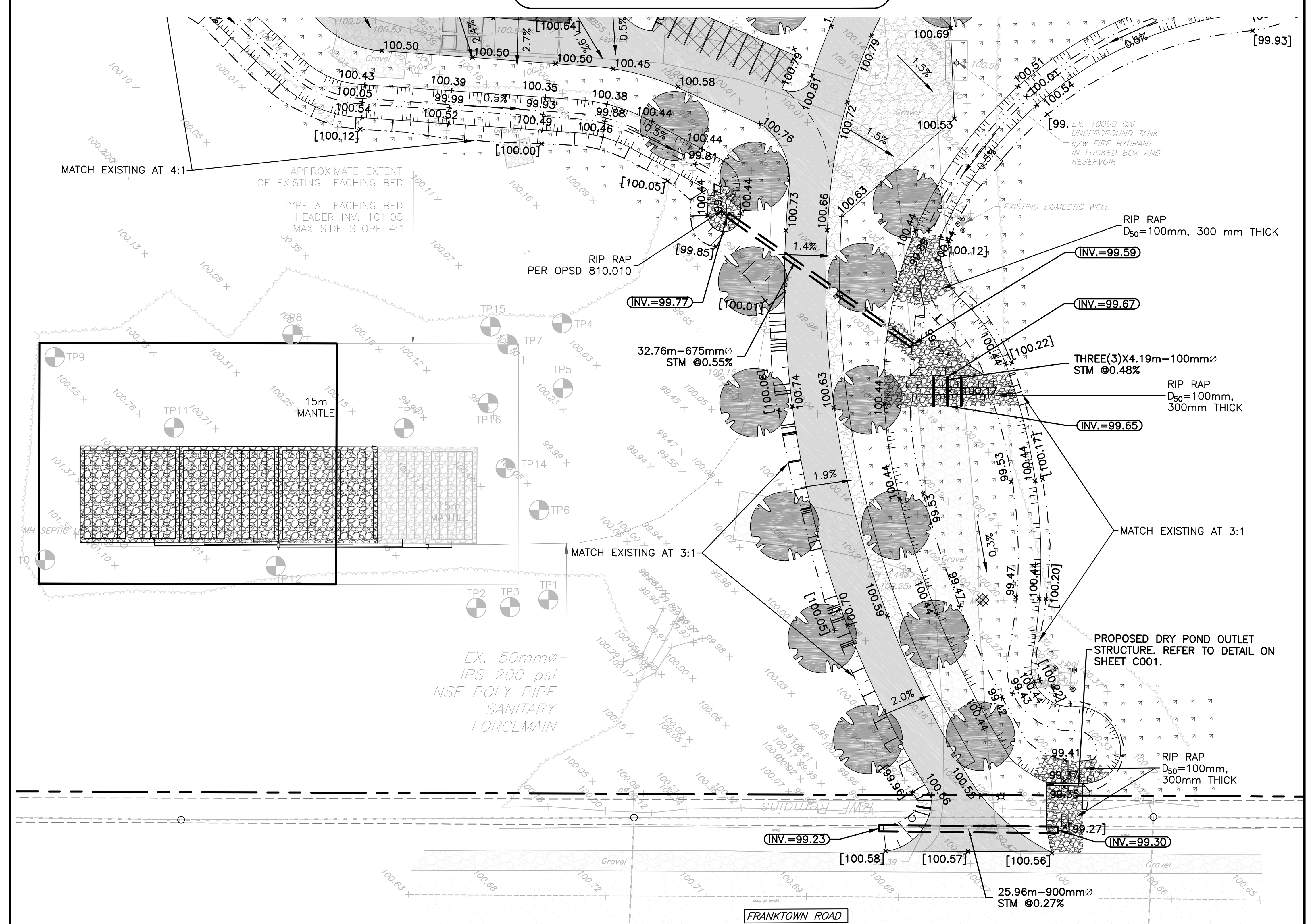
SCALE: 1:500

LEGEND

- PROPERTY LINE
- EXISTING ℄ OF ROAD
- EXISTING FEATURE (COVER NOTED)
- EXISTING ELEVATION
- EXISTING GRAVEL
- EXISTING ASPHALT
- EXISTING WATERLINE
- EXISTING SANITARY FORCEMAIN
- EXISTING FIRE HYDRANT
- PROPOSED SOFT LANDSCAPING
- PROPOSED HEAVY-DUTY ASPHALT
- PROPOSED LIGHT-DUTY ASPHALT
- PROPOSED GRAVEL
- PROPOSED TERRACING (3:1 MAX)
- PROPOSED SWALE/DITCH ℄
- PROPOSED STORM CULVERT
- PROPOSED WATER WELL
- PROPOSED SANITARY MANHOLE
- 99.29 PROPOSED GRADING ELEVATION
- [99.29] PROPOSED GRADING MATCH EXISTING ELEVATION
- 3.2% PROPOSED GRADING SLOPE

KEY PLAN





SITE GRADING PLAN (NORTH)

SCALE: 1:500

CAUTION
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

JOB BENCH MARK JBM
TOP OF SOUTH BOLT ON LIGHT STANDARD
ELEVATION=101.36

REV	REVISION DESCRIPTION	DATE	BY	APPD
4	ISSUED FOR SPA	18/12/24	AGJ	AKJ
3	REVISED SITE PLAN	16/10/24	AJ	AA
2	FOR CLIENT REVIEW	27/05/24	AGJ	AJ
1	FOR CLIENT REVIEW	18/04/24	AGJ	AJ

SCALE: 0 5m 10m 20m HORIZONTAL 1:500

NORTH

DESIGNED BY: LICENSED PROFESSIONAL ENGINEER A. K. JARIWALA 100562090 2024/12/18 PROVINCE OF ONTARIO

REVIEWED BY: LICENSED PROFESSIONAL ENGINEER M.A. ANSARI 2024/12/18 PROVINCE OF ONTARIO

CLIENT: GRC ARCHITECTS
47 CLARENCE STREET, SUITE 401
OTTAWA, ON. K1N 9K1
613.241.8203

exp Services Inc.
t: +1.613.688.1899 | f: +1.613.225.7330
2650 Queensview Drive, Unit 100
Ottawa, ON K2B 8H6
Canada
www.exp.com
• BUILDINGS • EARTH & ENVIRONMENT • ENERGY • INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

BASEPLAN	AGJ
DESIGN	AKJ
CHECKED	AA
CAD	AGJ
PROJECT MANAGER	AJ
APPROVED	AA

PROJECT: IBPS FO GUAN SHAN TEMPLE
6688 FRANKTOWN ROAD
OTTAWA, ONTARIO.

TITLE: SITE GRADING PLAN

PROJECT No.: OTT-22027645-A0
SURVEY: McINTOSH PERRY
DATE: 25/08/23
DRAWING No.: C200

D07-??-??-????

#?????

Filename: e:\ott\ott-22027645-a0\60 execution\65 drawings\ott-22027645-a0 - civil.dwg Last Saved:12/18/2024 9:55:31 AM Last Plotted:12/18/2024 4:29:33 PM Pen Table: exp-ott-pens.ctb Plotted by: JariwalaA References: XREF - SITEPLAN.dwg; XREF - SURVEY.dwg

APPENDIX D: SEISMIC HAZARD CALCULATION

Government of Canada Gouvernement du Canada

2020 National Building Code of Canada Seismic Hazard Tool

This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_D
Latitude (°)	45.178
Longitude (°)	-75.864

Please select one of the tabs below.

NBC 2020 | Additional Values | Plots | API | Background Information

The 5%-damped spectral acceleration (S_a(T,X), where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$
0.575	0.491	0.293	0.141	0.0392	0.0123	0.338	0.341

The log-log interpolated 2%/50 year $S_a(4.0, X_D)$ value is : **0.0535**

▼ Tables for 5% and 10% in 50 year values

NBC 2020 - 5%/50 years (0.001 per annum) probability

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$
0.382	0.327	0.185	0.0852	0.022	0.00678	0.233	0.21

The log-log interpolated 5%/50 year $S_a(4.0, X_D)$ value is : **0.0306**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$
0.264	0.223	0.121	0.0538	0.013	0.00398	0.163	0.135

The log-log interpolated 10%/50 year $S_a(4.0, X_D)$ value is : **0.0184**

Download CSV

Go back to the seismic hazard calculator form

Date modified: 2021-04-06

APPENDIX E: REVISED REPORT

6688 FRANKTOWN – GEOTECHNICAL REPORT (REVISED)

Project No.: CP-17-0503

Prepared for:

Bing Professional Engineering Inc.
248 Huntsville Drive
Ottawa, ON K2T 0C2

Prepared by:

McIntosh Perry
115 Walgreen Rd, R.R. 3
Carp, ON K0A 1L0

REVISED
January 2019

McINTOSH PERRY

TABLE OF CONTENTS

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 a layer of black organic soil 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 with no provision for 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 L-Shaped building beside the main prayer facility. This building is approximately 635 m² at the base. The main building and the L-Shaped building will be connected by an elevated covered link;
- A one storey building without basement proposed at a distance from the northeast of the main building with approximately 350 m² footprint.

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-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 Topsoil

A 0.2 m to 0.3 m layer of topsoil (containing peat) was present at the top of all boreholes and the most of the property as observed. 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 the depth 0.3 m below ground surface. Water level readings of the wells were taken on June 15, 2018, water levels were as shown in the table below. It should be noted that the monitoring wells are in different locations than geotechnical boreholes. The locations of both geotechnical boreholes and monitoring wells can be seen in Figure 2 (attached).

Table 5-2: Groundwater Levels

Borehole	BH Elev. (m)	Water Level Reading (m)	Groundwater Elev. (m)
MW18-1	100.900	1.532	99.368
MW18-2	100.680	1.327	99.353
MW18-3	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, beyond what is discussed in this report, 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 a layer of organic soil. 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

There were several discussions with structural engineers prior to submission of this revised report. Given the complexity of geotechnical conditions on site, different foundation solutions might be employed for each segment of the design.

Knowing the groundwater is relatively very high (close to the surface) or it can reach close to the surface in certain times of the year, permanent dewatering may not be practical. Due to lack of natural topographical features on site, the only possible dewatering method is through constant pumping, this solutions is impractical since; a) due to high permeability of the sand the discharge rate will be relatively high; b) constant pumping is expensive both in terms of energy consumption and establishment of pumping facilities and back up systems; c) in case of power outage or pump failure or back up failure, building may become subject to irreversible damages. Therefore, foundation system solutions should look at possibilities of minimizing the uplift forces on building elements and reducing the building's vulnerability to water seepage such as avoiding basements. Therefore, it was decided not to include basement.

Deep foundations such as caissons and shallow foundation such as spread footings, strip footings, and raft footings can be used for the design under certain conditions as described through the following sections.

As noted in the geotechnical report and as shown in the borehole logs, what encountered in the drilled boreholes indicated the upper 2.5 m of the existing overburden consists of loose to very loose sand. To achieve the above noted design objective and in order to construct the interim building on strip and spread footings, these recommendations shall be followed;

To achieve the bearing capacities as noted in the spread and strip footing design section on loose to compact native sand, the site shall be excavated to minimum 2.5 m below existing surface. A geotechnical staff shall attend the site to confirm the subgrade, excavation may need to be advanced to a lower depth. Both OPSS Granular A or Granular B Type II are suitable to be used as engineered fill. Once the subgrade is approved, granular fill shall be placed in lifts not thicker than 300 mm when loose, and to be compacted to minimum 100% Standard Proctor Maximum Dry Density (SPMDD). A minimum 900 mm of compacted engineered fill is needed to support the footings. Compaction specification of engineered fill for areas beyond the influence zone of the footings can be reduced to minimum 98% SPMDD. The influence zone of the footings is defined by straight line going downward and outward from the outside edge of the footing at a 1H:2V slope.

OPSS.MUNI 1010 shall be referenced for material used as engineered fill. Quality Control for placement of engineered fill can reference City of Ottawa Special Provision – General No. D-029 or as approved by the geotechnical engineer.

There is also a provision of design with helical pile systems which seems feasible for this site. A specialty contractor shall provide stamped drawings for the design and installation of helical piles. A pile testing program shall be carried as per required by the building code.

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 Seismic Site Classification

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).

For the project site, hydrostatic buoyancy effect cannot be reduced/removed. Dewatering will be limited to stormwater management and the groundwater table cannot be lowered (for practical and permitting reasons). This reduction in effective stress shall be considered for evaluation of liquefaction potential.

Based on OBC 2012 a PGA of 0.32g was used for liquefaction calculations (it should be noted the Federal hazard maps of 2010 indicate PGA of 0.301g for 2500 years return period). That results in seismic stress ratio to 0.05. Foundation soil will be still acceptable in terms of liquefaction potential for the depth of approximately 2.5 (to 3 m) as previously mentioned with approximately 60 kPa overburden load. However, at upper layers (top 2.5 m) of the soil with less SPT 'N' values, less overburden pressure, high water table (not lowered) and the noted seismicity, cyclic stress ratio may exceed cyclic resistance.

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 D;

Table 6-1: Selected Seismic Spectral Responses (2% in 50 Yrs)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.578	0.288	0.131	0.044	0.301

For design of those building element supported on raft slab for the design of the specific building proposed and mentioned in Section 2 for the proposed development; What is noted below is quoted from the building code Section 4.1.8.4.C. (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.”

Given the fundamental period of the building is less than 0.5, liquefaction can be ignored and **Site Class E** can be used for the design of all buildings founded on shallow raft slabs. For other building structures which may be founded on either deep foundation or below the existing 2.5 m depth **Site Class D** can be used. That includes shallow footings on engineered fill with removal of top 2.5 m, caissons, and helical piles.

6.5 Foundation Design Options

Both deep and shallow footings are viable foundation options for this project. However, the preferred footing option, especially for the main prayer facility is deep foundations on caissons. The following benefits can be considered for design with caissons;

- Excavation for a large building such as the main prayer facility demands removal of noticeable amount of soil, the cut has to be protected by sheet piles due to high water table and loose sand, all of this will be very costly;
- The subgrade has to be dried our (at least 1 m below the subgrade) to allow for granular backfill and construction of footings. A demanding dewatering is expected due to volume of the excavation and high hydraulic permeability of sand. Deep foundation solutions can eliminate/reduce temporary dewatering needs;
- Deep foundations can also transfer the loads directly to the bedrock. Design for caisson installation does not need a previous knowledge of accurate elevation of rock surface at a given location, the casing can be lowered as much as needed to reach the rock surface;
- Caissons can be also anchored in rock. Even in case of hydrostatic pressure, anchored caissons can resist an uplift on the main structure;

6.5.1 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

6.5.2 Raft Footings

It is understood raft footings 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 a layer of 300 mm of OPSS Granular A compacted to 100% SPMDD underlain by the native subgrade. The subgrade shall be approved by geotechnical staff and it shall be free from organic and deleterious material it shall remain undisturbed from the time excavated until covered with granular fill. 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.

Table 6-4: Bearing Capacity Values for Raft Slab

Design	Value (kPa)
SLS	55
ULS	300

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 building will remain constantly heated over cold season, insulation can be installed on the exterior face and project beyond the footing equal to the soil cover deficit. In this case the insulation does not need to be load bearing. Manufacturer catalogue shall be consulted for the equivalent insulation value.

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).

6.5.3 Footings and Caissons on Rock

As per previous discussions, there is an option to design the main prayer facility and the detached L-Shaped building on deep foundations, on caissons on rock. This is the preferred approach since the two-storey L-Shaped 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 at the rock surface. Serviceability Limit State is not applicable for footings placed on rock and considering expected conventional loads.

Soil improvement options such as rammed aggregate piers supporting strip footings and spread footings are not discussed in this report. More information can be provided upon request.

The following capacities can be used for the reinforced concrete caisson design;

Table 6-5: Caisson Capacity

Caisson Dia. (m)	Capacity (kN)
0.4	50
0.6	150
0.8	250
1	400
1.2	600

Caissons shall be socketed into the rock for at least 300 mm. If there are concerns regarding uplift resistance due to expected buoyancy forces acting on the underside of the building (hypothetical), the uplift can be resisted by bell toe caissons or rock anchors within the caissons. Rock anchors if used in the design, are to be sized and specified by specialty contractor.

6.5.4 Caisson Lateral Capacity

There were two cases considered for lateral resistance of a single caisson socketed a minimum 0.3 m to 0.5 m into the rock, considering the upper layer (2.5 m below existing surface) has a potential for loss of resistance in a seismic event, therefore the lateral resistance of the upper layer may be ignored. The provided capacity values can be used for ultimate capacity check against factored loads.

Broms method was used for calculation of lateral capacity. Values offered in Table 6-6 are for lateral loads applied at or near the ground surface and assume the upper 2.5 m is ignored, and the caisson is implanted as a minimum 0.3 m to 0.5 m into the rock.

Lateral capacities are not provided for serviceability loads (or SLS design) as those capacities, considering a drained condition in absence of seismicity, are larger than ultimate design capacities, therefore they may not govern the design in any load combination.

Table 6-6: Single Caisson Unfactored Ultimate Lateral Resistance

Caisson Dia. (m)	Lateral Capacity (kN)
0.4	9
0.6	32
0.8	75
1	150
1.2	250

Following parameters are used in calculation of lateral capacity;

Soil Bulk Density γ = 20 kN/m^3 (saturated soil below 3.5 m)

Effective Internal Friction Angle ϕ'= 25°
Coefficient of Passive Pressure K_P = 2.5

6.6 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 6-7: 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

Following coefficients as shown in Table 6-8 can be used to calculate lateral pressure on structural elements. Seismic lateral pressure coefficients are calculated based on PGA of 0.32g.

Table 6-8: Static and Dynamic Lateral Pressure Coefficients

Material Type	φ'	Static Active K_a	Static Passive K_p	Static App. Ht. from Base	Dynamic Active K_{aE}	Dynamic Passive K_{pE}	K_{aE} App. Ht. from Base	K_{pE} App. Ht. from Base
Upper 3.5 m of native	22°	0.45	2.20	0.33	0.6	1.92	0.38	0.27
Below 3.5 m native	25°	0.41	2.46	0.33	0.54	2.17	0.38	0.27
OPSS Granular A	35°	0.27	3.69	0.33	0.38	3.34	0.39	0.28
OPSS Granular B	30°	0.33	3.00	0.33	0.45	2.68	0.38	0.28

The shaded data shall be used with caution or to be used only if conservative. Data might become irrelevant in case of strength loss in dynamic condition (undrained condition) when the internal friction angle temporarily tends to zero.

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.

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.

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 project, a final total of approximately 206 parking spots will be constructed on this 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. Aside from frost effects, excessive water within the pavement structure can cause softening and damage under traffic loads in warm temperatures.

It is understood the access road to the interim building will be most possibly constructed before completion of the project. Asphalt surface won't be placed as it will be damaged during construction. For the interim use, granular layers of pavement structure as shown in Table 9-2 of the final report can be constructed without asphalt binder and surface. However, the base layer (150 mm GA) is expected to be damaged over the winter and during proposed construction activities. When it comes to placing the asphalt binder, the base layer shall be shaved, granular B Type II subbase shall be repaired and recompacted at the surface, granular A base shall then be reconstructed to receive the asphalt layers.

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.

Juli Ushey, EIT
Geotechnical Engineering Intern

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

12.0 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

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.

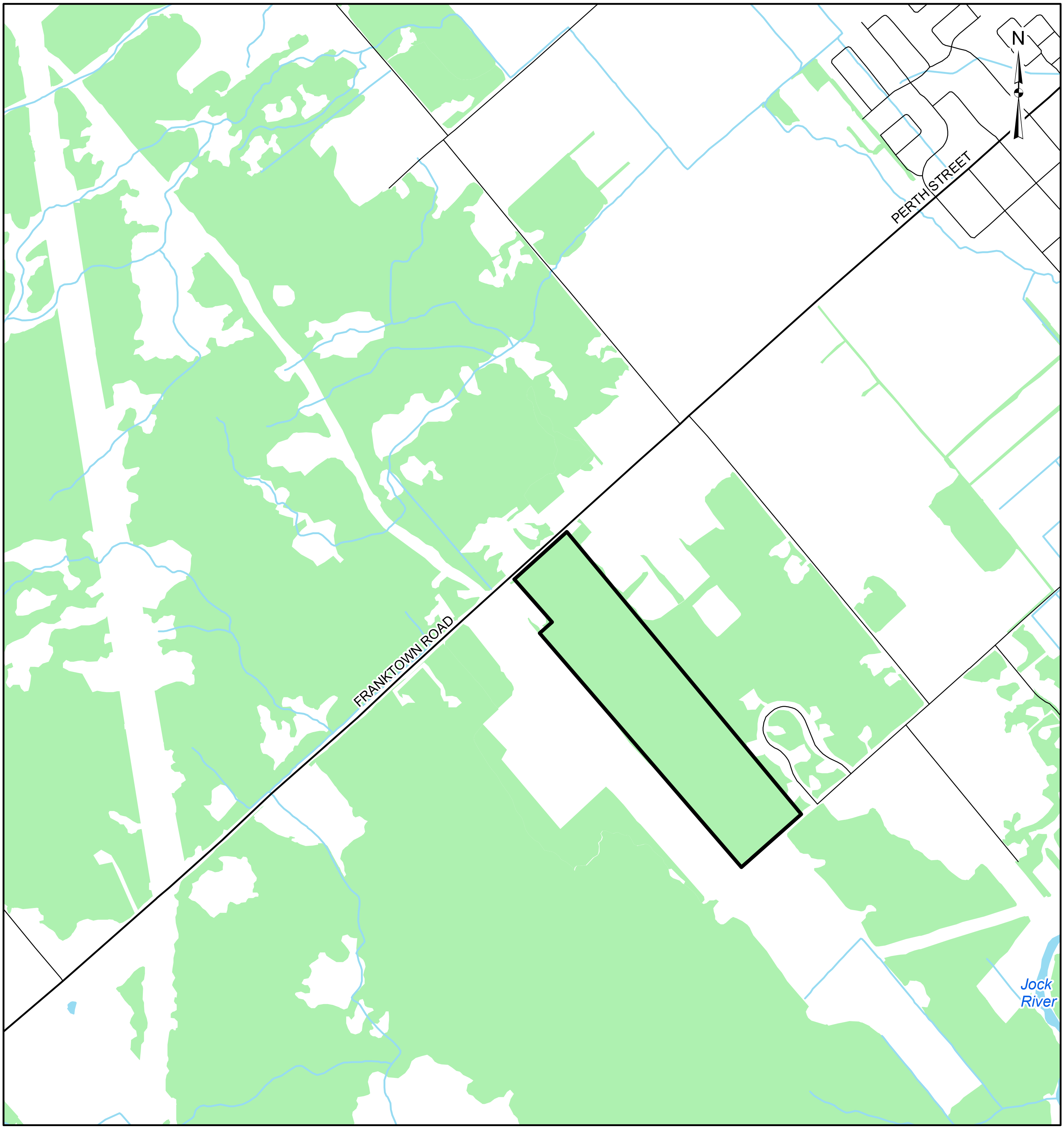
McIntosh Perry prepared this report for the exclusive use of the Client. Any use which a third party makes of this report, or any reliance on or decision to be made based on it, are the responsibility of such third parties. McIntosh Perry accepts no responsibility and will not be liable for damages,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

115 Walgreen Road, R.R.3. Carp, ON K0A 1L0 | T. 613-836-2184 | F. 613-836-3742
info@mcintoshperry.com | www.mcintoshperry.com

6688 FRANKTOWN ROAD

APPENDIX B
FIGURES

McINTOSH PERRY



600 300 0 600

Scale 1:20,000 Metres

LEGEND

Local Road
Major Road
Wooded Area
Watercourse
Waterbody

REFERENCE

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

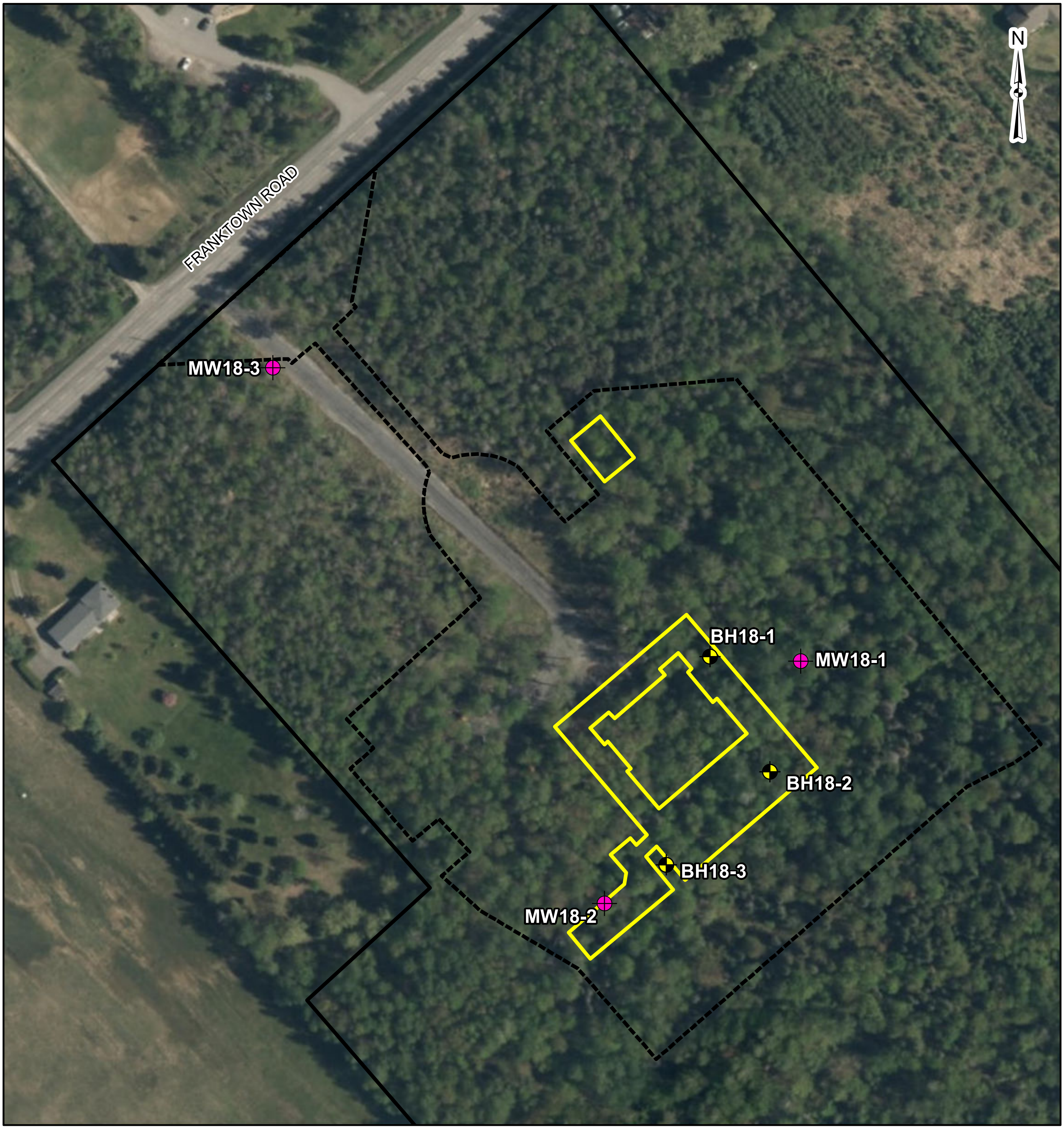
CLIENT: BING PROFESSIONAL ENGINEERING

PROJECT: GEOTECHNICAL INVESTIGATION 6688 FRANKTOWN ROAD

TITLE: SITE LOCATION

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

PROJECT NO:CP-17-0503		FIGURE:
Date	Jun. 01, 2018	1
GIS	JD	
Checked By	MG	



40 20 0 40

Scale 1:2,000 Metres

LEGEND

- Borehole Location
- Monitoring Well
- Approximate Property Boundary
- Proposed Development
- Proposed Building Footprint

REFERENCE

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

CLIENT: BING PROFESSIONAL ENGINEERING

PROJECT: GEOTECHNICAL INVESTIGATION 6688 FRANKTOWN ROAD

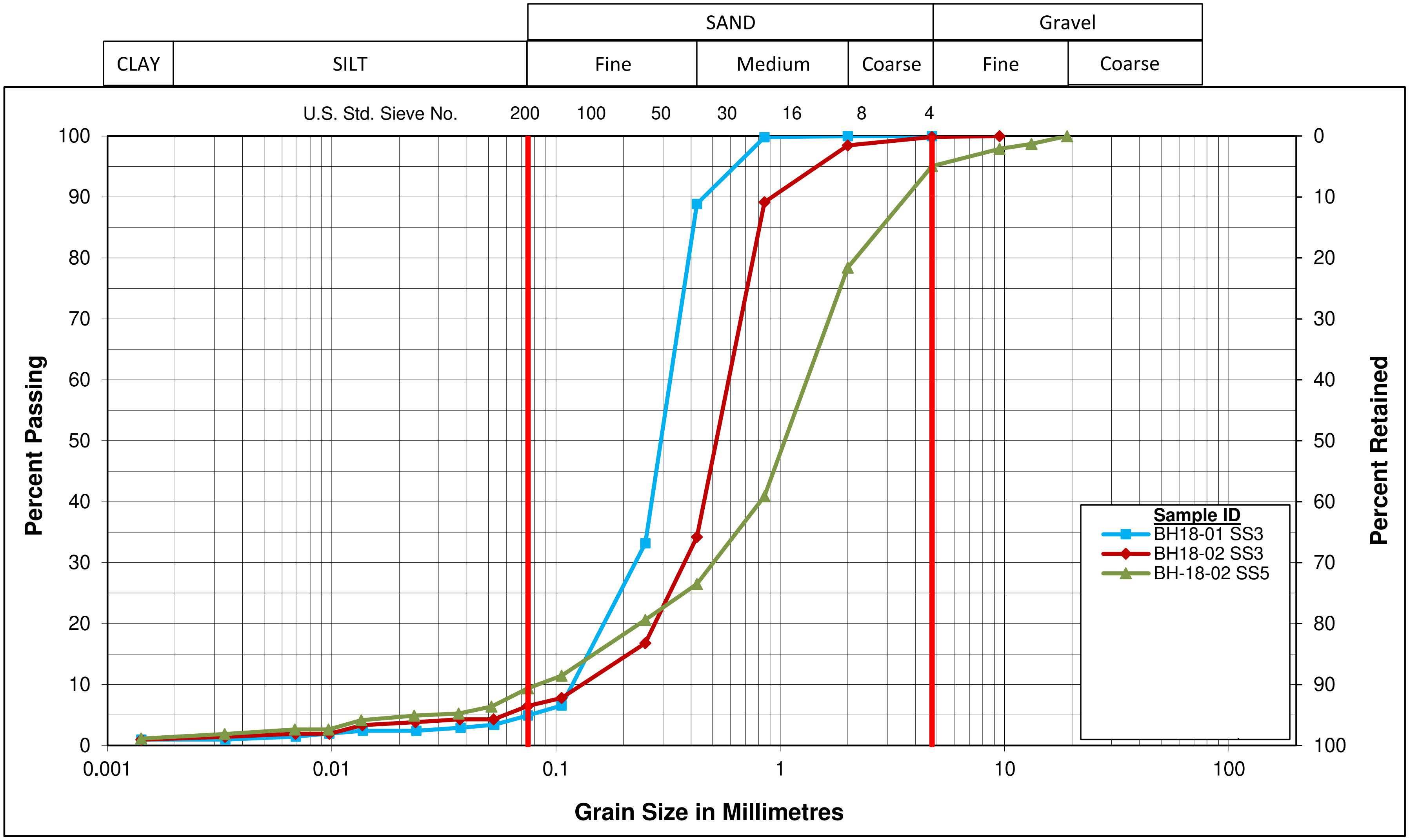
TITLE: BOREHOLE LOCATIONS

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

PROJECT NO:	CP-17-0503
Date	Jan. 22, 2019
GIS	LC
Checked By	MG

FIGURE: 2

Unified Soil Classification System



McINTOSH PERRY	GRAIN SIZE DISTRIBUTION SAND	Figure No. 3 Project No. CP-17-0503

6688 FRANKTOWN ROAD

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
σ_l, σ_2, σ_3	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
ε_1, ε_2, ε_3	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
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 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

SOIL PROFILE

ELEVATION - m / DEPTH - m	DESCRIPTION
100.9	Natural ground surface
0.0	Silty sand, traces of gravel, brown, moist to wet, very loose. Presence of organic matter.
100.7 / 0.2	Sand, traces of clay and silt, brown to light grey, wet, loose.
97.4 / 3.5	Sand, traces of clay, silt and gravel, light grey, wet, compact to dense.
96.3 / 4.6	Limestone Bedrock, good quality, slightly weathered, grey.
93.0 / 7.9	**END OF BOREHOLE**

SAMPLES

TYPE AND NUMBER	RECOVERY	"N" or RQD	WATER CONTENT (%) W	REMARKS & GRAIN SIZE DISTRIBUTION (%) G	S	M	C
SS-01	75	5					
SS-02	67	7					
SS-03	88	2	~25	0	95	4	1
SS-04	100	10					
SS-05	100	27					
SS-06	44	49					
RC-1	100	*76*		UCS = 143 MPa			
RC-2	100	*76*					

GROUNDWATER CONDITIONS: 0.3 m

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

McINTOSH PERRY

RECORD OF BOREHOLE No 18-02

DATE: 23/05/2018 - 24/05/2018
PROJECT: CP-17-0503BING
CLIENT: Bing Professional Engineering Inc.
ELEVATION: 100.9 m

LOCATION: 6688 Franktown Road ()
COORDINATES: Lat: 45.17596968 , Lon: -75.86225266
DATUM: Geodetic
REMARK:

ORIGINATED BY: PH
COMPILED BY: MG
CHECKED BY: NT
REPORT DATE: 29/06/2018

SOIL PROFILE

ELEVATION - m / DEPTH - m	DESCRIPTION
100.7	Natural ground surface
0.0	Peat, trace silty sand, light grey, wet
100.4 / 0.3	Sand, traces of clay and silt, brown, moist to wet, very loose to loose.
97.3 / 3.4	Sand, traces of clay, silt and gravel, grey, wet, loose to compact.
95.5 / 5.2	Limestone Bedrock, good quality, slightly weathered, grey.
93.8 / 6.9	**END OF BOREHOLE**

SAMPLES

TYPE AND NUMBER	RECOVERY	"N" or RQD	WATER CONTENT (W)	G	S	M	C
SS-01	58	2					
SS-02	75	4					
SS-03	100	0	○	0	94	5	1
SS-04	42	6					
SS-05	57	20	○	5	86	8	1
SS-06	65	15					
RC-1	100	*86*					
RC-2	100	*85*					

GROUNDWATER CONDITIONS: 0.3 m

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

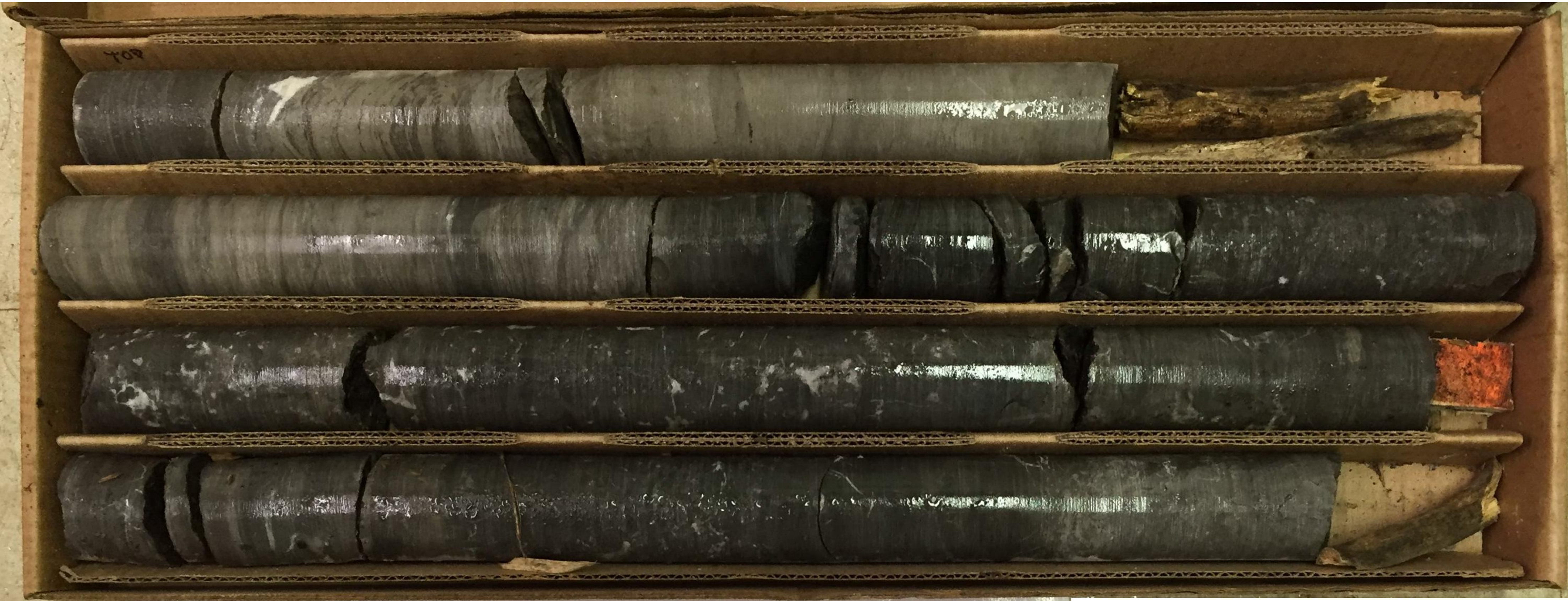
McINTOSH PERRY

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

DEPTH - feet	DEPTH - meters	ELEVATION - m / DEPTH - m	SOIL PROFILE DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	STATE	RECOVERY	"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
		101.0	Natural ground surface									
		0.0 / 100.7	Peat, trace organics (roots)									
		0.3	Sand, traces of clay and silt, light brown to grey, wet, loose to compact.		SS-01		67	2	0.3 m			
	1				SS-02			4				
5	2				SS-03			3				
					SS-04		75	10				
10	3				SS-05		83	12				
	4				SS-06		92	15				
15		96.0			SS-07		100	33				
	5	5.0	Sand, traces of clay, silt and gravel, grey, wet, dense.									
		95.3			SS-08		80	58				
		5.7	**END OF BOREHOLE**									
20	6		**Auger refusal on probable bedrock.**									
	7											
25	8											
30	9											

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



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
LABS RESULTS

PARACEL LABORATORIES LTD. | TRUSTED. RESPONSIVE. RELIABLE.

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

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	**Reference No.:**	CP-17-0503
Project:	Materials Testing	**File No.:**	170496-32
Location:	Franktown Road	**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^{c}Laughlin, Geo.Tech., C.Tech.

6688 FRANKTOWN ROAD

APPENDIX E
SEISMIC HAZARD CALCULATION

McINTOSH PERRY

2010 National Building Code Seismic Hazard Calculation

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

Requested by: , December 12, 2018

Site Coordinates: 45.178 North 75.864 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)
0.587	**0.288**	**0.131**	**0.044**	**0.301**

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)	0.082	0.229	0.361
Sa(0.5)	0.040	0.116	0.177
Sa(1.0)	0.017	0.053	0.084
Sa(2.0)	0.0058	0.017	0.027
PGA	0.033	0.112	0.187

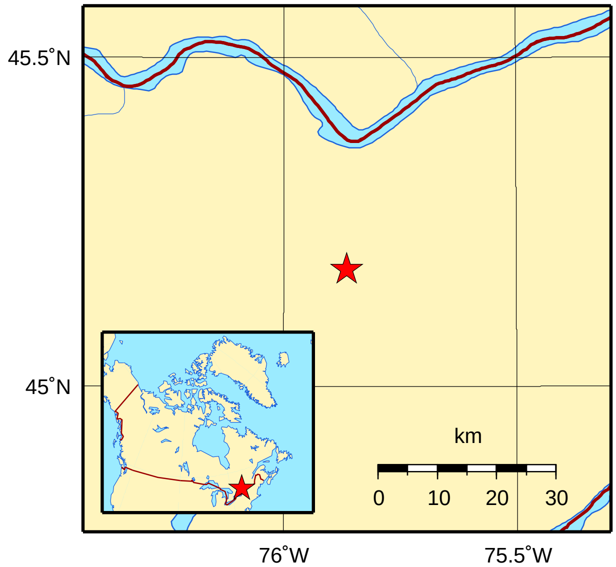
References

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