ST. MARY COPTIC CHURCH PROPOSED DEVELOPMENT

GEOTECHNICAL INVESTIGATION

ST. MARY COPTIC CHURCH

GEOTECHNICAL REPORT

PROJECT NO.: 191-04634-00
DATE: DECEMBER 2019

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TABLE OF CONTENTS

TABLES

APPENDICES

1 INTRODUCTION

1.1 CONTEXT

WSP Canada Inc. (WSP) was retained by St. Mary Coptic Orthodox Church to conduct a geotechnical investigation at 1 Canfield Road and the seven surrounding properties, in Ottawa, Ontario as shown in Drawing No. 1 in Appendix A.

The scope of work for this investigation is outlined in WSP's Proposal, dated January 9, 2019 and subsequent project correspondence.

The purpose of the geotechnical investigation was to obtain subsurface information at the site by means of an exploratory borehole. This report presents the findings of the investigation and provides comments and recommendations which may affect the design and construction of the proposed development. The Phase 1 Environmental Site Assessment (ESA) is provided under a separate cover.

1.2 PROJECT AND SITE DESCRIPTION

The Site is located in the northeast corner of the Canfield Road and Greenbank Road intersection, in the City of Ottawa and consists of seven municipal properties; 1 and 9 Canfield Road, and 9, 11, 13, 15, and 17 Parkmount Crescent. A site location map is provided as Drawing No. 1 in Appendix A.

The Site is irregular in shape, with frontage on both Canfield Road and Parkmount Crescent, and is approximately 0.95 ha in plan area. Currently, the property at 1 Canfield Road is occupied by the St. Mary Coptic Orthodox Church, while all other properties are occupied by residential dwellings.

Our understanding is that the project will be undertaken in two phases. Phase 1 is the construction of a community building to be constructed north of the existing church building. Phase 2 will consist of demolishing the existing church building and reconstruction of a church building that will be larger than the existing church. Based on conversations with the client it is understood that basement is being considered for the proposed structures. It is also understood that the properties at 9 Canfield Road, and 9, 11, 13, 15, and 17 Parkmount Crescent are to be demolished in order to provide space for a parking lot with approximately 100 spaces. It is not known at this time how many storeys are proposed for the new structures.

1.3 OBJECTIVES AND LIMITATIONS

The current report was prepared at the request and for the sole use of the St. Mary Coptic Orthodox Church according to the specific terms of the mandate given to WSP. The use of this report by a third party, as well as any decision based upon this report, is under this party's sole responsibility. WSP may not be held accountable for any possible damages resulting from third party decisions based on this report.

Furthermore, any opinions regarding conformity with laws and regulations expressed in this report are technical in nature; the report is not and shall not, in any case, be considered as a legal opinion.

Information in this report is only valid for the borehole locations as described.

Reference should be made to the Limitations of this Report, attached in Appendix D, which follows the text but forms an integral part of this document.

2 SITE INVESTIGATION

2.1 SCOPE OF WORK

The scope of work for this assignment included:

- A desk study and review of existing geotechnical information in the general area;
- Laying out of boreholes and obtaining utility locates at the project site;
- Drilling of exploratory boreholes;
- In-situ soil sampling and testing, including Standard Penetration Testing (SPT);
- Obtaining soil samples for additional review and laboratory testing;
- Laboratory testing;
- Geotechnical analysis; and
- Preparation of this report which presents the results of the investigation and provides geotechnical recommendations related to the design and construction of the proposed development.

2.2 INVESTIGATION PROCEDURES

The geotechnical investigation was carried out in April and May 2019.

2.2.1 DESKTOP STUDY

Surficial geology maps indicate that the area is underlain offshore marine deposits consisting of silt and clay, with minor amounts of sand and gravel. This deposit is underlain by deltaic and estuarine deposits of Champlain Sea Sediments consisting of medium to fine grained sand. Bedrock geology maps indicate the bedrock in the general area consists of dolostone and limestone from the Oxford Formation.

2.2.2 FIELD INVESTIGATION

The field investigation was carried out on April 22 and May 2, 2019 and included the drilling of four boreholes, BH19-1 thru BH19-4, within the footprint of the proposed buildings.

The boreholes were advanced using a track-mounted drill rig supplied and operated by Ohlmann Geotechnical Services (OGS) of Almonte, Ontario. The boreholes were advanced using hollow-stem augers to depths ranging from 3.7 metres (m) to 9.8 m below the existing ground surface. Soil samples retrieved during drilling were logged and visually classified in the field by a member of WSP's geotechnical staff.

The borehole locations are shown on Drawing No. 2 in Appendix A. The borehole logs are included in Appendix B of this report.

2.2.3 LABORATORY TESTING

Upon completion of drilling and in-situ testing, soil samples were returned to WSP's laboratory for further examination, classification and testing. The testing program consists of the determination of natural water content, grain size distribution, Atterberg limits (Plasticity) and chemical analyses of soil corrosivity (sulphate content, chloride content, pH, and resistivity).

The results of natural water content tests are included on the relevant borehole logs in **Appendix B**. The results of determination of grain size distribution are summarized on the individual borehole logs and are presented in **Appendix C**. The results of Atterberg limits (Plasticity) are summarized on the individual borehole logs and presented in **Appendix C**. Chemical testing to determine sulphate content, chloride content, pH and resistivity was also carried out on selected soil samples obtained during drilling. The results of these tests are included in **Appendix C**.

3 SUBSURFACE GEOTECHNICAL CONDITIONS

The subsurface conditions encountered within the boreholes are discussed in the following sections. Detailed descriptions of the soil and groundwater conditions encountered at the borehole locations are included in the individual borehole logs in Appendix B.

3.1 SOIL CONDITIONS

3.1.1 PAVEMENT STRUCTURE

An asphaltic flexible concrete pavement structure was encountered at the surface of all boreholes. The pavement structure encountered consists of hot mix asphalt supported by a granular base. The asphalt thickness was found to be 30 mm in all the boreholes drilled at the site. Supporting the asphalt surface was a granular base consisting of crushed sand and gravel. The thickness of the granular base ranged from 270 mm to 320 mm and extended to depths ranging from 300 mm to 350 mm below the existing ground surface.

3.1.2 FILL

A layer of fill was encountered underlying the granular base in all the boreholes. This layer of fill extended to depths ranging from 1.0 m to 1.7 m below the existing ground surface.

In boreholes BH19-1 and BH19-3 this layer of fill consisted of silty sand with trace to some clay. The SPT “N” values within this layer of fill ranged from 5 blows to 12 blows per 305 mm of penetration indicating a loose to compact state of packing.

The grain size distribution for one selected sample of granular portion of the fill is presented in Appendix C. A summary of this grain size distribution is also presented in the table below.

Table 3.1 Results of Grain Size Analyses for Fill

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Fines (Silt and Clay)
BH19-2	SS-2	2	54	44

The natural moisture content of this sample of the fill was 24 percent.

In boreholes BH19-2 and BH19-4 the layer of fill consisted of silty clay. The two SPT “N” values within the silty clay fill were 7 blows and 8 blows per 305 mm of penetration indicating a firm to stiff state of packing.

3.1.3 SILTY CLAY

In boreholes BH19-1 and BH19-3, a layer of native silty clay was encountered underlying the silty sand fill. This deposit extended to a depth of 4.7 m in boreholes BH19-1 and BH19-3. This deposit was not encountered in boreholes BH19-2 and BH19-4.

SPT “N” values within the silty clay ranged from 6 blows to 14 blows per 305 mm indicating a firm to stiff consistency.

The results of Atterberg limit testing, carried out on a selected sample of the silty clay gave a liquid limit value of 33 percent and a plasticity index of 18 percent, indicating a low plasticity clay soil. The measured water content of one sample of the silty clay was 27 percent, which is below the liquid limit of this sample.

3.1.4 SILTY SAND

A layer of native silty sand with trace clay was encountered underlying the silty clay fill in boreholes BH19-2 and BH19-4 and underlying the native silty clay in boreholes BH19-1 and BH19-3. This silty sand deposit extended to the depth of drilling, ranging from 3.7 m to 9.8 m below the existing ground surface.

SPT "N" values within the silty sand ranged from 12 blows per 305 mm of penetration and greater than 50 blows per 50 mm of penetration, indicating a compact to very dense state of packing.

Table 3.2 Results of Grain Size Analyses for Silty Sand

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
BH19-1	SS-8	0	72	21	7

The natural moisture content of one sample of the silty sand was 14 percent.

3.2 BEDROCK CONDITIONS

Neither bedrock nor auger refusal were encountered in the boreholes drilled at the Site. Bedrock is therefore inferred to be more than 9.8 m below the existing ground surface.

3.3 GROUNDWATER CONDITIONS

Piezometers were installed in boreholes BH19-1 and BH19-3 during the field investigation to allow for subsequent observations of the groundwater levels. The groundwater levels within the piezometers were measured on May 9th, 2019, sixteen days and seven days after the well installations for boreholes BH19-1 and BH19-3, respectively. The following are the results.

Table 3.3 Measured Groundater levels

Borehole No.	Groundwater Depth
BH19-1	7.6
BH19-3	7.4

These piezometers have been left in place after this investigation and should be properly decommissioned by others during construction.

3.4 SUMMARY

A summary of the soil and groundwater conditions encountered at the site is presented in the table below.

Table 3.4 Simplified Stratigraphy and Groundwater Depths

Borehole No. (Elev. m)	Simplified Stratigraphy (Depth in metres)					Measured Groundwater Depth (m) (Elev. m)	Notes
	Asphaltic Concrete	Granular Base	Fill	Silty Clay	Silty Sand		
BH19-1 (89.0)	0 - 0.03	0.03 - 0.3	0.3 - 1.7	1.7 - 4.7	4.7 - 9.8	7.6 (81.4)	Borehole terminated at 9.8 m in depth
BH19-2 (88.9)	0 - 0.03	0.03 - 0.3	0.3 - 1.7	--	1.7 - 9.8	--	Borehole terminated at 9.8 m in depth
BH19-3 (89.0)	0 - 0.03	0.03 - 0.35	0.35 - 1.0	1.0 - 4.7	4.7 - 9.8	7.4 (81.6)	Borehole terminated at 9.8 m in depth
BH19-4 (89.2)	0 - 0.03	0.03 - 0.3	0.3 - 1.7	--	1.7 - 3.7	--	Borehole terminated at 3.7 m in depth

4 RECOMMENDATIONS

4.1 GENERAL

This section of the report provides engineering guidance related to the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities. Reference should be made to the Limitations of this Report, attached in Appendix D, which follows the text but forms an integral part of this document.

The general subsurface conditions encountered in the boreholes include an asphaltic concrete pavement structure overlying fill consisting of either silty sand or silty clay. In boreholes BH19-2 and BH19-4 the fill overlies a layer of native silty sand. In boreholes BH19-1 and BH19-3 a layer of native silty clay was encountered underlying the fill and is overlying the native silty sand. Neither bedrock nor auger refusal were encountered and therefore the bedrock elevation is inferred to be below 9.8 m in depth.

4.2 SEISMIC CONSIDERATION

4.2.1 LIQUEFACTION POTENTIAL

The soils at the site are not considered to be susceptible to seismic liquefaction based on the soil type, the SPT N values encountered within these soils and the groundwater level observed at the site.

4.2.2 SEISMIC SITE CLASSIFICATION

As outlined in the 2012 Ontario Building Code, building foundations must be designed to resist a minimum earthquake force. In accordance with Table 4.1.8.4.A of the 2012 Ontario Building Code, the seismic site response for foundations placed on either engineered fill or compact to very dense silty sand would have a site classification of Class D.

4.3 GRADE RAISE

It is understood that no grade raise is proposed at the Site.

4.4 FOUNDATIONS

It is understood that a basement is being considered for the proposed development and therefore excavations would extend to an approximate depth of 3.5 m and therefore be founded on either the engineered fill, native silty sand or the native silty clay.

For a foundation with a minimum width of 1.0 m placed on native silty sand, silty clay or engineered fill (extending to the native material) the following resistances may be assumed:

- The unfactored ultimate geotechnical bearing resistance can be taken as 500 kPa. A resistance factor of 0.5 should be applied to this value, yielding a factored bearing resistance of 250 kPa at ULS (Ultimate Limit States).
- The geotechnical resistance at the Serviceability Limit State (SLS) can be taken as 100 kPa.

It should be noted that placing foundations on different materials (such as silty sand or silty clay) may results in differential settlement. Foundations should either be placed all on the same material type, or the structure designed such that different sections can move independent of each other.

For a foundation with a minimum width of 1.0 m placed on the native silty sand or on engineered fill which itself is placed on the native silty sand the following resistances may be assumed:

- The unfactored ultimate geotechnical bearing resistance can be taken as 600 kPa. A resistance factor of 0.5 should be applied to this value, yielding a factored bearing resistance of 300 kPa at ULS (Ultimate Limit States).
- The geotechnical resistance at the Serviceability Limit State (SLS) can be taken as 160 kPa.

4.5 FROST PROTECTION

Foundations for heated structures should be protected against frost with a minimum of 1.5 m of earth cover or the thermal equivalent if insulation is used. Foundations for unheated structures should be provided with a minimum of 1.8 m of earth cover or the thermal equivalent if insulation is used.

In the event that foundations are to be constructed during the winter months, foundation soils and side slopes of excavations are required to be protected from freezing temperatures immediately upon excavation and exposure to sub-zero temperatures until such time as heat can be applied to the building or the foundations have sufficient earth cover to prevent freezing of subgrade soils.

4.6 SLABS-ON-GRADE

In preparation for the construction of the basement floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab. Provision should be made for at least 200 millimetres of Ontario Provincial Standard Specification (OPSS) Granular A to form the base of the floor slab. Any bulk fill required below the underside of the Granular A should consist of OPSS Granular B Type II. The under-slab fill should be placed in maximum 300-millimetre thick lifts. The required degree of compaction is discussed in section 4.11.

All subgrades should be reviewed by WSP prior to placement of any geotextile, granular base, concrete, etc.

4.7 LATERAL EARTH PRESSURES

Lateral Earth Pressure

The lateral earth pressure acting on retaining walls, etc. may be calculated using the following expression:

$$P = K(\gamma h+q)$$

Where: P = lateral earth pressure (kPa) acting at depth h

K = earth pressure coefficient; for unrestrained walls and structures where some movement is acceptable (such as retaining walls) use a coefficient of active earth pressure (K_a) equal to 0.3, for restrained walls (such as basement walls) use the coefficient of earth pressure at rest (K_0) equal to 0.5

γ = the density of the backfill; use 21.5 kN/m^3 for compacted granular backfill, 19 kN/m^3 for native silty sand or silty clay

h = the depth to the point of interest (m)

q = the magnitude of any design surcharge at the ground surface;

The above values assume free-draining granular backfill will be used. If this is not the case then the above values may need to be adjusted based on the soil type used, and water pressures should be considered in the calculation of lateral pressures. WSP can provide additional guidance based on actual building plans if required.

Seismic Earth Pressure

Earth pressures will be higher under seismic loading conditions. In order to account for seismic earth pressures the total earth pressure during a seismic event (including both the seismic and static components) may be assumed to be:

$$\sigma_h(z) = K_a \gamma z + (K_{AE} - K_a) \gamma (H\text{-}z)$$

Where: $\sigma_h(z)$ = the total earth pressure at depth z (kPa);

K_a = the active earth pressure coefficient (0.3);

γ = the unit weight of soil (21.5 kN/m^3 for granular fill) or 19 kN/m^3 for native silty sand or silty clay;

K_{AE} = the combined active earth pressure and seismic earth pressure coefficient (use 0.8);

H = the total height of the wall (m)

z = the depth below the top of the wall (m)

The above earth pressure values (both static and seismic) are unfactored values.

4.8 FOUNDATION WALL BACKFILL

Foundation elements should be backfilled with either:

- non-frost-susceptible sand and/or gravel which meets the gradation requirements for OPSS Granular B Type I;
- or 19-millimetre clear crushed stone, which is separated from other soils with a Class II non-woven geotextile having an FOS not exceeding 100 microns to prevent loss of adjacent sand, or silty soils into the clear stone. It should be noted that the use of clear stone as foundation backfill may lead to unfavourable growing conditions for plant matter placed in overlying topsoil.

In areas where pavement or other hard surfacing will be in contact the building, differential frost heaving could occur between the granular fill (if sand or crushed stone is used) and other areas. To reduce this differential heaving, the backfill adjacent to the wall can be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 metres below finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The fill should be placed in maximum 300-millimetre thick lifts and compacted to the specifications in section 4.10.

To avoid damaging or laterally displacing the structures, care should be exercised when compacting fill adjacent to new structures. Heavy equipment should be kept a minimum of 1 m away from the structure during backfilling. The 1 m width adjacent to the wall should be compacted using hand-operated equipment unless otherwise authorized.

4.9 SITE SERVICES

Excavations are expected to be within the fill, silty clay or silty sand. Details of the proposed site services are not available at this time; however, it is assumed that they will include localized trenches throughout the site. Trenches can be temporarily supported using sloped excavations (see Section 4.13.2) or trench boxes.

Bedding for site services should be in accordance with the relevant OPSD standard drawing and would typically consist of Granular "A". Where wet or disturbed conditions are encountered in the base of the trench it may be necessary to over-excavate and replace unsuitable soils with compacted granular fill to provide a stable sub-grade for the bedding. The use of clear stone as a bedding and cover material is not recommended as the finer particles of the native soils and backfill may migrate into the voids of the clear stone, resulting in loss of pipe support.

Cover material above the spring line should consist of Granular "A" or Granular "B" material with a maximum particle size of 25 mm.

Backfill may consist of additional granular fill, or properly moisture conditioned silty clay. Where backfill is below paved areas (such as parking lots) and is within the frost depth, the backfill profile (above the minimum cover required) in the trench should be made to match the native soils on either side as much as is practical in order to minimize the potential for differential frost heave. As a result, portions of the silty clay above the water table may be retained, moisture conditioned (if necessary) and re-used.

Any service trenches which extend below the water table should have clay cut-offs installed across the trench at regular intervals (typically 100 m) to prevent the trench acting as a drain and lowering the groundwater table in the general area. These cut-offs should extend the full width of the trench and must completely penetrate the bedding, cover and any other granular materials in the trench.

The above are general guidelines for typical site services. All services installations should be completed in accordance with the relevant OPSS's and OPSD's for the particular application and size. WSP can provide additional review during detailed design based on the actual services proposed if required.

The required degree of compaction is discussed in section 4.11.

4.10 PAVEMENTS

4.10.1 PREPARATION FOR PAVING

The scope of the geotechnical investigation included boreholes drilled at the existing church property at 1 Canfield Road. Due to access constraints, no boreholes were drilled on the properties of 9 Canfield Road, and 9, 11, 13, 15, and 17 Parkmount Crescent. Prior to the placement of any granular materials, any existing topsoil any other deleterious material must be removed and the underlying soil proof rolled and inspected by a geotechnical engineer. Any sort of "spongey" material will need to be sub-excavated. Fill material to raise the grade or replace sub-excavated material must meet the requirements for OPSS Select Subgrade Material (SSM). This fill material should be placed in lifts not exceeding 300 mm (loose) and be uniformly compacted. The base and sub-base material are to be placed in lifts not exceeding 300 mm (loose). Both stripping and proof-rolling operations should be observed and carried out to the satisfaction of a geotechnical engineer.

It is understood that the existing structures of the above properties are to be demolished. The debris from the demolition must be removed from any area underlying the proposed parking lot. Material used to level the properties after the demolition is complete must meet the requirements of a SSM and should be placed in lifts not exceeding 300 mm (loose).

The requirements for compaction and discussed in section 4.11 below.

4.10.2 PAVEMENT RECOMMENDATIONS

Detailed traffic loads have not been provided at this time, however based on the subsoil conditions encountered, conventional asphaltic (flexible) pavement designs are considered to be appropriate for proposed paved parking areas for cars and light weight trucks, driveways and access roads. Based on the results of this investigation and experience, the following asphaltic pavement design is recommended for the indicated areas.

Table 4-1 Recommended Pavement Structures

Pavement Layer	Light Duty Roads and Parking Areas	Heavy Duty Roads (Delivery Trucks, Fire Routes, Access Roads, etc.)
Asphaltic Concrete	40 mm HL3 or SP-12.5 50 mm HL8 or SP-19.0	40 mm HL3 or SP-12.5 90 mm HL8 or SP-19.0
OPSS Granular A Base	200 mm	200 mm
OPSS Granular B Sub-Base	260 mm	400 mm

Asphalt materials and placement specifications should be in accordance with relevant Provincial standard specifications. The asphaltic cement should be PG 58-34.

4.11 BACKFILLING AND COMPACTION

Backfill for foundation excavations and any below grade structures should comprise free draining OPSS Granular "A" or "B" materials. Backfill should be placed in shallow lifts, not exceeding 200 mm loose thickness.

The existing site materials are not considered suitable for reuse as structural fill. The suitability of imported materials should be confirmed prior to placement from both a geotechnical and environmental perspective. However, the existing soils at the site are adequate for use as general earth fill but may require moisture conditioning (either wetting or drying) prior to placement and compaction.

To avoid damaging or laterally displacing the structures, care should be exercised when compacting fill adjacent to new structures or adjacent to existing retaining walls. Heavy equipment should be kept a minimum of 1 m away from the structure during backfilling. The 1 m width adjacent to the wall should be compacted using hand-operated equipment unless otherwise authorized.

The compaction requirements for OPSS Granular "A" base underlying slabs-on-grade or asphaltic concrete or OPSS Granular "B" sub-base underlying OPSS Granular "A" as part of the pavement structure should be compacted to 100% of the material's Standard Proctor Maximum Dry Density (SPMDD). Fill material underlying structural elements, supporting site services or underlying the pavement structure should be compacted to a minimum of 98% of the material's SPMDD. Bedding for site services not underlying the pavement structure or structural elements and general fill should be compacted to a to a minimum of 95% of the material's SPMDD.

4.12 CORROSION AND CEMENT TYPE

Two samples were submitted to Eurofins for testing related to soil corrosivity and potential exposure of concrete elements to sulphate attack. The results of these tests are included in **Appendix C** and summarized in table below.

Table 4-2 Results of Soil Corrosivity Testing

Borehole/ Sample No.	Soil Type	Chloride (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)	Sulphate (%)
BH19-1/SS5	Silty Clay	0.012	0.27	8.3	3700	0.02
BH19-1/SS7B	Silty Sand	0.006	0.11	8.6	9090	<0.01

The soil resistivity values suggest a low to moderately corrosive environment for buried steel elements. These values must be taken into consideration during design of below-grade steel elements.

The test result indicates a negligible soluble sulphate content and sulphate resistant Portland cement is not required.

4.13 CONSTRUCTION CONSIDERATIONS

4.13.1 CONSTRUCTION DEWATERING

The groundwater level at the site was found to be between 7.4 m and 7.6 m below the existing ground surface elevation at the time of the investigation. This corresponds to elevations ranging from 81.4 m to 81.6 m. It is expected that is the proposed structures will have a one storey basement and excavation for foundations could extend to a maximum depth of 5 m below the existing ground surface (the maximum depth the native silty sand was encountered). Based on these assumptions it is likely that seepage into the excavations can be managed using properly filtered sumps or ditches. For deeper excavations extended close to, or below the expected groundwater level additional or more complex dewatering will be required. WSP can provide additional guidance based on the size and depth of anticipated excavations, if required during detailed design.

The excavations above the observed groundwater level would not be expected to require a MOECC Environmental Activity and Sector Registration (EASR – which covers construction dewatering up to 400,000 l/day) or a Permit to Take Water (PTTW – which is required for dewatering in excess of 400,000 l/day). If substantially deeper excavations are required or construction is scheduled during wetter periods (such as the spring) then this assumption should be reviewed during detailed design.

4.13.2 TEMPORARY EXCAVATIONS

All excavations should be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). Part III of Ontario Regulation 213/91 deals with excavations.

The soils within the expected excavation include fill, native silty clay and native silty sand above the groundwater level. For preliminary planning purposes these soils can be classified as a Type 3 Soil above the groundwater table (or depth of dewatering). Excavations within Type 3 soil require side slopes with a minimum gradient of 1 horizontal to 1 vertical. Should the excavation extend below the groundwater table (or depth of dewatering), the soils would be considered to be Type 4.

If required, WSP can provide additional guidance based on preliminary excavation plans, depths, etc. during the detailed design phase of the project.

4.13.3 FOUNDATION SUBGRADE PREPARATION

The geotechnical bearing resistances provided in Section 4.4 assume that the foundation soils will not be disturbed by construction activities. Proper de-watering and protection of exposed soil subgrades will be important to the construction of the foundations. All excavated surfaces should be kept free of frost, water, etc. during the course of construction. All excavated surfaces should be inspected by a qualified geotechnical engineer who is familiar with the findings of this investigation and the design and construction of similar structures.

4.13.4 WINTER CONSTRUCTION

In the event that construction is required during freezing temperatures, the potentially frost susceptible subgrade below the footings and floor slabs should be protected immediately from freezing using straw, propane heaters, polystyrene insulation, insulated tarpaulins, or other suitable means that prevent any underlying soil from freezing.

5 CLOSURE

The Limitations of Report, as presented in Appendix D, are an integral part of this report.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

WSP Canada Inc.

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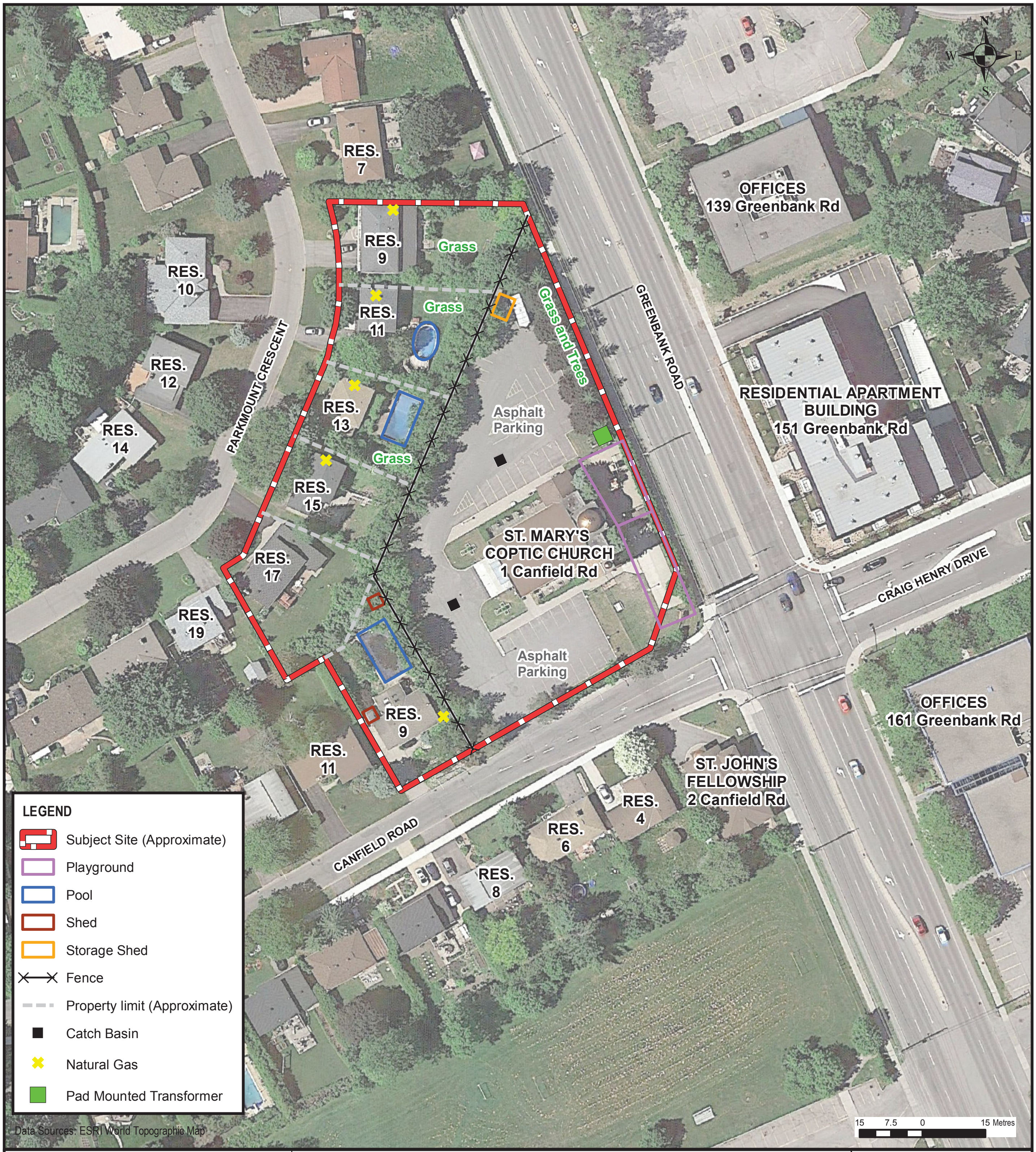
APPENDIX

A

DRAWINGS



Data Sources: ESRI World Topographic Map

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PROJECT:
ST. MARY COPTIC CHURCH

TITLE:
SITE LOCATION PLAN

CLIENT:
ST. MARY COPTIC CHURCH

SCALE:
1:1 300

DRAWN BY:
CP

CHECKED BY:
AM

PROJECT NO:
191-04634-00

DATE:
MAY 2019

FIGURE NO:
1

REV.:
-

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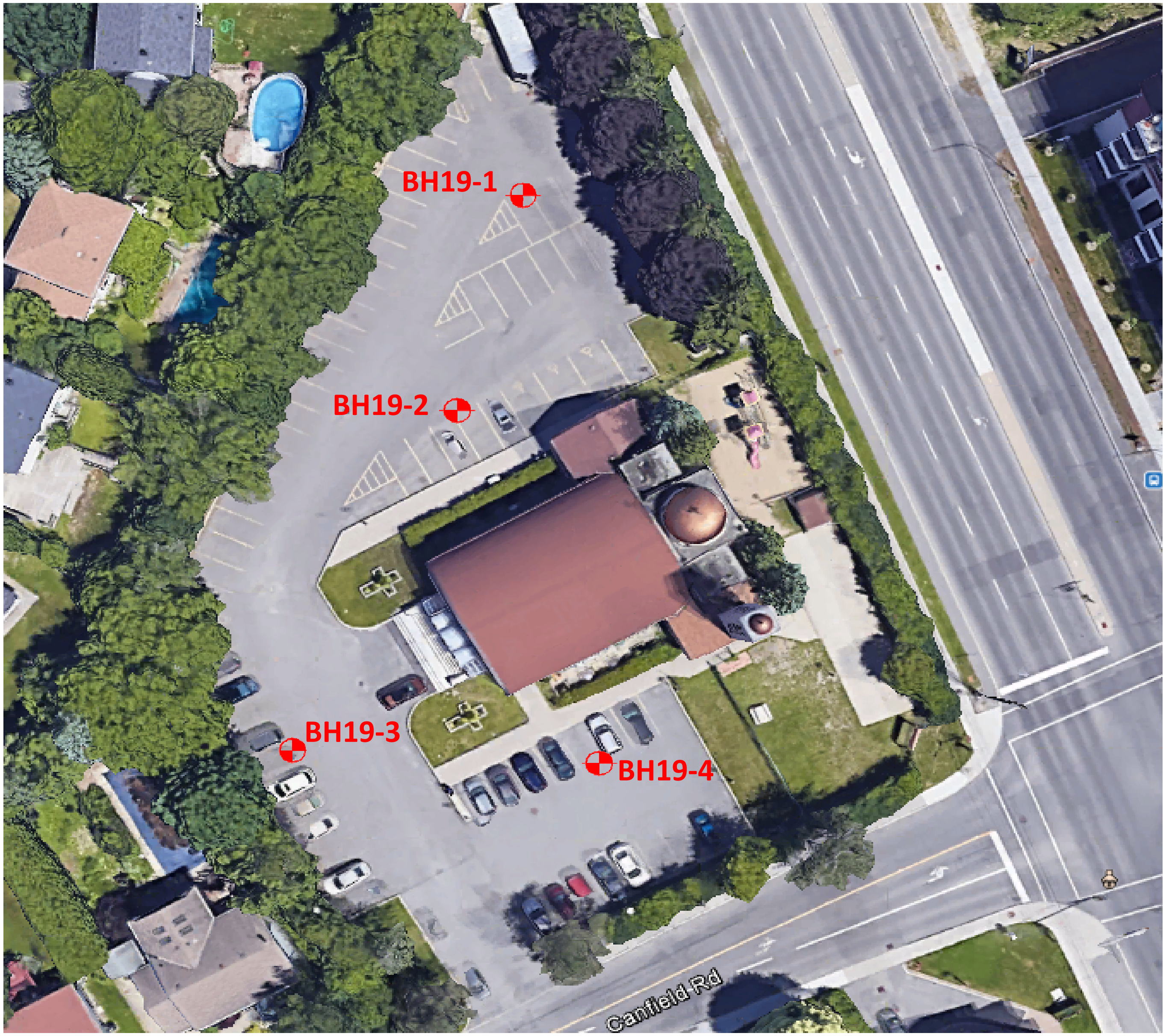


Client:	St. Mary Coptic Church		
Project#:	191-03052-00	DWG #:	2
Drawn:	DW	Approved:	ME
Date:	May 2019	Scale:	N. T. S.
Size:	Letter	Rev:	0

Title:	Borehole Location Plan
Project:	Geotechnical Investigation St. Mary Coptic Church

wsp

APPENDIX

B

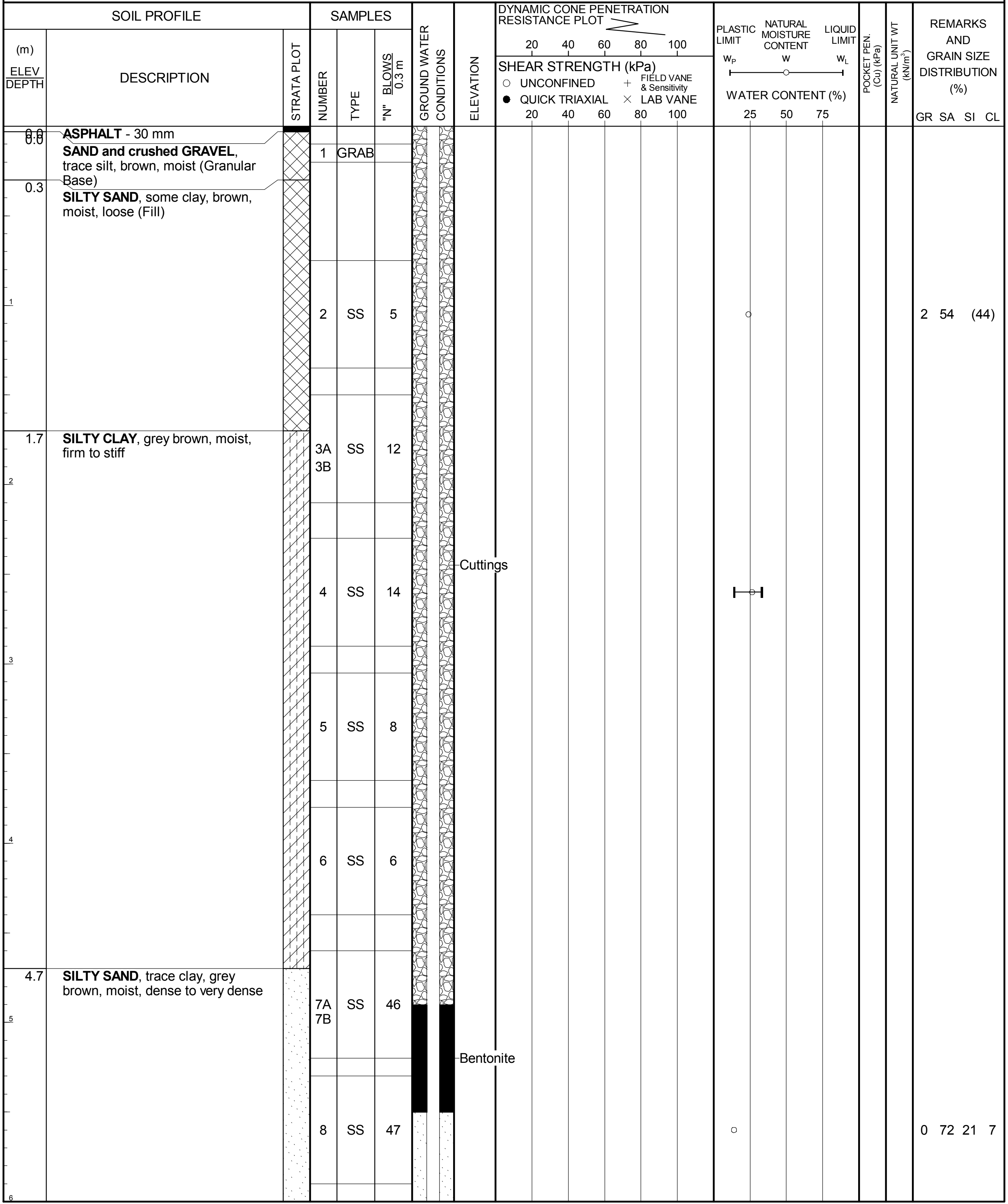
BOREHOLE LOGS

LOG OF BOREHOLE BH19-1

PROJECT: St. Mary Coptic Church
CLIENT: St. Mary Coptic Church
PROJECT LOCATION: 1 Canfield Rd, Nepean
DATUM: n/a
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Apr 22/2019

REF. NO.: 191-04634-00
ENCL NO.:



(m) ELEV DEPTH	DESCRIPTION
0.0	**ASPHALT** - 30 mm
0.0	**SAND and crushed GRAVEL**, trace silt, brown, moist (Granular Base)
0.3	**SILTY SAND**, some clay, brown, moist, loose (Fill)
1.7	**SILTY CLAY**, grey brown, moist, firm to stiff
4.7	**SILTY SAND**, trace clay, grey brown, moist, dense to very dense

SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	GRAB			
2	SS	5		2 54 (44)
3A 3B	SS	12		
4	SS	14	Cuttings	
5	SS	8		
6	SS	6		
7A 7B	SS	46	Bentonite	
8	SS	47		0 72 21 7

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

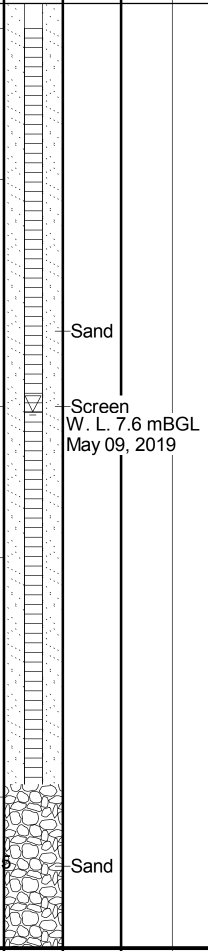
GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE BH19-1

PROJECT: St. Mary Coptic Church
CLIENT: St. Mary Coptic Church
PROJECT LOCATION: 1 Canfield Rd, Nepean
DATUM: n/a
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Apr 22/2019

REF. NO.: 191-04634-00
ENCL NO.:

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND, trace clay, grey brown, moist, dense to very dense(Continued) - Silty Clay seam noted between 6.1 m to 6.4 m in depth		9	SS	49							
7												
			10	SS	31							
8												
9												
			11	SS	>50/125 mm							
9.8	**END OF BOREHOLE** 1) 37.5 mm piezometer installed at 9.1 m below the existing ground surface. 2) Date Groundwater Depth -- May 9, 2019 7.6 m											

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE BH19-2

PROJECT: St. Mary Coptic Church
CLIENT: St. Mary Coptic Church
PROJECT LOCATION: 1 Canfield Rd, Nepean
DATUM: n/a
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: May 02/2019

REF. NO.: 191-04634-00
ENCL NO.:

SOIL PROFILE

(m) ELEV DEPTH	DESCRIPTION
0.0	**ASPHALT** - 30 mm
0.0	**SAND and crushed GRAVEL**, trace silt, brown, moist (Granular Base)
0.4	**SILTY CLAY**, grey brown, moist, firm (FILL)
1.7	**SILTY SAND**, trace clay, brown, moist, compact to dense - Silty Clay seam noted between 3.8 m to 3.95 m in depth - Silty Clay seam noted between 5.8 m to 5.9 m in depth

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m
1	GRAB	
2	GRAB	
3	SS	7
4A	SS	23
4B	SS	
5A	SS	12
5B	SS	
6	SS	23
7	SS	23
8	SS	14
9	SS	29

GROUND WATER CONDITIONS | ELEVATION

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

POCKET PEN. (Cu) (kPa) | NATURAL UNIT WT (kN/m^3)

REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH19-2

PROJECT: St. Mary Coptic Church
CLIENT: St. Mary Coptic Church
PROJECT LOCATION: 1 Canfield Rd, Nepean
DATUM: n/a
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: May 02/2019

REF. NO.: 191-04634-00
ENCL NO.:

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75 | POCKET PEN. (Cu) (kPa) | NATURAL UNIT WT (kN/m^3) | REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m
	SILTY SAND, trace clay, brown, moist, compact to dense(Continued)				
			10	SS	35
7					
	- wet below 7.7 m in depth				
			11	SS	27
8					
9					
			12	SS	33
9.8	**END OF BOREHOLE** 1) Groundwater observed at 7.2 m below existing ground surface				

WSP SOIL LOG GINT.GPJ SPL.GDT 12/5/19

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE BH19-3

PROJECT: St. Mary Coptic Church
CLIENT: St. Mary Coptic Church
PROJECT LOCATION: 1 Canfield Rd, Nepean
DATUM: n/a
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: May 02/2019

REF. NO.: 191-04634-00
ENCL NO.:

SOIL PROFILE		SAMPLES				
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION
0.0	**ASPHALT** - 30 mm				Cuttings	
0.0	**SAND and crushed GRAVEL**, trace silt, brown, moist (Granular Base)	1	GRAB			
0.3	**SILTY SAND**, brown, moist (Fill)					
1.1	**SILTY CLAY**, grey brown, moist, firm to stiff	2A 2B	SS SS	6		
		3	SS	16		
		4	SS	9		
		5	SS	8		
		6	SS	7		
4.7	**SILTY SAND**, trace clay, grey brown, moist, compact to very dense	7A 7B	SS SS	24	Bentonite	
		8	SS	20		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m^3)

REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: +³, ×³: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

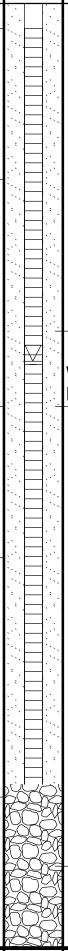
LOG OF BOREHOLE BH19-3

PROJECT: St. Mary Coptic Church
CLIENT: St. Mary Coptic Church
PROJECT LOCATION: 1 Canfield Rd, Nepean
DATUM: n/a
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: May 02/2019

REF. NO.: 191-04634-00
ENCL NO.:

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS / 0.3 m							
	SILTY SAND, trace clay, grey brown, moist, compact to very dense(Continued) - Silty Clay seam noted between 5.3 m to 5.5 m in depth		9	SS	24							
7						Sand W. L. 7.4 mBGL May 09, 2019 Screen						
	- very dense below 7.6 m in depth		10	SS	50/50 mm							
8												
9			11	SS	50/50 mm	Sand						
9.8	**END OF BOREHOLE** 1) 50 mm piezometer installed at 9.1 m below the existing ground surface. 2) Date Groundwater Depth May 9, 2019 7.4 m											



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH19-4

PROJECT: St. Mary Coptic Church
CLIENT: St. Mary Coptic Church
PROJECT LOCATION: 1 Canfield Rd, Nepean
DATUM: n/a
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Hollow Stem Auger Drilling
Diameter: 203 mm
Date: Apr 22/2019

REF. NO.: 191-04634-00
ENCL NO.:

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION					
0.0	**ASPHALT** - 30 mm	■										
0.0	**SAND and crushed GRAVEL**, trace silt, brown, moist (Granular Base)	▨	1	GRAB								
0.3	**SILTY CLAY**, grey brown, moist, firm to stiff (FILL)	▨	2	SS	8							
1.7	**SILTY SAND**, trace clay, moist, compact to dense	░	3A 3B	SS	32							
	- Silty Clay seam noted between 2.3 m to 2.6 m in depth	░	4A 4B	SS	16							
		░	5	SS	19							
3.7	**END OF BOREHOLE** 1) Borehole is dry upon completion of drilling											

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^{3}, \times^{3}$: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

Explanation of Terms Used in the Record of Boreholes

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Spoon sample
SH	Shelby tube Sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

WH – Samples sinks under "weight of hammer"

Dynamic Cone Penetration Resistance, N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Textural Classification of Soils

Classification	Particle Size
Boulders	> 200 mm
Cobbles	75 mm - 200 mm
Gravel	4.75 mm - 75 mm
Sand	0.075 mm – 4.75 mm
Silt	0.002 mm-0.075 mm
Clay	<0.002 mm

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	> 35%

Soil Description

a) Cohesive Soils(*)

Consistency	Undrained Shear Strength (kPa)	SPT "N" Value
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

(*) Hierarchy of Shear Strength prediction

1. Lab triaxial test
2. Field vane shear test
3. Lab. vane shear test
4. SPT "N" value
5. Pocket penetrometer

b) Cohesionless Soils

Density Index (Relative Density)	SPT "N" Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

APPENDIX

C

LABORATORY TESTING RESULTS

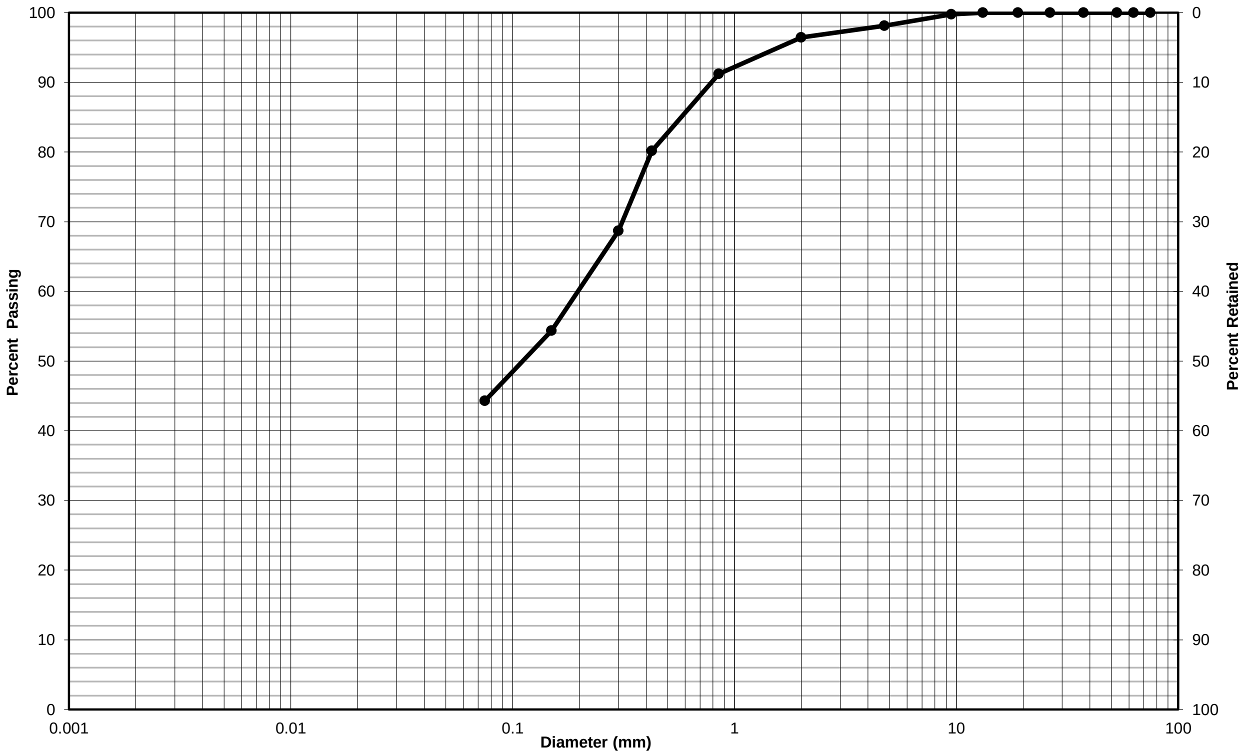
Particle-Size Analysis of Soils
(ASTM D422)

Client:	St. Mary Coptic Church	**Lab no.:**	508-1
Project/Site:	St. Mary Coptic Church	**Project no.:**	191-04634-00

Borehole no.: BH19-1

Sample no.: SS2

Depth: 0.75-1.35m



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

	Gravel	Sand	Clay & Silt	Silt	Clay
Percent %	1.9	53.9	44.3	-	-

Remarks:

Performed by:	Rupesh Subedi	**Date:**	May 2, 2019
Verified by:	Nick Krebs	**Date:**	May 6, 2019

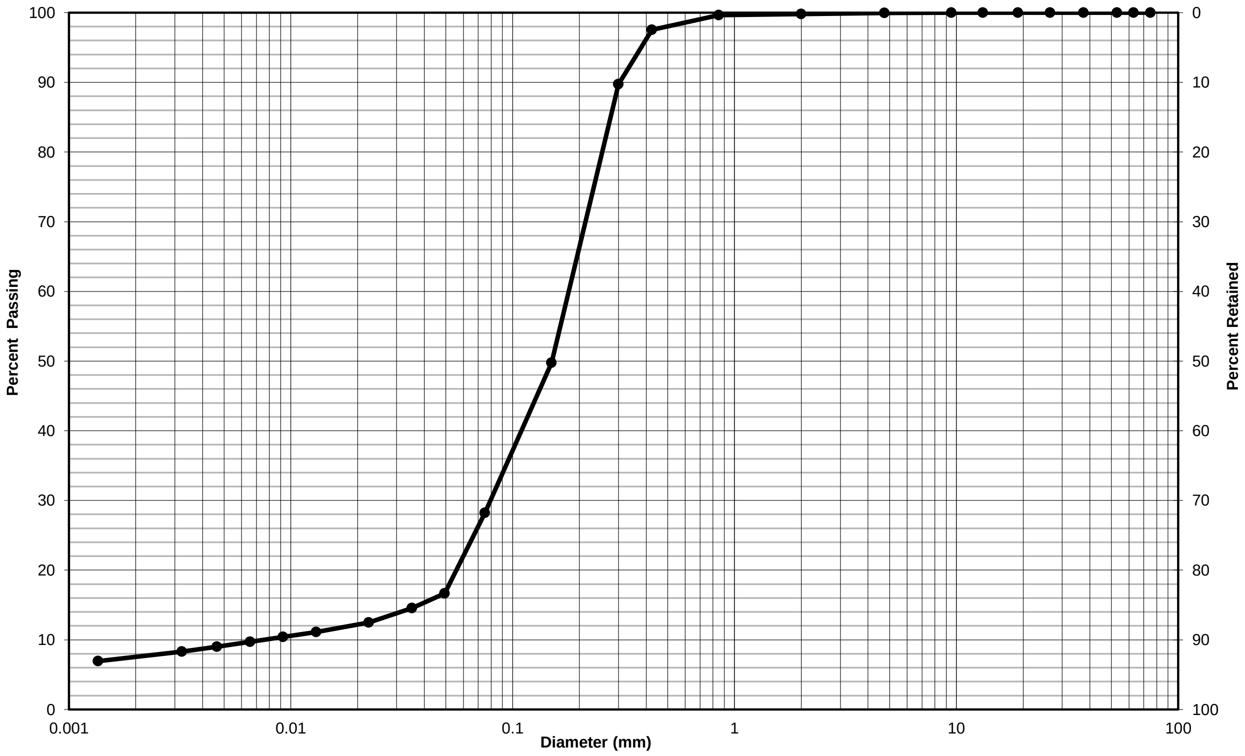
Particle-Size Analysis of Soils
(ASTM D422)

Client: St. Mary Coptic Church **Lab no.:** 508-5

Project/Site: St. Mary Coptic Church **Project no.:** 191-04634-00

Borehole no.: BH19-1 Sample no.: SS8

Depth: 5.3-5.9m



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	0.0	71.8	28.2	20.8	7.4

Remarks:

Performed by: Rupesh Subedi **Date:** May 3, 2019

Verified by: Nick Krebs **Date:** May 6, 2019

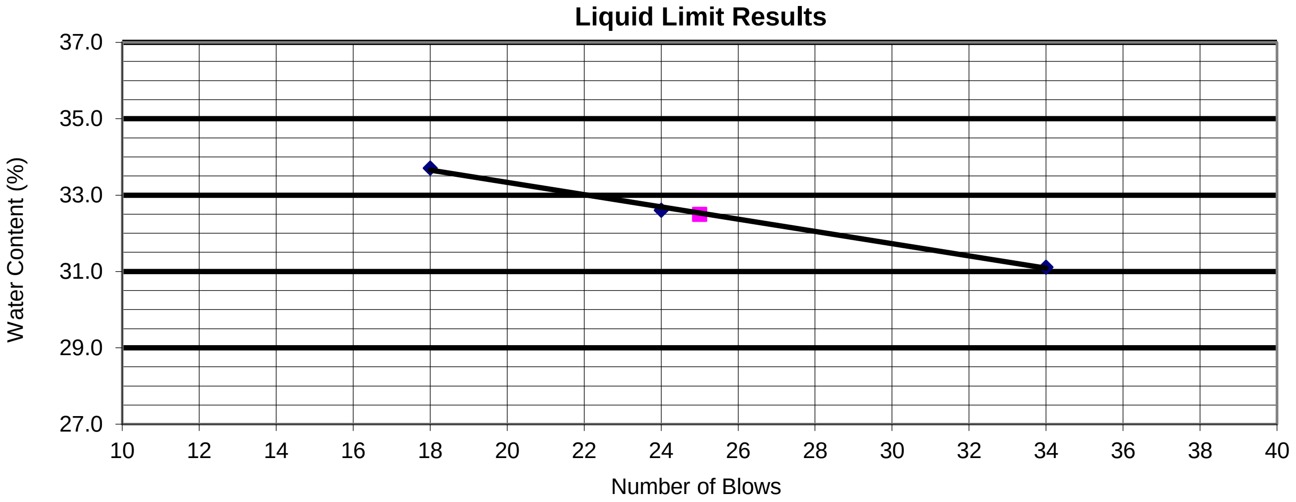
Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

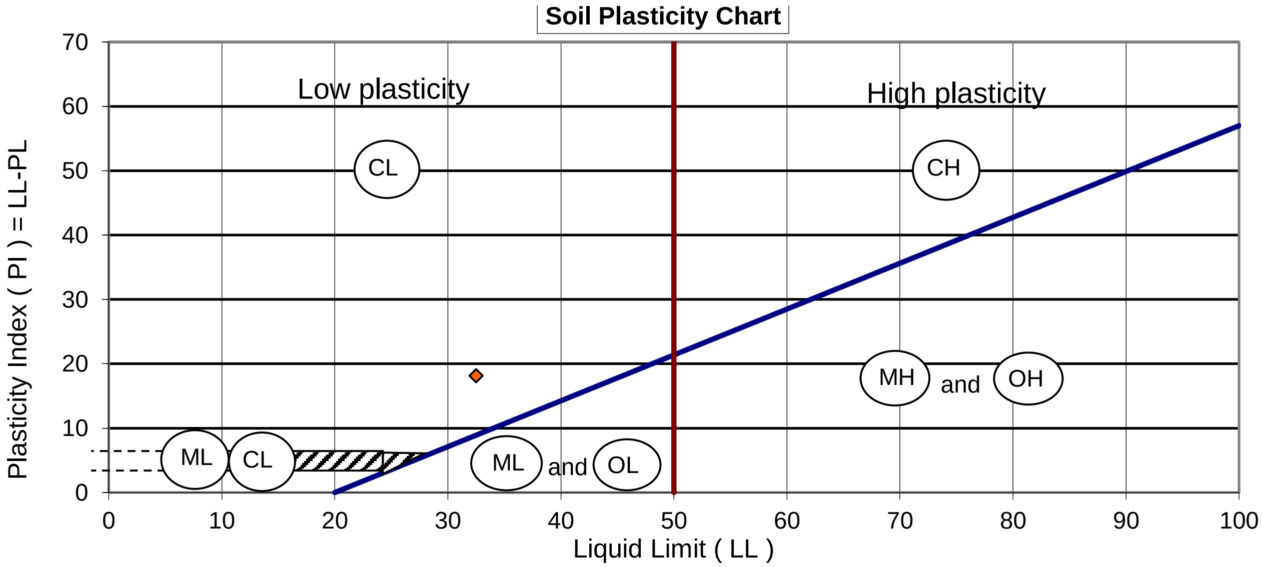
Client: St. Mary Coptic Church **Lab No.:** 508-2

Project/Site: St. Mary Coptic Church **Project No.:** 191-04634-00

Borehole No.: BH19-1 Sample No.: SS4

Sample Depth: 2.3-2.9m





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
33	14	18	26.5

Sample Description: **CL - Low plasticity, inorganic clay**

Performed By: Rupesh Subedi **Date:** May 2, 2019

Verified By: Nick Krebs **Date:** May 6, 2019

Certificate of Analysis

Client:	WSP Canada Inc. (SPL) 146 Colonnade Rd., Unit 17 Ottawa, ON K2E 7Y1
Attention:	Mr. Daniel Wall
PO#:	
Invoice to:	WSP Canada Inc.

Report Number:	1906324
Date Submitted:	2019-04-29
Date Reported:	2019-05-07
Project:	St Mary Coptic Church 191-04634-00
COC #:	201126

				Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1423299 Soil 2019-04-22 BH19-1 SS5	1423300 Soil 2019-04-22 BH19-1 SS7B
Group	**Analyte**	**MRL**	**Units**	**Guideline**		
Anions	Cl	0.002	%		0.012	0.006
	SO4	0.01	%		0.02	<0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.27	0.11
	pH	2.00			8.30	8.57
	Resistivity	1	ohm-cm		3700	9090

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX

D

LIMITATIONS OF THIS REPORT

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP Canada Incorporated (WSP) at the time of preparation. Unless otherwise agreed in writing by WSP, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. WSP accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.