Stantec

Geotechnical Investigation Report

New Church

3856, 3866 and 3876 Navan Road
Ottawa, ON

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Table of Contents

LIST OF TABLES

LIST OF APPENDICES

APPENDIX A
Statement of General Conditions

APPENDIX B
Drawing No. 1 – Key Plan
Drawing No. 2 – Borehole Location Plan
Figure B1. Geotechnical Model
Figure B2. Soil Sensitivity Profile

APPENDIX C
Symbols & Terms Used on the Borehole Records
Borehole Records
Bedrock Core Log and Photograph

APPENDIX D
Laboratory Test Results
Figure D1: Grain Size Distribution Plot – SILTY SAND (SM)
Figure D2: Grain Size Distribution Plot – Champlain Sea Clay (CH)
Figure D3: Plasticity Chart – Champlain Sea Clay (CH)
Report: Oedometer (Consolidation) Test Results – Champlain Sea Clay (CH)

APPENDIX E
Laboratory Chemical Analysis Results

APPENDIX F
NBC Seismic Hazard Calculation Data Sheet
Figure F1: Factor of Safety against Liquefaction

1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) has been retained by St. George and St. Anthony Church to carry out a geotechnical investigation for a new church to be constructed in the City of Ottawa, Ontario. The geotechnical investigation was completed in order to determine the subsurface conditions at the site and to provide geotechnical recommendations and design parameters.

This report presents the results of the field investigation program and laboratory testing, as well as geotechnical design recommendations. Limitations associated with this report and its contents are provided in the Statement of General Conditions included in Appendix A.

2.0 SITE AND PROJECT DESCRIPTIONS

The site is located on 3856, 3866 and 3876 Navan Road in Ottawa, Ontario as shown on Drawing No. 1 in Appendix B. The property is a vacant, undeveloped land. Based on a topographical plan of the site prepared by Annis, O'Sullivan, Vollebekk Ltd. dated September 2016, the ground surface at the site is generally flat with existing ground surface elevations varying between about 85.2 m and 85.8 m.

It is noted that beyond the south edge of the property is a treed slope that is approximately 15 m in height extending down towards the Prescott-Russell recreational trail and the Mer Bleue Bog located approximately 700 m away from the site. The slope appears to be locally as steep as 4H:1V.

Based on the information provided by Eternal Engineering Corp, Stantec understands that the proposed development will consist of a one-storey church building with an approximate plan area of 846 m^2, a one-storey service building with an approximate plan area of 1,585 m^2, paved parking areas and driving lanes. The proposed buildings will not include underground levels.

3.0 GEOLOGY

Available geological maps indicate that the surficial geology at the site is anticipated to consist of surficial sand deposit underlain by a thick deposit of Champlain Sea clay over shale bedrock of Billings formation. Nearby borehole and water well records suggest that the clay may extend to depths of about 25 to 30 m below ground surface.

The Champlain Sea clays are typically highly compressible and can undergo large settlements when subjected to new loads associated with site grade fills or foundations. In accordance with the Interactive Vs30 Google Map for the City of Ottawa, the site is located in a seismic class E zone.

4.0 INVESTIGATION METHODS

4.1 BOREHOLE INVESTIGATION

Prior to commencing the field investigation, Stantec arranged for utility clearances to be completed by a private utility locating contractor, USL-1.

A geotechnical field investigation consisting of advancing six (6) boreholes, designated as BH18-1 to BH18-6, was carried out on February 2 to 6, 2018. The approximate borehole locations are shown on Drawing No. 2.

The boreholes were drilled using a track-mounted drill rig equipped with 200 mm diameter, hollow-stem augers and rock coring capabilities that was supplied and operated by George Downing Estate Drilling Ltd.

The subsurface stratigraphy encountered in each borehole was recorded in the field by Stantec field personnel. Soil samples were recovered at regular intervals using a 50-mm (outside diameter) split-tube sampler by conducting Standard Penetration Tests (SPTs) in accordance with the procedures outlined in ASTM specification D1586. In-situ shear vane measurements were carried out at selected depths within the cohesive soil deposit. A series of Shelby Tube samples were collected within the clayey soils. Dynamic Cone Penetration Testing (DCPT) was completed in BH18-4, BH18-5 and BH18-6 to confirm the inferred depth to bedrock. Coring was carried out in BH18-6 to confirm the type and engineering characteristics of the bedrock.

All soil samples recovered from the boreholes were placed in moisture-proof bags. Soil and bedrock samples collected during the investigation were returned to Stantec's Ottawa laboratory for detailed classification and testing.

A standpipe piezometer was installed in BH18-5 to facilitate the measurement of the groundwater level at the site. The remaining boreholes were backfilled with drill cuttings mixed with bentonite.

Borehole location information is presented on the Borehole Records in Appendix C and summarized in Table 4.1 below.

Table 4.1: Summary of Borehole Details

Borehole No.	Approximate UTM Coordinates (Zone 18T)		Approximate Ground Elevation (m)
	Northing	Easting	
BH18-1	5030115	462323	85.3
BH18-2	5030037	462357	85.4
BH18-3	5030054	462401	85.5
BH18-4	5030054	462360	85.3
BH18-5	5030105	462398	85.8
BH18-6	5030087	462377	85.6

4.2 LABORATORY TESTING

The following geotechnical laboratory testing was performed on selected samples:

- Moisture contents;
- Grain size distribution/hydrometer analyses; and
- One oedometer (consolidation) test.

The results of the laboratory tests are discussed in the text of this report and are provided on the Borehole Records and Bedrock Core Log in Appendix C. Figures illustrating the results of the grain size distribution tests, and Atterberg Limits tests are included in Appendix D.

Chemical analyses related to parameters associated with the potential for corrosion or sulphate attack (i.e. pH, resistivity, and chloride and sulphate content) were completed on two (2) samples by Paracel Laboratories Inc.

Samples remaining after testing will be stored for a period of three (3) months after issuance of the final report. Samples will then be discarded after this period unless otherwise directed.

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL

Detailed descriptions of the subsurface soil and bedrock conditions are presented on the Borehole Records, Bedrock Core Log, and Rock Core Photographs provided in Appendix C. Documents providing explanations of the symbols and terms used on the borehole records are also provided in Appendix C. Laboratory test results are presented in Appendix D as well as on the borehole records.

The stratigraphic boundaries on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact boundaries between geological units. The borehole records depict conditions at the particular locations and at the particular times indicated. The subsurface soil and groundwater conditions between boreholes and/or at locations away from the borehole locations will vary from those indicated on the borehole records.

It is noted that information provided in the following sections is intended to summarize the conditions encountered; however, the borehole records provided in Appendix C should be used as the primary source of the subsurface information for the site.

A summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

5.2 OVERBURDEN

In general, the subsurface stratigraphy encountered at the site consists of a surficial layer of topsoil followed by silty sand that is underlain by a thick Champlain Sea clay deposit followed by shale bedrock.

5.2.1 Topsoil

The thickness of the topsoil was measured to be approximately 100 to 190 mm at the surface of all borehole locations. The topsoil was typically comprised of silty clay that was black/grey in colour.

5.2.2 Silty Sand

A layer of silty sand was encountered beneath the topsoil and extended to depths of approximately 3 to 4 m below ground surface. Standard Penetration Test (SPT) penetration resistances of 3 to 14 per 0.3 m of penetration were measured within this layer indicating these materials are in a loose to compact state.

Laboratory testing conducted on samples of the silty sand measured natural moisture contents of between 24 and 36%, expressed as a percentage of the dry weight of the soil.

Grain size distribution tests were completed on four (4) samples of the silty sand. The results of the tests are presented on Figure D1 in Appendix D and summarized in Table 5.1.

Table 5.1: Grain Size Distribution – Silty Sand (SM)

Borehole	Sample	Depth (m)	Description	% Gravel	% Sand	% Silt and Clay
BH18-1	SS2	1.1	SILTY SAND (SM)	0	86.4	13.6
BH18-1	SS3	1.8	SILTY SAND (SM)	2.1	77.1	20.7
BH18-2	SS3	1.8	POORLY GRADED SAND with silt (SP-SM)	0	89	11
BH18-5	SS3	1.8	SILTY SAND (SM)	0	86.1	13.9

In accordance with the Unified Soil Classification System, the sample tested can be generally classified as SILTY SAND (SM).

5.2.3 Champlain Sea Clay

The silty sand deposit was underlain by a deposit of sensitive to extra-sensitive Champlain Sea clay. This deposit extended to depths of approximately 28 to 29 m below ground surface.

In-situ vane shear tests conducted on the Champlain Sea clay measured undrained shear strength values of about 27 to 60 kPa. The estimated sensitivity values of the Champlain Sea clay are presented on Figure B2 in Appendix B. The sensitivity of the clay ranged from 4 to 14 and the clay is classified as sensitive to extra-sensitive in accordance with the errata to the 4th (2006) Edition of the Canadian Foundation Engineering Manual (CFEM).

Laboratory testing conducted on samples of the Champlain Sea clay measured natural moisture contents of between 51 and 84%.

The results of grain size distribution tests completed on two (2) samples of the Champlain Sea clay are displayed on Figure D2 in Appendix D and are summarized in Table 5.2 below.

Table 5.2: Grain Size Distribution – Champlain Sea Clay

Borehole	Sample	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH18-4	SS7	5.6	0	16	46	38
BH18-6	ST8	7.9	0	1	69	30

The results of Atterberg limits testing carried out on representative samples of this material are summarized in Table 5.3 below. The results of this testing are also shown on the Borehole Records included in Appendix C and on Figure D3 in Appendix D, indicate that the Champlain Sea clay samples tested can be classified as Clay of high plasticity (CH).

In addition, the calculated Liquidity Index for the Champlain Sea clay samples were 1.1 and 1.3 as presented in Table 5.3 below. A liquidity index greater than about 1 corresponds to sensitive to extra sensitive clay and values of approximately 1.4 or greater are indicative of quick clay conditions.

Table 5.3: Atterberg Limits Test Results – Champlain Sea Clay (CH)

Borehole	Sample	Depth (m)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI = (LL-PL)	Liquidity Index, LI LI = (Wn-PL)/(LL-PL)
BH18-4	SS7	5.6	67	23	44	1.1
BH18-6	ST8	7.9	63	22	41	1.3

The results of a consolidation test carried out on a Shelby tube sample collected from a depth of 7.9 m at Borehole BH18-6 are summarized in Table 5.4 below. The results of the consolidation testing are also presented graphically in Appendix D.

Table 5.4: Summary of Consolidation Test Results - Champlain Sea Clay (CH)

Borehole No/Sample No	Sample Depth/Elevation (m)	Moisture Content (%)	Initial Void Ratio	Specific Gravity, G_s	C_r	C_c	P'_c (kPa)
BH18-6/ST 8	7.9/77.7	77	2.2	2.8	0.03	2.2	98

where:

C_r = Recompression Index.
C_c = Compression Index
P'_c = Estimated Preconsolidation Pressure

5.3 BEDROCK

Bedrock was encountered in Borehole BH18-6 at a depth of 27.8 m (corresponding to elevation of 57.8m). DCPT refusal was encountered on inferred bedrock at depths of 28.5 and 29 m in BH18-4 and BH18-5, respectively (corresponding to an elevation of 56.8 m). The bedrock core obtained from the borehole consisted predominantly of good quality, black, slightly weathered shale. A detailed description of the rock core is provided on the Bedrock Core Log in Appendix C. Rock core photographs are also provided in Appendix C.

A compressive strength test conducted on one (1) rock core sample collected from a depth of about 28 m in BH18-6 showed that the compressive strength of the sample tested was 35 MPa. The test result indicates that the bedrock is medium strong.

5.4 GROUNDWATER CONDITIONS

A groundwater monitoring well, with a screen from 3.4 m to 6.4 m below ground surface, was installed in BH18-5. The groundwater level in this well was recorded at approximately 0.2 m below ground surface on February 27th, 2018 (corresponding to an approximate elevation of 85.6 m). An additional measurement, taken on July 4, 2018, encountered water at a depth of 1.8 m below grade (approximate elevation of 84.0 m). Groundwater levels at the site will be subject to fluctuations due to seasonal changes and precipitation events.

5.5 CHEMICAL ANALYSIS

Chemical testing was completed on selected soil samples from BH18-5 and BH18-6. Table 5.5 below summarizes the results.

Table 5.5: Summary of Chemical Testing Results

Borehole No.	Sample No./Depth	Physical Characteristics				
		% Solids (by Wt.)	**pH**	**Resistivity (Ohm-m)**	**Chloride (ug/g)**	**Sulphate (ug/g)**
BH18-5	SS4/2.6m	79.2	7.6	49.6	109	27
BH18-6	SS1/1.1m	80.6	7.2	150	9	29

6.0 DISCUSSION AND RECOMMENDATIONS

The geotechnical investigation was carried out based on an early proposed site layout. It is understood that due to the geotechnical and site grading constraints, the site plan has been modified resulting in portions of the proposed buildings falling outside of the area enclosed by the boreholes. The recommendations provided in this report include a light weight fill design option which is applicable to the proposed new layout locations, however, prior to construction, additional boreholes will be required in order to confirm that no other issues are present and to provide a final confirmation of the slope stability analysis for the natural slope immediately south of the property and within approximately 7 m of the south building limit beyond which the ground slopes down for an approximate height of 15 m towards the Prescott-Russell recreation trail and the Mer Bleue Bog.

Although it is not anticipated that soil conditions will differ significantly from the conditions encountered at the boreholes, it is recommended that prior to construction at least a total of three additional boreholes be drilled and sampled within the final building footprints and near the natural slope.

6.1 KEY GEOTECHNICAL ISSUES

Key geotechnical issues that require consideration for this project include the following:

- The site includes a 3 m to 4 m silty sand cap that generally in a loose to compact state and would be liquefiable under the standard applicable earthquake design loading. Therefore, as part of the site preparation works, the sands at the site will need to be densified.
- All topsoil and/or organic soils should be removed.
- The site is underlain by an approximately 25 m thick, compressible deposit of Champlain Sea clay. The clay deposit has a firm consistency and has a limited capacity to support new loads (e.g. from site grade fill placement, foundation and floor loads and/or potential groundwater level lowering, etc.).
- The in-situ and laboratory shear vane test results suggest that the Champlain Sea clay deposit is highly sensitive to strength loss when disturbed. In addition, laboratory test results indicate the natural moisture content of the clay is higher than the measured liquid limit. Therefore, the clay can behave like a fluid when excavated and/or disturbed. This material is not considered suitable for re-use and could require specialized handling procedures (e.g. drying) prior to transport off-site.
- Grade raises of more than 0.6 m in depth shall be achieved using light weight fill (i.e., Styrofoam blocks) within the building footprints and shall be extended to at least 6 m away from the building footprints.
- As discussed before, the sand will have to be densified before the installation of the shallow foundations. Otherwise, piled foundations driven to the bedrock surface may be used.
- The Champlain Sea clay deposit is typically expected to be highly frost susceptible. It is typically prone to large amounts of heaving for the first few years; magnitudes of over 150 mm should be expected. It is generally not recommended to cut significantly within this type of soil unless large frost heave movements can be tolerated or unless insulation is applied below pavement structures. In this regard, the site is covered with 3 to 4 m of sand, and therefore, it is not anticipated that the clay will be exposed to freezing conditions.
- The Champlain Sea clay is typically sensitive to settlement from the water demand from trees. The selection and planting of trees should follow the City of Ottawa guidelines for tree planting in sensitive marine clay. The overgrowth of tree roots, as well as the phenomenon of tree root removing moisture from surrounding soils, may modify the soils properties. Therefore, species of tree whose characteristics are known to match these concerns should not be proposed in the landscape areas. In general, the planting of trees should be offset from foundations by a distance equal to at least the theoretical mature tree height.

The following sections incorporate the above-mentioned key geotechnical issues.

6.2 GRADE RAISE RESTRICTION

The site is underlain by a highly compressible Champlain Sea clay deposit that is approximately 25 m thick. The results of a consolidation test carried out on a sample of the clay indicates that the material has a pre-consolidation pressure of approximately 98 kPa at an approximate depth of 7.9 m, which is slightly higher than current loading conditions due to self-weight of the sand and clay and considering a design anticipated ground water level of being at elevation 83.0 m.

Large consolidation settlements may occur when the application of new loads such as site grade fills and building loads result in final loads exceeding the maximum past loading conditions (i.e. the preconsolidation pressure) of the Champlain Sea clays.

Calculation of the potential settlement of the compressible clay beneath this site due to the placement of the proposed site grade fill materials was performed using the computer program Settle 3D (Rocscience, 2009) which is a three-dimensional program for the analysis of the consolidation/vertical settlement of soil under surface loads. The geotechnical design parameters used for the Settle 3D analysis are summarized on Figure B1 in Appendix B.

The results of the settlement analyses indicate that the placement of granular fill to raise the site grades 0.6 m would result in settlements of approximately 25 mm. A maximum grade raise restriction of 0.6 m is, therefore, recommended for the development due to the compressible soils encountered at the site.

Grade raises of more than 0.6 m in depth shall be achieved using light weight fill (i.e., Styrofoam blocks) within the building footprints and shall be extended to at least 6 m away from the building footprints.

6.3 FROST PENETRATION

The frost penetration depth for foundation design at this site is 1.8 m.

It is noted that the above frost penetration depth is applicable only to foundation design. Short period deeper frost penetrations, which would have little impacts on foundations, may occur. The typical soil cover for water mains is 2.4 m below ground surface in the City of Ottawa.

6.4 SITE PREPARATION

An approximately 100 to 190 mm thick layer of topsoil containing organic matters was encountered at the surface of the boreholes. Prior to carrying out soil densification works, all existing surficial topsoil, vegetation and/or other deleterious materials (e.g. any loose, wet, and/or otherwise disturbed native materials) should be completely removed from within the footprint of the new development.

As discussed in Section 6.10.1 (Seismic Design Considerations), the 3 to 4 m thick silty sand layer which overlies the deep clay layer is in a loose to compact state and has the potential of liquefying under the earthquake design conditions defined by the Ontario Building Code. Therefore, as part of the site preparation works, soil densification will be required and should incorporate the following:

- Densification methods, such as dynamic compaction or rapid impact methods should be considered.
- Construction vibrations should be monitored during the construction works to meet the vibration constraints at nearby buildings that are imposed by the City of Ottawa.
- The in-situ densification program should be designed to achieve an in-situ density corresponding to an SLS bearing pressure of 150 kPa for the sand. The in-situ densification should be designed to achieve a minimum factor of safety against liquefaction of 1 throughout the densified pad.
- The densified sand should extend to 6 m beyond the limits of the proposed buildings.

The prepared subgrade soils will require inspection by geotechnical personnel prior to structural fill placement to verify all unsuitable material has been removed.

The site grades should then be raised/reinstated using Structural Fill consisting of Ontario Provincial Standard Specification (OPSS) Granular B Type I or II materials that are placed in lifts no thicker than 300 mm and compacted to at least 100% of the material’s Standard Proctor Maximum Dry Density (SPMDD). The final layer of fill should consist of OPSS Granular A materials with a minimum thickness of 300 mm beneath the floor slabs and 200 mm in other areas.

The placement of all engineered fill materials should be monitored on a full-time basis by qualified and experienced geotechnical personnel under the supervision of a geotechnical engineer, with the authority to stop the placement of fill at any time when conditions are considered to be unacceptable.

All fill materials imported to the site must meet all applicable municipal, provincial, and federal guidelines and requirements associated with environmental characterization of the materials.

The contractor should be responsible for protecting the subgrade soils from disturbance due to construction traffic. This may require that construction access routes are temporarily overbuilt (i.e. provided with increased granular fill) and/or geotextiles are provided between the granular fill and the subgrade surface.

Imported fill materials should be tested and approved by a geotechnical engineering firm prior to delivery/use. Monitoring of fill placement and in situ compaction testing should be carried out to confirm that all fill is placed and compacted to the required degree.

6.5 SLOPE STABILITY CONSIDERATION

The current proposed building footprint is located within 7 m of the south limit of the property beyond which the ground surface slopes down about 15 m towards the recreational Prescott-Russell recreational trail and the Mer Bleue Bog. Based on a cursory review of the slope, it appears to be relatively gentle with slopes ranging from 7H:1V to 10H:1V, with local portions possibly as steep as 4H:1V. As a result of the current proposed building location, it is recommended that the following be incorporated in the final geotechnical investigation phase:

- Confirmation of the nearby slope geometry by either an elevation survey or by a lidar survey.
- Geotechnical soil stability analysis incorporating the nearby slope geometry and future borehole information to be obtained near the slope.

Generally, the surface drainage within the site should be collected and directed towards a storm water management system. Drainage should not be directed towards the sloping ground to the south in order to avoid surface erosion of any natural clay slopes. Surface erosion in some circumstances can destabilize stable clay slopes.

6.6 TEMPORARY EXCAVATIONS

All temporary excavations should be carried out in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. Care should be taken to direct surface water away from open excavations.

It is anticipated that shallow open cut excavations extending to depths of less than 2 to 3 m below existing ground surface. The potential for instability of excavations extending to greater depths should be reviewed by a geotechnical engineer.

Based on the boreholes advanced within the site, excavations within upper 3 m to 4 m of existing site grades are expected to be within the silty sand deposit. This material would be classified as Type 3 soils, as defined by the Occupational Health and Safety Act and Regulations for Construction Projects.

Provided that appropriate groundwater control is provided to maintain the water level below the base of the excavation, OHSA indicates that temporary excavations made within Type 3 soils should be developed with side slopes no steeper than 1H:1V.

Steeper side slopes would require shoring to meet the requirements of the OHSA. All shoring systems should be designed and approved by a qualified Professional Engineer.

The stability of the wall of the excavation may be affected by surcharge loads, stockpiles as well as groundwater seepage conditions. Therefore, soils excavated from the trenches and/or construction materials should not be stockpiled adjacent to excavations.

The base of excavations should not be exposed for extended periods of time.

6.7 DEWATERING

Groundwater inflows into small and shallow excavations of less than 2.0 to 3.0 m deep developed within the silty sand deposit that extends slightly below the water table could be handled by pumping from filtered sumps within the excavation areas.

More significant groundwater inflows should be expected for deeper excavations. Therefore, more extensive dewatering systems could be required for such conditions requiring Ministry of the Environment and Climate Change (MOECC) permitting.

6.8 REUSE OF ON-SITE MATERIALS

The surficial topsoil materials are unsuitable for reuse in any application except for general landscaping purposes.

The native silty sand soils are not considered to be suitable for reuse as engineered/structural fill below or adjacent to new foundations. These materials that are free of organic matter and other deleterious materials, may be considered suitable for reuse as trench backfill (outside of foundation areas) or as general site grade fill (i.e. materials used to raise the site grade to the design elevations outside building footprints).

The ability to compact these materials to required levels is dependent on the moisture content of the materials; thus, the amount of re-useable material will be dependent on the natural moisture content, weather conditions and the construction techniques at the time of excavation and placement. Although not expected for this site, any boulders or cobbles with dimensions greater than 150 mm should be removed from these materials prior to placement.

The Champlain Sea clay soils encountered at depths of approximately 3 to 4 m are not considered to be suitable for re-use due to the high natural water content(s) of these materials. As indicated previously, this clay material has natural water contents that are above their Liquid Limits. These materials can behave like a fluid once excavated/disturbed and could require drying of the soil prior to transport.

6.9 PIPE BEDDING AND BACKFILL

OPSS Granular A materials should be placed below sewer and water pipes as bedding material. The bedding should have a minimum thickness of 150 mm or more to meet City of Ottawa standards. Where unavoidable disturbance to the subgrade surface does occur, it may be necessary to thicken the bedding layer or provide a sub-bedding layer of compacted Granular B Type II materials. Pipe backfill and cover materials should also consist of OPSS Granular A material. A minimum of 300 mm vertical and side cover should be provided. These materials should be compacted to at least 95% of the material’s SPMDD in lifts no greater than 300 mm. Clear crushed stone backfill should not be permitted as pipe bedding materials.

Where the pipe trenches will be covered with hard surfaced areas, the type of native material placed in the frost zone (i.e. between subgrade level and 1.8 meters depth or the top of the pipe cover materials) should match the soil exposed on the trench walls for frost heave compatibility.

Trench backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 98 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

If there is insufficient reusable material at the site, any bulk fill required to raise the site grades should consist of imported granular fill meeting the requirements of OPSS Select Subgrade Material (SSM).

All imported fill materials should be tested and approved by a geotechnical engineering firm prior to delivery to the site.

6.10 FOUNDATION DESIGN

6.10.1 Seismic Design Considerations

The seismic Site Class value, as defined in Section 4.1.8.4 of the 2012 Ontario Building Code (OBC), contains a seismic analysis and design methodology which uses a seismic site response and site classification system defined by the shear stiffness of the upper 30 metres of the ground below the foundation level. There are six site classes (from A to F), decreasing in stiffness from A (hard rock) to E (soft soil); Site Class F denotes problematic soils for which a site-specific evaluation is required.

Based on the results of the field investigation conducted at the site, a Site Class designation of E is considered appropriate. If a higher site classification is required, a site-specific shear wave velocity test could be carried out to see if more favourable results are possible. However, given the depth to bedrock at this site, it should be anticipated that the results of further testing could leave the site class unmodified.

A copy of the NBC Seismic Hazard Calculation Data sheet is provided in Appendix F.

The potential liquefaction of the site soils under seismic loading conditions was assessed using the analysis methodology suggested by Idriss and Boulanger (2008). The evaluation was completed based on the SPT resistance values from the boreholes and based on the following:

- A Site Adjusted PGA of 0.349g.
- An earthquake magnitude, M_w of 6.47.

The assessment indicates that the Silty Sand soils extended to depths of approximately 3 to 4 m are considered susceptible to liquefaction as presented on Figure F1 in Appendix F. As a result of liquefaction, earthquake-induced settlements in the order of 40 mm to 70 mm should be anticipated. Soil strata susceptible to liquefaction will also undergo loss of strength, stiffness and support capacity. Soil improvement techniques should be implemented to improve the Silty Sand condition (e.g., dynamic compaction). The dynamic compaction program should be designed and implemented so that a factor of safety against liquefaction of 1 or more within the proposed building footprints is achieved.

6.10.2 Shallow Footings

The buildings could be supported on shallow footings bearing on the native silty sand deposit encountered above the clay deposit at the site. The following will need to be incorporated in the design:

- The sand is under a loose to compact condition and was determined to be liquefiable. Therefore, the sand will have to be densified before the installation of the footing foundations. This can be achieved using dynamic compaction techniques or rapid impact compaction methods. As stated earlier, the soil densification program should be designed and implemented so that a factor of safety against liquefaction of 1 or more within the proposed building footprints is achieved.
- Light weight fill materials (Styrofoam Blocks) shall be used to achieve the proposed grade raises of more than 0.6 m in depth within the building footprint and should be extended to at least 6 m away from the building perimeters.
- Subdrains shall be installed around the perimeters of the building and the Styrofoam blocks.

Assuming that the sand is treated as discussed above, shallow footing foundations may be considered for the proposed buildings and should be installed at a depth of 1.8 m below the final grade. Resistances at Ultimate Limits States (ULS) and at Serviceability Limits States (SLS) for new square and strip footings have been calculated and are provided in Table 6.1 below.

Table 6.1: Geotechnical Resistance for Shallow Footings

Footing Width (m)	Minimum Footing Embedment (m) Below Final Ground Surface	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)
Square Footings			
1	1.8	380	150
2		410	115
3		345	60
4		260	45
Strip Footings			
0.8	1.8	260	150
1		270	110
1.5		295	75
2		250	60

Notes:

The geotechnical resistances in the above table are provided for the range of footing widths and the minimum footing embedments listed in the above table. Additional input should be provided by the geotechnical engineer if the foundation sizes or depths are outside of the ranges outlined above.

The factored geotechnical bearing resistance at ULS incorporates a resistance factor of 0.5. The post-construction total settlements of footings sized using the above SLS bearing pressure should be less than about 25 mm,.

The subgrade surfaces beneath all footings must be inspected by qualified geotechnical personnel prior to placing concrete in order to confirm the above design pressures and to ensure there are no disturbances or deleterious materials at the bearing surface.

The unfactored horizontal resistance to sliding of the spread foundations may be calculated using the following unfactored coefficients of friction:

0.45	between native silty sand and cast-in-place concrete
0.55	between OPSS Granular A or B Type II materials and cast-in-place concrete

In accordance with Table 8.1 of the Canadian Foundation Engineering Manual 4th Edition (CFEM), a resistance factor (ϕ_{gu}) against sliding (for frictional materials) of 0.8 should be applied to obtain the resistance at ULS.

6.10.3 Piled Foundations

Deep foundation systems are considered technically feasible for the proposed development at this site. The buildings could be supported on deep foundations transferring the foundation loads to below the compressible Champlain Sea clay layer (i.e., down to the bedrock surface).

A suitable pile type would be concrete filled steel pipe piles (driven closed-ended) or H-piles, with the piles end-bearing on bedrock. For this site, the piles should be driven to practical refusal on the bedrock surface which appears to be at 28 m to 29 m below the existing ground surface. The piles should attain refusal at the surface of the weathered bedrock; it is likely that some limited penetration of the piles into the bedrock may occur.

For piles attaining refusal at or slightly below the bedrock surface, settlement at the toe will be negligible and the total pile head settlement will correspond to the elastic deformation of the piles.

The ultimate limit states (ULS) axial geotechnical resistance in compression of piles driven to refusal on bedrock (or slightly within) at this site should be considered to be the structural capacity of the pile.

Due to stresses imposed by the pile driving methods and to avoid damaging the steel during driving, it is recommended that the ULS geotechnical resistance be limited to 141 N/mm^2 of the steel cross-sectional area of the piles. In the case where pipe piles are to be filled with concrete and the pile driving contractor proposes higher capacities to incorporate the structural benefits of the concrete, the contractor would be required to demonstrate that the piles have achieved the proposed higher capacities by field testing.

Based on a limiting stress value of 141 N/mm^2 against steel cross-sectional area, the following ULS geotechnical resistances may be considered.

HP 310x110	1988 kN at ULS
Pipe 324 mm diameter, 11 mm thick wall	1540 kN at ULS

The actual piles selected will depend on the pile load requirements and the pile cap configurations.

Given that the potentially liquefiable layer is shallow and thin, drag loads normally associated with liquefaction is not required to be incorporated in the pile design.

It is anticipated that piles will be spaced more than three diameters apart and that pile groups will contain relative few piles. Therefore, group effects requiring reduction in pile capacities or resulting in significant ground heaving around the piles are not anticipated.

As discussed elsewhere in this report, measures (i.e. a grade raise restriction of 0.6 m) are to be undertaken at the site to prevent soil consolidation within the footprint of the proposed building. Therefore, it has been assumed that drag loads due to soil settlements may not be considered in the design.

For piles driven to bedrock, the geotechnical resistance at serviceability limit state (SLS) exceeds the ULS value and therefore is considered to be not applicable to the design.

The pile driving contractor should be required to submit the following information prior to mobilizing to the site.

- Outline of proposed pile driving equipment
- Pile driving refusal criteria to provide the ULS design value selected for the project

Pile caps/grade beams for unheated areas such as exterior structures should be provided with 1.8 m of soil cover.

10% of the driven piles should be subjected to dynamic pile testing to confirm that they are well seated on bedrock and that the pile driving strategy did damage the piles upon reaching bedrock. Dynamic testing should be carried out using a pile driving analyser (PDA).

6.10.4 Floor Slab

The recommendations provided herein are based on the assumption that the average net slab loads will not exceed 12 kPa. Should a greater average load be proposed, Stantec should review the recommendations presented herein.

The subgrade beneath the floor slab should be prepared in accordance with the recommendations included in Section 6.4. Generally, the floor slab should be supported by a minimum of 300 mm of OPSS Granular A material that is compacted to at least 100% of the material's Standard Proctor Maximum Dry Density (SPMDD).

Non-structural slab-on-grade units should 'float' independently of all load-bearing walls and columns and sawcut control joints should be provided at regular intervals and along column lines to minimize shrinkage cracking and to allow for normal differential settlement of the floor slabs.

A modulus of subgrade reaction of 30 MPa/m may be used for the floor structural design.

6.11 PARKING AREAS

Provided that subgrade preparation below pavements will comply with the requirements outlined in Section 6.4 of this report, the pavement structure provided in Table 6.2 below may be used for design.

Table 6.2: Recommended Pavement Structure

Location	Asphalt Thickness	Base Thickness OPSS Granular A (mm)	Subbase Thickness Granular B Type II (mm)
Standard Duty Parking Areas	60 mm SP12.5 mm	150	300
Heavy Duty Parking	40 mm SP12.5 mm 50 mm SP SP19.0 mm	150	400

Notes:

- The finished sub-grade surface must be compacted to achieve a minimum of 95% of the materials SPMDD immediately prior to placement of the granular materials.
- Asphalt performance grade PG 58-34 should be specified.
- The Superpave mix designs should use a Traffic Category of B.
- The compaction of the asphalt layers should be to at least 92% Maximum Theoretical Relative Density (MTRD) in accordance with OPSS 310.
- All granular materials should be in accordance with the requirements of OPSS Specification. These materials should be compacted to at least 100% of the material's Standard Proctor maximum dry density (SPMDD) in lifts no greater than 300 mm.
- A tack coat is recommended between asphalt layers and along the edges of any cuts in asphalt.
- In the event that the asphalt layer is not placed at the same time as the granular sub-base/base and the base is left exposed for a period of time, the top layer of granular material should be re-shaped, surface compacted and replaced with a fresh layer of Granular A prior to the placement of the asphalt surface.
- Control of surface water is a critical factor in achieving good performance over the pavement structure life. In this regard, the elevations of the surface of the parking areas should be designed to promote adequate surface drainage.

6.12 COLD WEATHER CONSTRUCTION

Placement of fill materials in cold weather requires a considerable increase in effort from that required in "better" weather conditions. Additional costs are typically incurred as a result, and general productivity can be expected to suffer. In addition to the prevailing weather conditions, the quantity of fill to be placed, the required lateral extent and thickness, the equipment used for placement and compaction, and the protection methods employed by the contractor, will all have an influence on the success of placing fill in adverse weather conditions.

Notwithstanding the comments provided in the previous sections of this report pertaining to backfilling and engineered fill, when construction is undertaken during periods of inclement weather or when freezing conditions exist, the placement of fill materials for any purpose should consider the comments provided below.

- Foundations shall be constructed on non-frozen ground only; where non-frozen ground includes the material at surface and all underlying soils. The non-frozen nature of the ground must be confirmed by a geotechnical inspection within 1 hour of concrete placement.

- Following construction of foundations, protection measures must be provided to prevent freezing of the foundation subgrade/bearing soils and for protection of the concrete during curing. The protective measures must also keep the subgrade soils beneath the foundations from freezing after the concrete has cured.
- Foundations shall be backfilled with free-draining granular material and drainage shall be provided to prevent lifting of the foundations due to adfreeze during the construction period.
- Structural fill shall not be placed on frozen ground and the structural fill materials shall be free of snow and frozen material.
- Overnight frost penetration into the existing sub-grade or the structural fill must be prevented. Alternatively, the frozen fill must be completely removed prior to placing subsequent lifts. Breaking the frost in-situ is not considered acceptable.
- Moisture adjustment of the fill materials (i.e. adding water or allowing fill to dry) is not practical in freezing conditions. Therefore, obtaining the required compaction levels of 98 percent of the materials Standard Proctor maximum dry density for Structural Fill will not be practical if the fill materials are not supplied to the site near their optimum water content for compaction.
- Regular checks of the temperature of the fill should be made. The soil temperature should be greater than +2C to allow for compaction to the specified degree.
- Imported fill should not be stockpiled on site in such a condition where freezing of the material in the stockpile can develop. Direct import, placement, and compaction is recommended.
- Full-time inspection and testing services is required during earthworks in winter conditions.

6.13 CEMENT TYPE AND CORROSION POTENTIAL

Two (2) tests were conducted on selected soil samples to determine the water soluble sulphate content of the site soils. The sulphate concentrations in the samples were 27 and 29 ug/g. Results of the sulphate analysis are shown in Table 5.5. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected on concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The test results provided in Table 5.5 should be used by the designers in assessing the potential for corrosion of steel elements and may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The soil pH results were 7.2 and 7.6 which is within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment. The reported resistivity of 49.6 and 150 (ohm-m) suggests a low corrosive environment.

7.0 CLOSURE

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of the St. George and St. Anthony Church, who is identified as “the Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying of unexpected site conditions
- Planning, design or construction

This report has been prepared by Ramy Saadeldin, Ph.D., P.Eng. and reviewed by Raymond Haché, M.Sc., P.Eng., ing.

Respectfully submitted,

STANTEC CONSULTING LTD.

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APPENDIX A

Statement of General Conditions

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.

APPENDIX B

Drawing No. 1 – Key Plan

Drawing No. 2 – Borehole Location Plan

Figure B1. Geotechnical Model

Figure B2. Soil Sensitivity Profile

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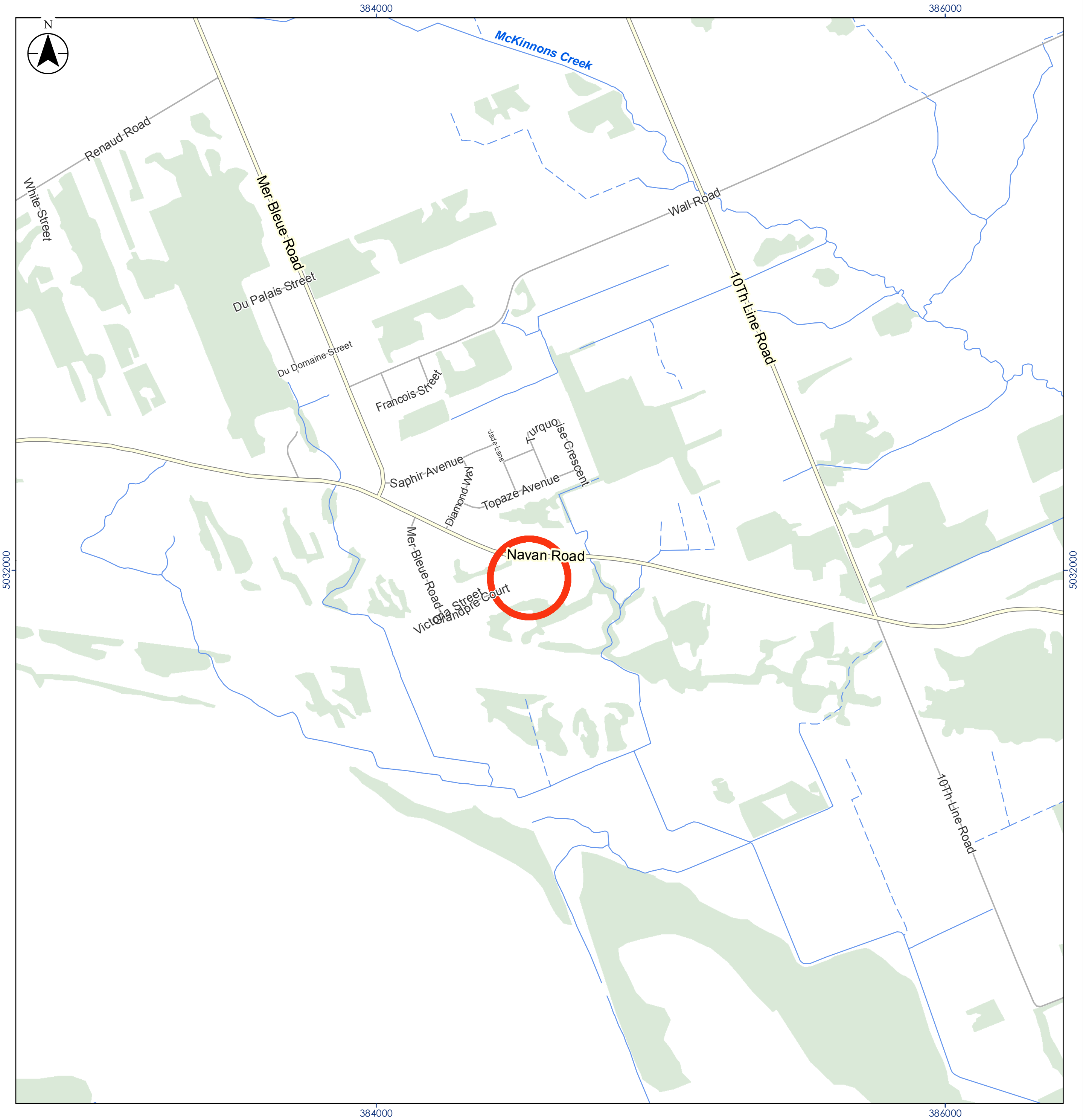

Legend

🟥 Approximate Site Location

═ Major Road

— Minor Road

- - Watercourse (Intermittent)

— Watercourse (Permanent)

🟩 Wooded Area

0 500 1,000 metres

1:20,000 (at original document size of 8.5x11)

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Notes

1. Coordinate System: NAD 1983 UTM Zone 18N.
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2016.

Project Location 3856, 3866 and 3876 Navan Road Ottawa, Ontario	Project No. 160410200 Prepared by Gliceria Briones on 2018-02-15

Client/Project
ST. GEORGE AND ST. ANTHONY CHURCH
NEW CHURCH
GEOTECHNICAL INVESTIGATION

Drawing No.
1

Title
Key Plan

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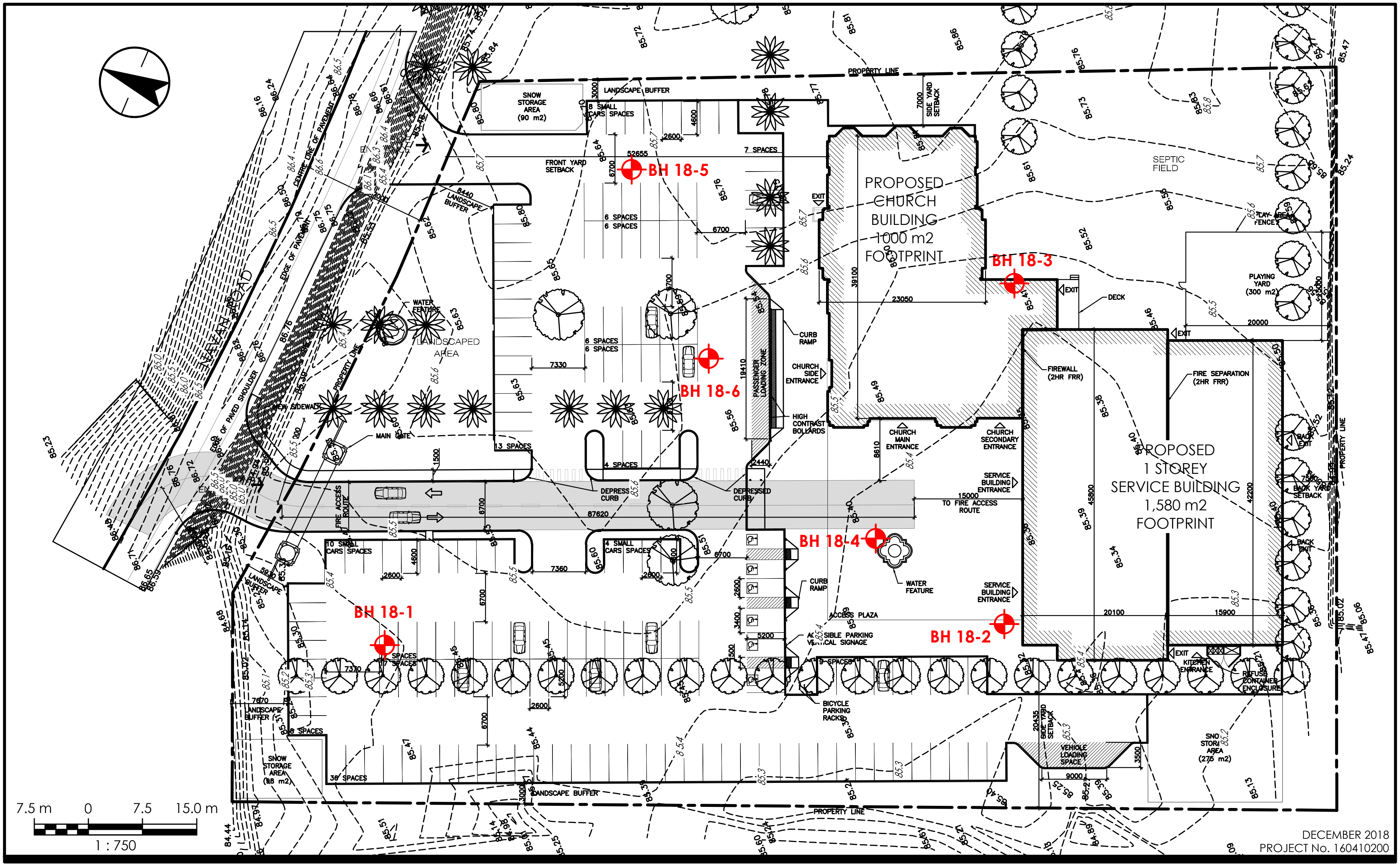
2018/12/07 2:02 PM By: Briones, Gliceria



DECEMBER 2018
PROJECT No. 160410200

Stantec

400 - 1331 Clyde Avenue
Ottawa, ON, Canada K2C 3G4
www.stantec.com

LEGEND

PROPERTY LINE

BOREHOLE

GROUND SURFACE ELEVATION (m)

GROUND SURFACE CONTOUR (m)

NOTES

1. PROPOSED SITE PLAN PROVIDED BY TEMPRANO & YOUNG ARCHITECTS INC., PROJ. No. 709.00, DWG. No. A-0, DATED 16/11/18.

Client/Project

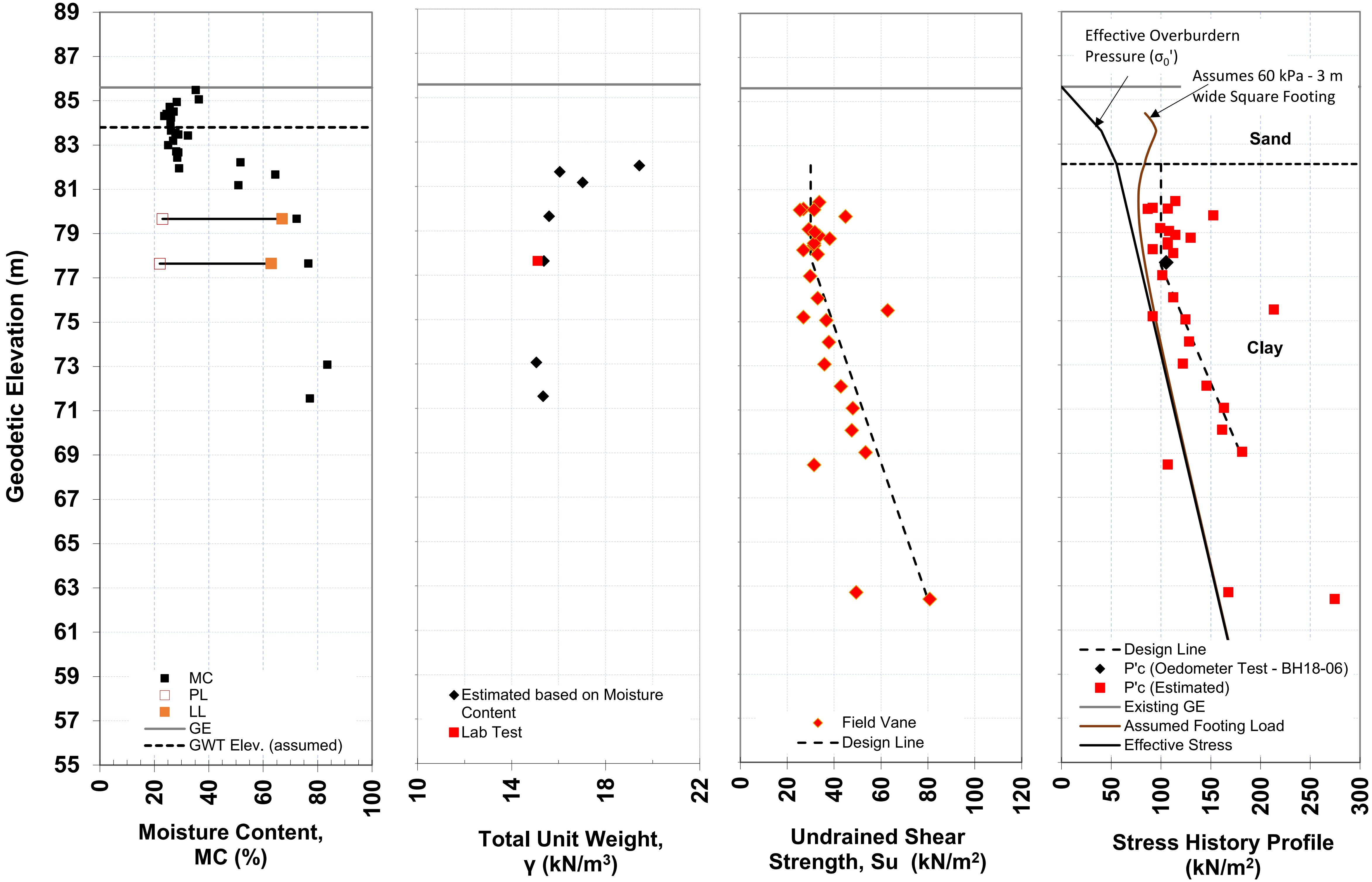
ST. GEORGE AND ST. ANTHONY CHURCH
NEW CHURCH, GEOTECHNICAL INVESTIGATION
3856, 3866 & 3876 NAVAN ROAD, OTTAWA, ON.

Drawing No.

2

Title

BOREHOLE LOCATION PLAN



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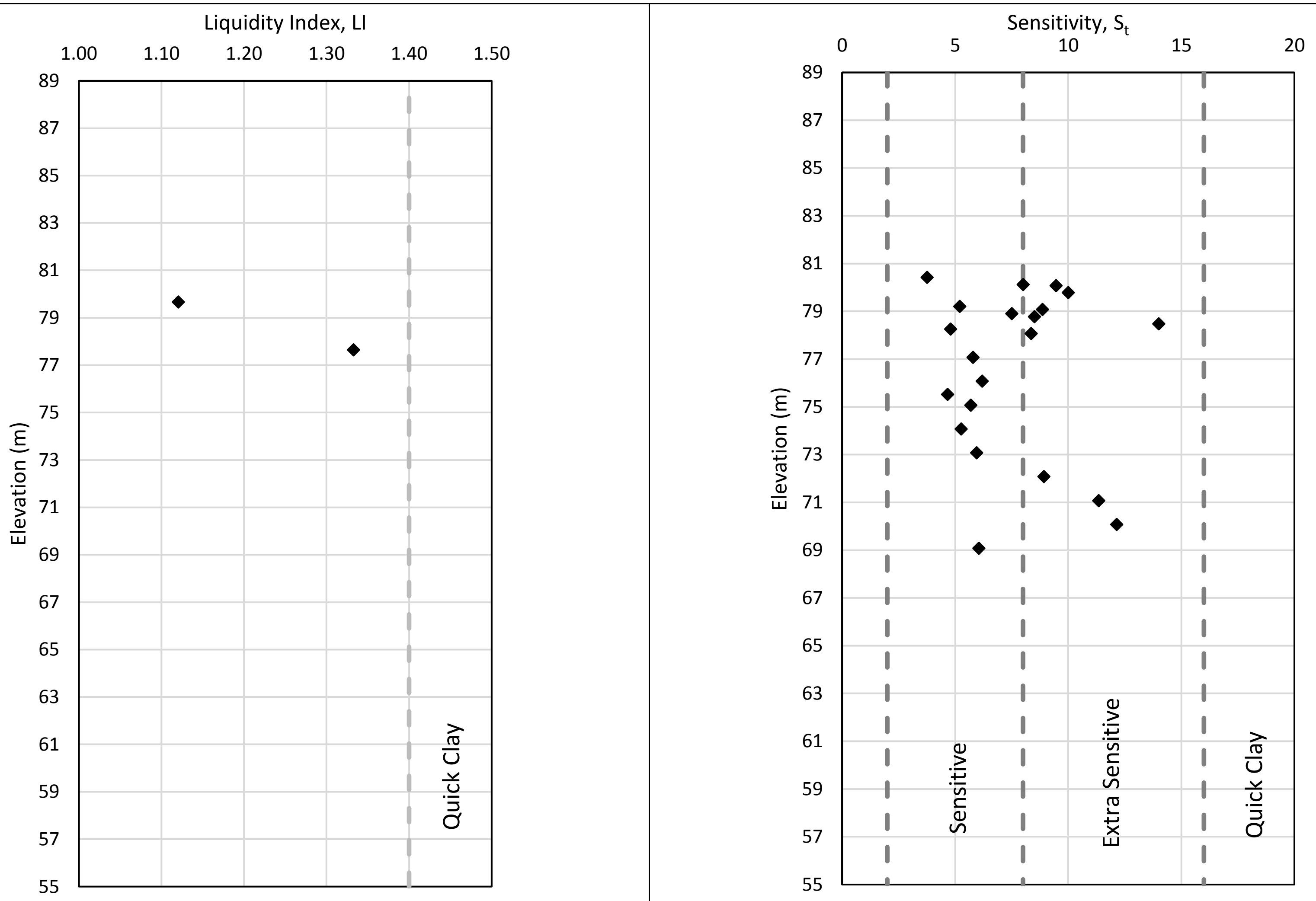
St. George and St. Anthony Church
3856, 3866 and 3876 Navan Road, Ottawa, Ontario
Geotechnical Model

Figure B1



St. George and St. Anthony Church
3856, 3866 and 3876 Navan Rd, Ottawa, Ontario
Soil Sensitivity

Figure B2

APPENDIX C

Symbols & Terms Used on the Borehole Records

Borehole Records

Bedrock Core Log and Photograph

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

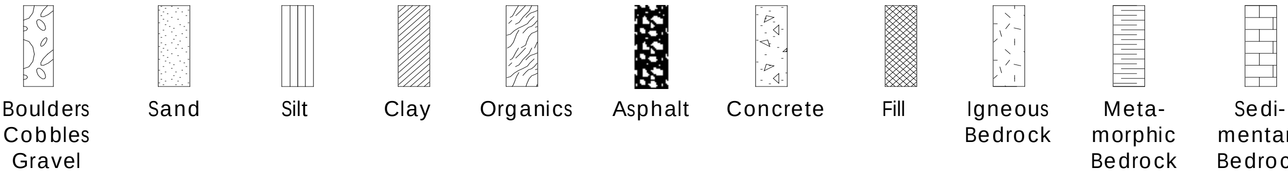
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼ measured in standpipe, piezometer, or well

▽ inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to ‘A’ size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals I_p(50) in which the index is corrected to a reference diameter of 50 mm)

↓	Single packer permeability test; test interval from depth shown to bottom of borehole
↕	Double packer permeability test; test interval as indicated
↓	Falling head permeability test using casing
↓	Falling head permeability test using well point or piezometer

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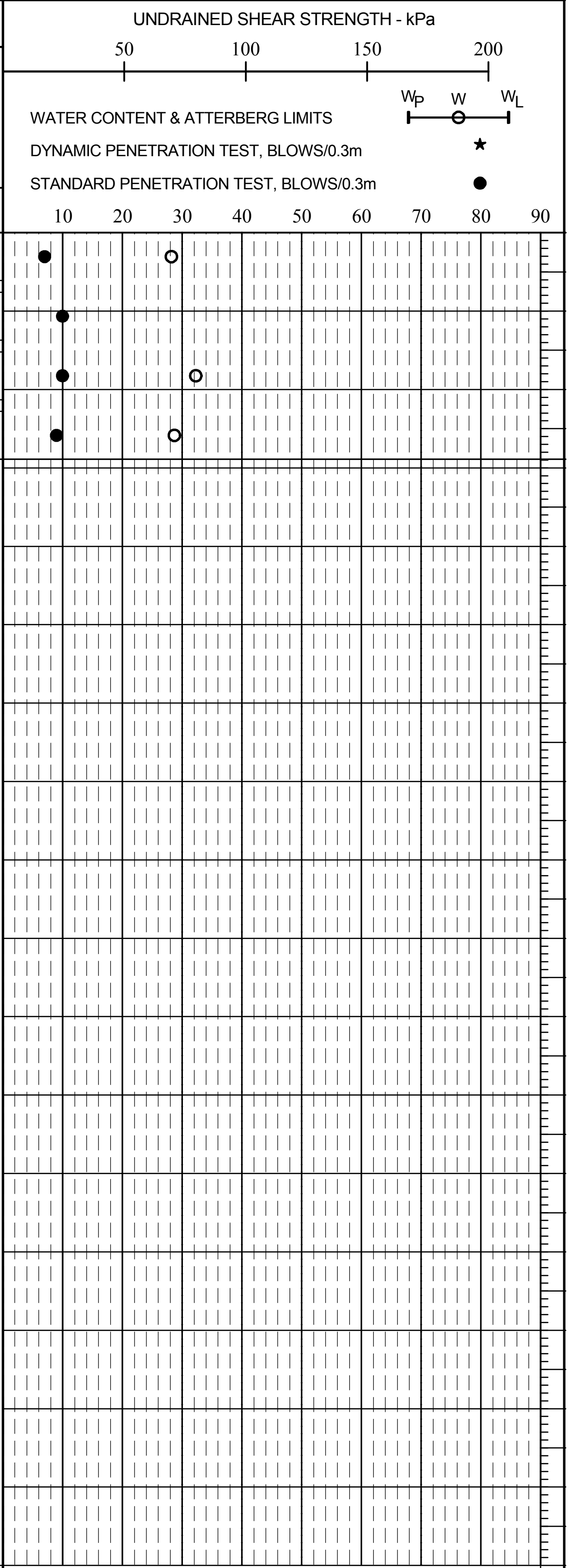
BOREHOLE RECORD

N: 5 030 115 E: 462 323

BH18-1

CLIENT Saint George & Saint Anthony Church
LOCATION Navan Road, Ottawa, Ontario
DATES: BORING February 5, 2018 WATER LEVEL N/A
BOREHOLE No. BH18-1
PROJECT No. 160410200
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	85.25							
	85.1	100 mm TOPSOIL			SS	1	300	7
1		Loose, brown, SILTY SAND (SM) - Moist to wet			SS	2	340	10
2					SS	3	350	10
	82.4				SS	4	400	9
3		End of Borehole						
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

![Stantec]

BOREHOLE RECORD

N: 5 030 037 E: 462 357

1 of 1

BH18-2

CLIENT Saint George & Saint Anthony Church

LOCATION Navan Road, Ottawa, Ontario

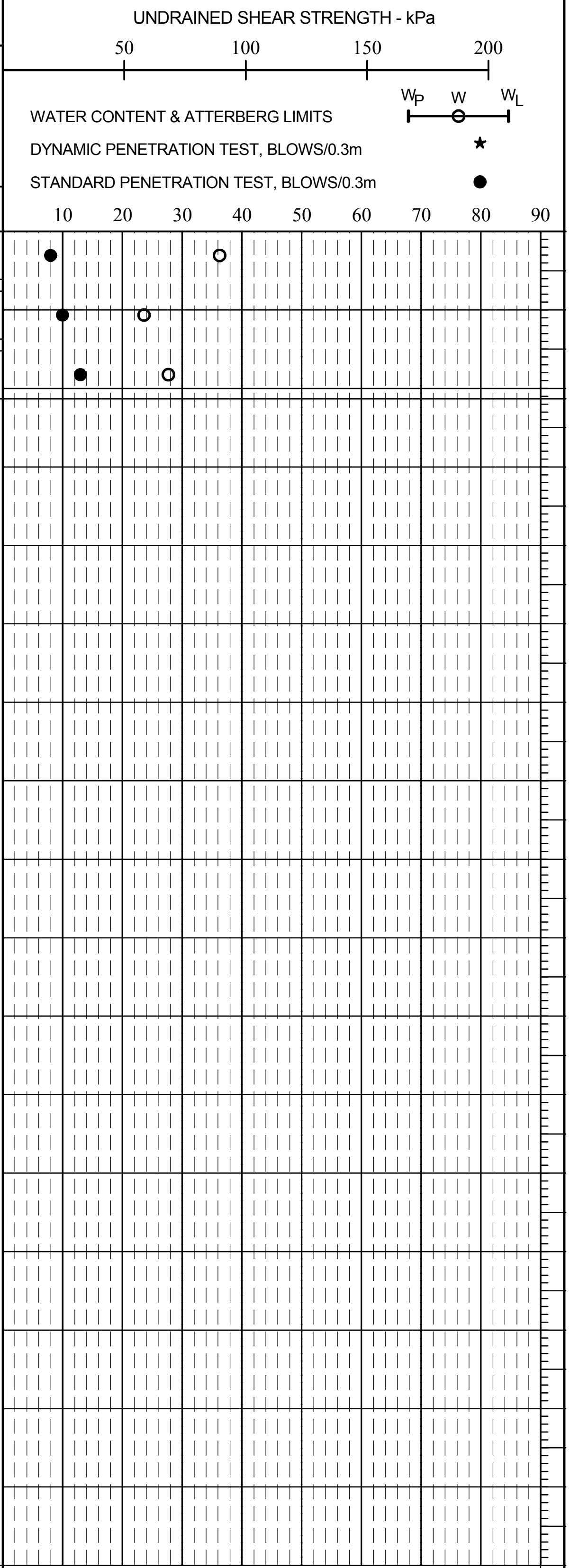
DATES: BORING February 5, 2018 WATER LEVEL N/A

BOREHOLE No. BH18-2

PROJECT No. 160410200

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES			
					TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	**85.37**							
	85.2	165 mm TOPSOIL with roolets			SS	1	150	8
1		Loose to compact. brown, SILTY SAND (SM) - Moist to wet			SS	2	200	10
2	83.2				SS	3	420	13
		End of Borehole						
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								



⊻ Inferred Groundwater Level

⯆ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

STN13-STAN-GEO 160410200 SAINT GEORGE & SAINT ANTHONY CHURCH.GPJ SMART.GDT 6/11/18

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BOREHOLE RECORD

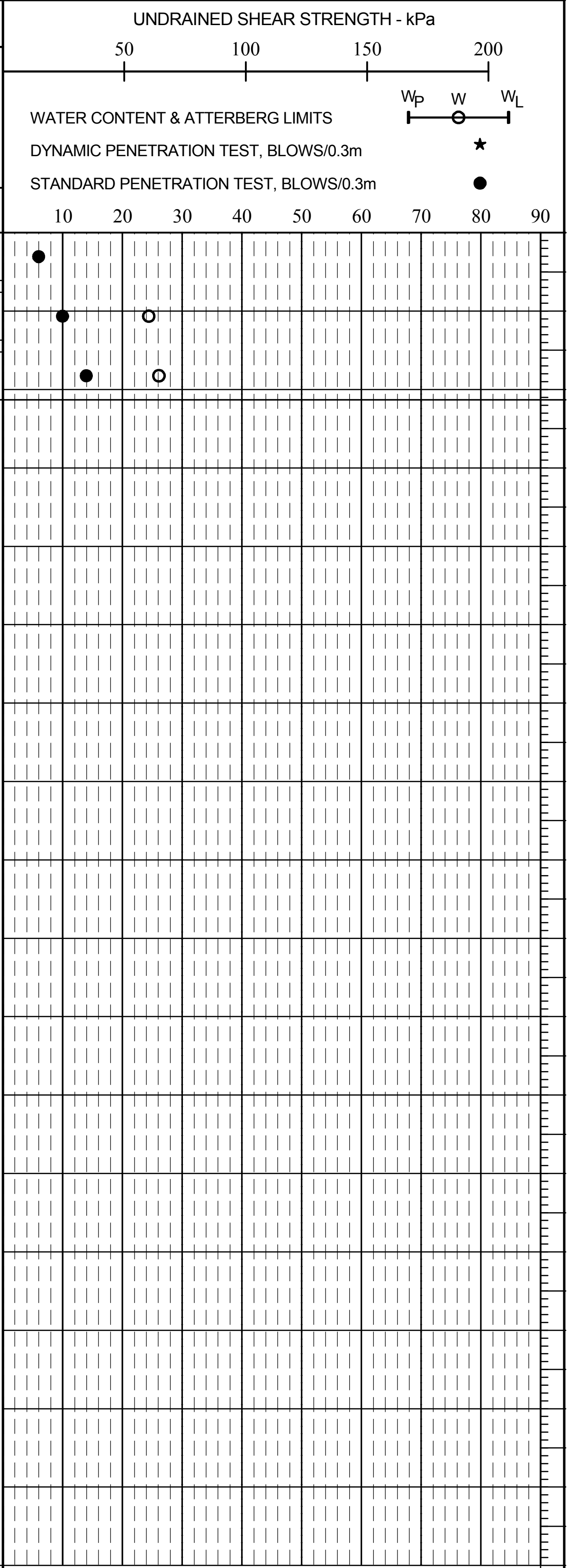
N: 5 030 054 E: 462 401

1 of 1

BH18-3

CLIENT Saint George & Saint Anthony Church
LOCATION Navan Road, Ottawa, Ontario
DATES: BORING February 5, 2018 WATER LEVEL N/A
BOREHOLE No. BH18-3
PROJECT No. 160410200
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
	85.47							
0	85.3	180 mm TOPSOIL			SS	1	380	6
1		Loose to compact, dark brown, SILTY SAND (SM) - Moist to wet			SS	2	480	10
2	83.3				SS	3	420	14
		End of Borehole						
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

Stantec

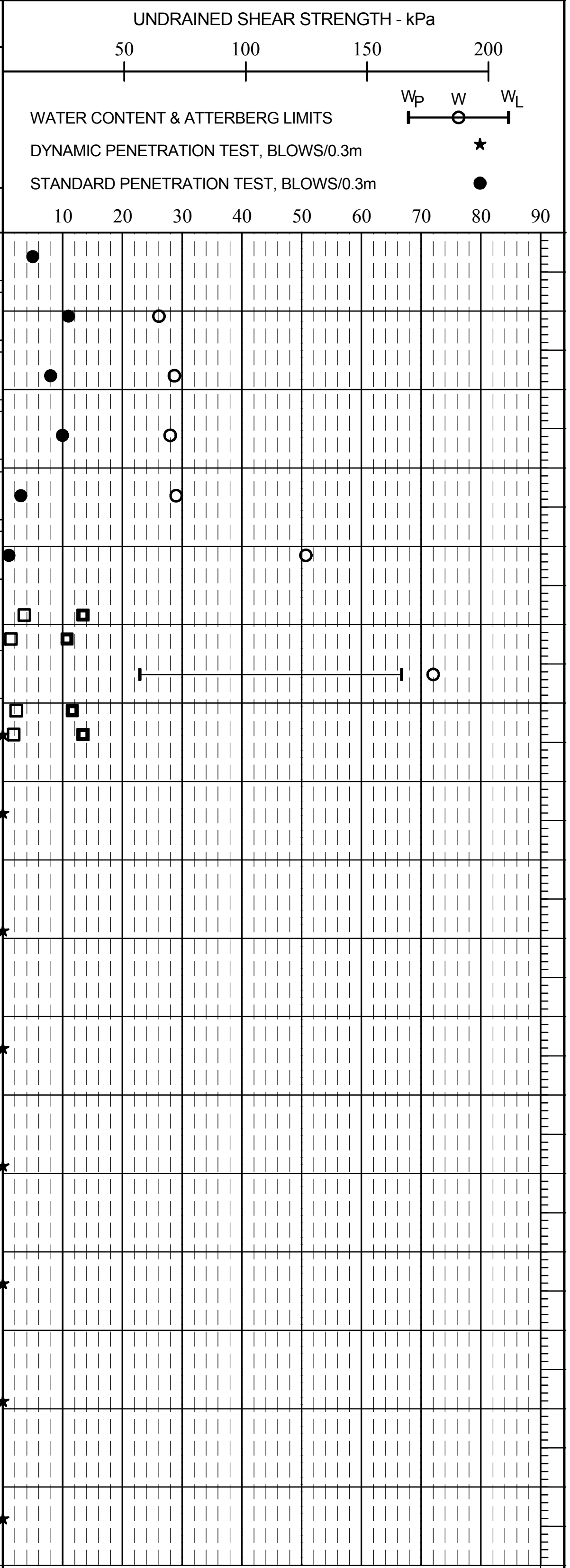
BOREHOLE RECORD

N: 5 030 054 E: 462 360

BH18-4

CLIENT Saint George & Saint Anthony Church
LOCATION Navan Road, Ottawa, Ontario
DATES: BORING February 5, 2018 WATER LEVEL N/A
BOREHOLE No. BH18-4
PROJECT No. 160410200
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	85.30							
	85.1	150 mm TOPSOIL			SS	1	400	5
		Loose to compact, dark brown, SILTY SAND (SM) - Moist to wet			SS	2	520	11
					SS	3	480	8
					SS	4	400	10
		- Becomes clayey below 3 m			SS	5	200	3
	81.3	Firm, grey, SILTY CLAY (CH) - Wet - Champlain Sea marine clay - Extra-sensitive			SS	6	350	1
					SS	7	500	WOH
	78.9	DCPT conducted below 6.4 m						



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

Stantec

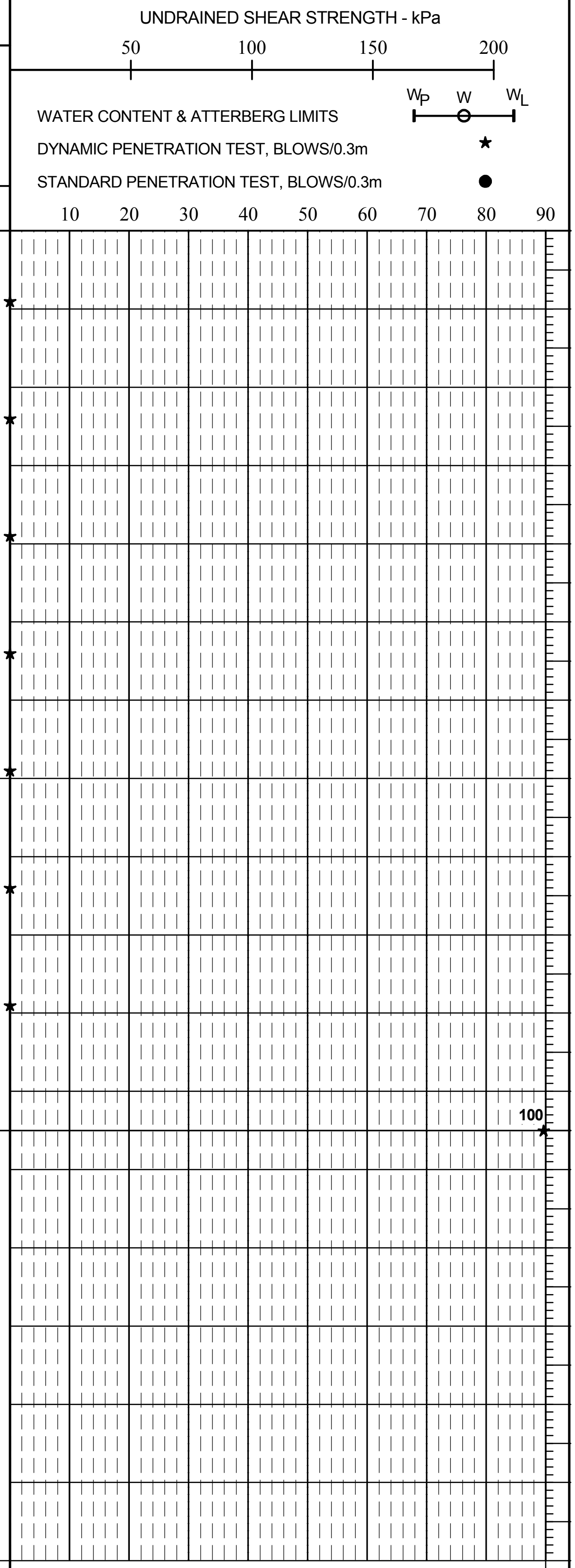
BOREHOLE RECORD

N: 5 030 054 E: 462 360

BH18-4

CLIENT Saint George & Saint Anthony Church
LOCATION Navan Road, Ottawa, Ontario
DATES: BORING February 5, 2018 WATER LEVEL N/A
BOREHOLE No. BH18-4
PROJECT No. 160410200
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES			
					TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								
28	56.8	End of Borehole DCPT refusal at 28.5 m						
29								
30								
31								
32								
33								
34								



⊻ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 030 105 E: 462 398

BH18-5

CLIENT Saint George & Saint Anthony Church

LOCATION Navan Road, Ottawa, Ontario

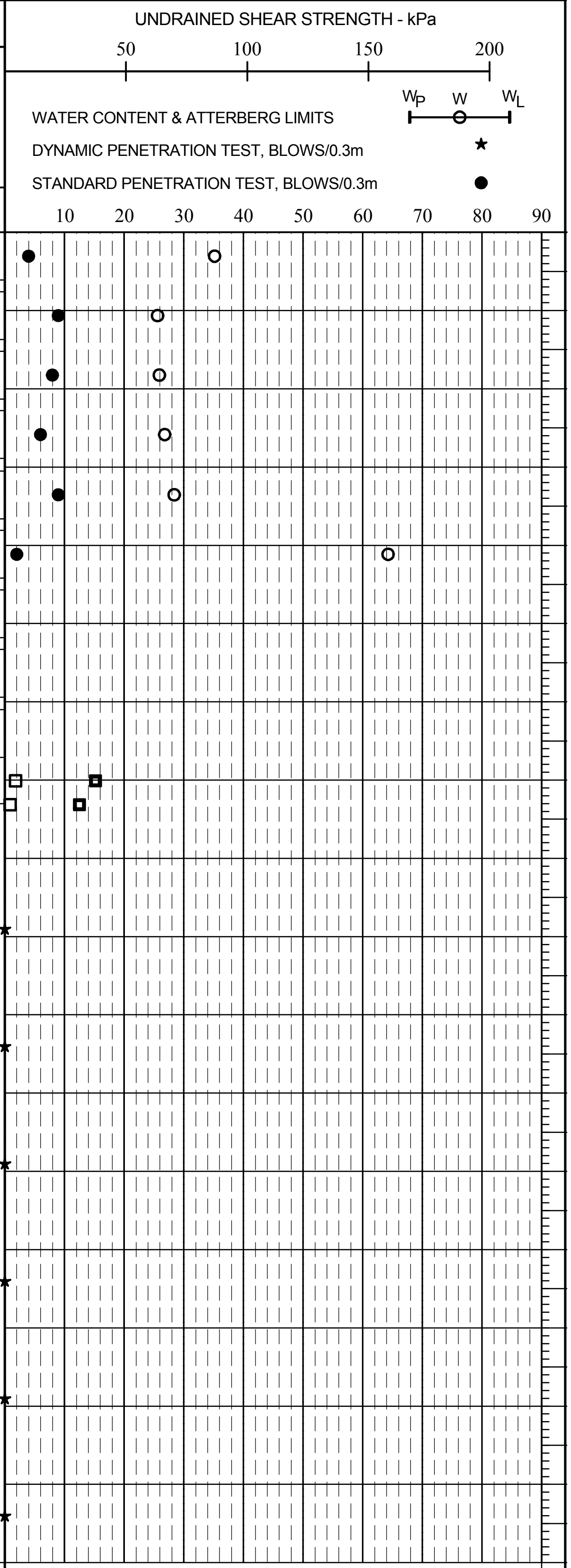
DATES: BORING February 5, 2018 WATER LEVEL 0.2m on Feb 27, 2018

BOREHOLE No. BH18-5

PROJECT No. 160410200

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
	85.78							
0	85.6	190 mm TOPSOIL		▼	SS	1	420	4
1		Loose, dark brown, SILTY SAND (SM) - Moist to wet			SS	2	500	9
2		- Becomes siltier below 2m			SS	3	510	8
					SS	4	300	6
3		- Becomes clayey below 3.5 m			SS	5	320	9
4	81.8	Firm, grey, SILTY CLAY (CH) - Wet - Champlain Sea marine clay - Extra-sensitive			SS	6	600	2
5					ST	7	0	-
6					ST	8	610	-
					ST	9	610	-
7	78.5	DCPT from 7.3 to 29 m						
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								



∑ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 030 105 E: 462 398

BH18-5

CLIENT Saint George & Saint Anthony Church

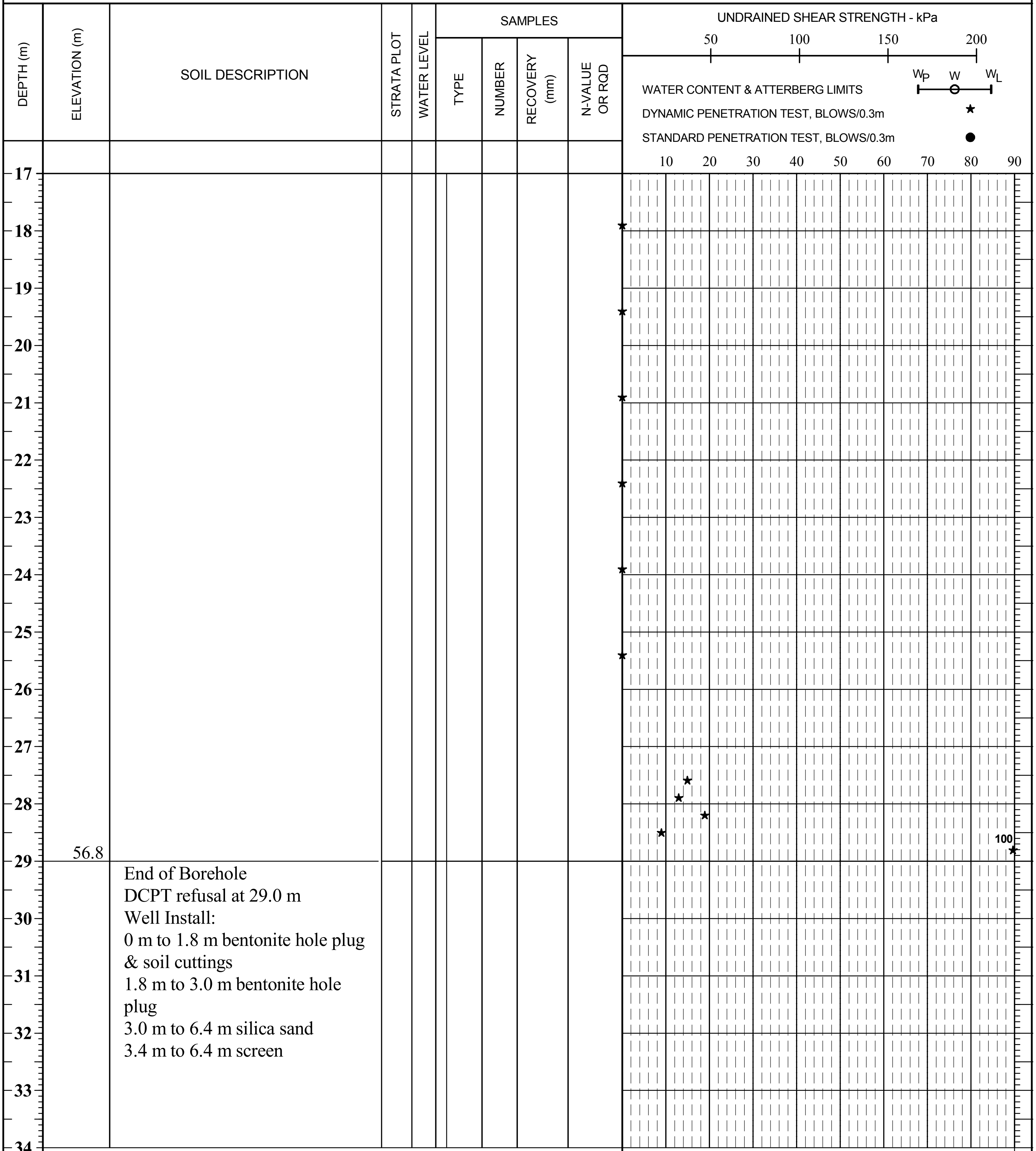
BOREHOLE No. BH18-5

LOCATION Navan Road, Ottawa, Ontario

PROJECT No. 160410200

DATES: BORING February 5, 2018 WATER LEVEL 0.2m on Feb 27, 2018

DATUM Geodetic



End of Borehole
DCPT refusal at 29.0 m
Well Install:
0 m to 1.8 m bentonite hole plug & soil cuttings
1.8 m to 3.0 m bentonite hole plug
3.0 m to 6.4 m silica sand
3.4 m to 6.4 m screen

∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

BOREHOLE RECORD

N: 5 030 087 E: 462 377

BH18-6

CLIENT Saint George & Saint Anthony Church

LOCATION Navan Road, Ottawa, Ontario

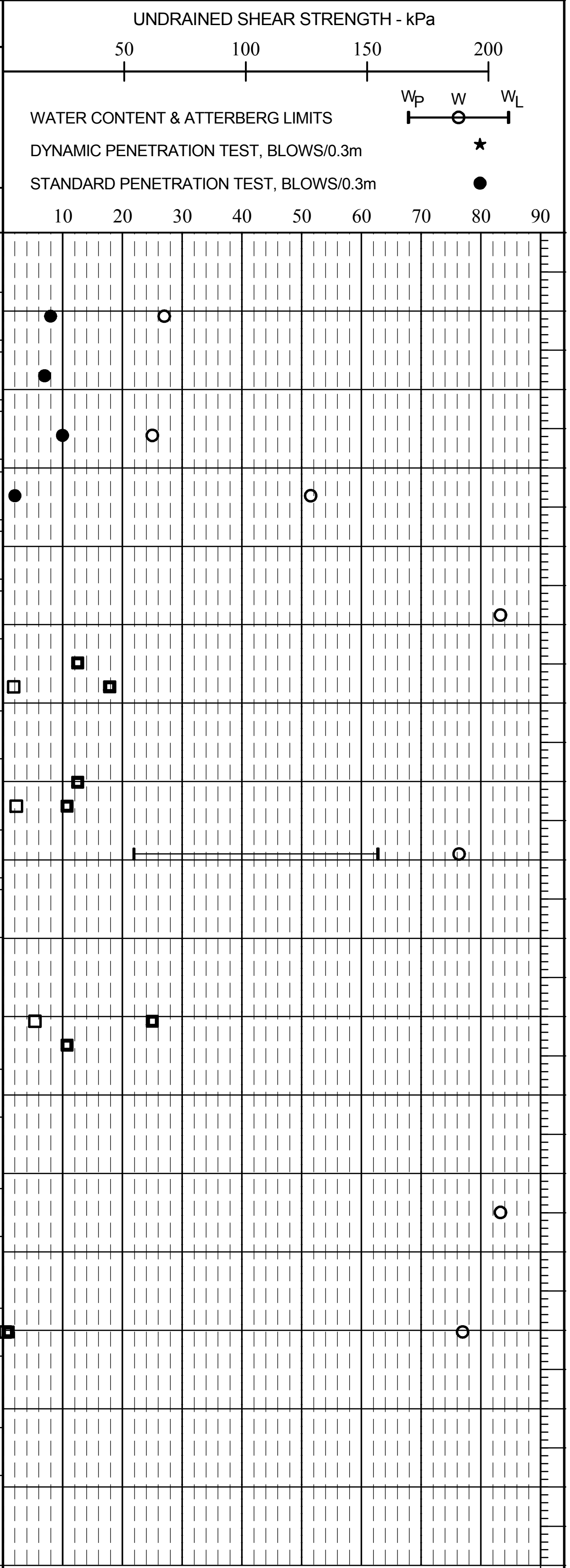
DATES: BORING February 2, 2018 WATER LEVEL N/A

BOREHOLE No. BH18-6

PROJECT No. 160410200

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	85.57							
	85.4	130 mm TOPSOIL						
		Loose, brown, SILTY SAND (SM) - Moist to wet						
1					SS	1	480	8
2					SS	2	400	7
					SS	3	320	10
3	82.4	Firm, grey SILTY CLAY (CH) - Wet - Champlain Sea marine clay - Extra-sensitive			SS	4	560	2
4					ST	5	610	-
5					SS	6	420	WOH
6					SS	7	610	-
8					ST	8	610	-
9					SS	9	600	WOH
11					ST	10	610	-
12					SS	11	300	WOH
14		- Very soft at 14 m			SS	12	600	WOH
15					SS	13	610	-
17								



▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 030 087 E: 462 377

BH18-6

CLIENT Saint George & Saint Anthony Church

LOCATION Navan Road, Ottawa, Ontario

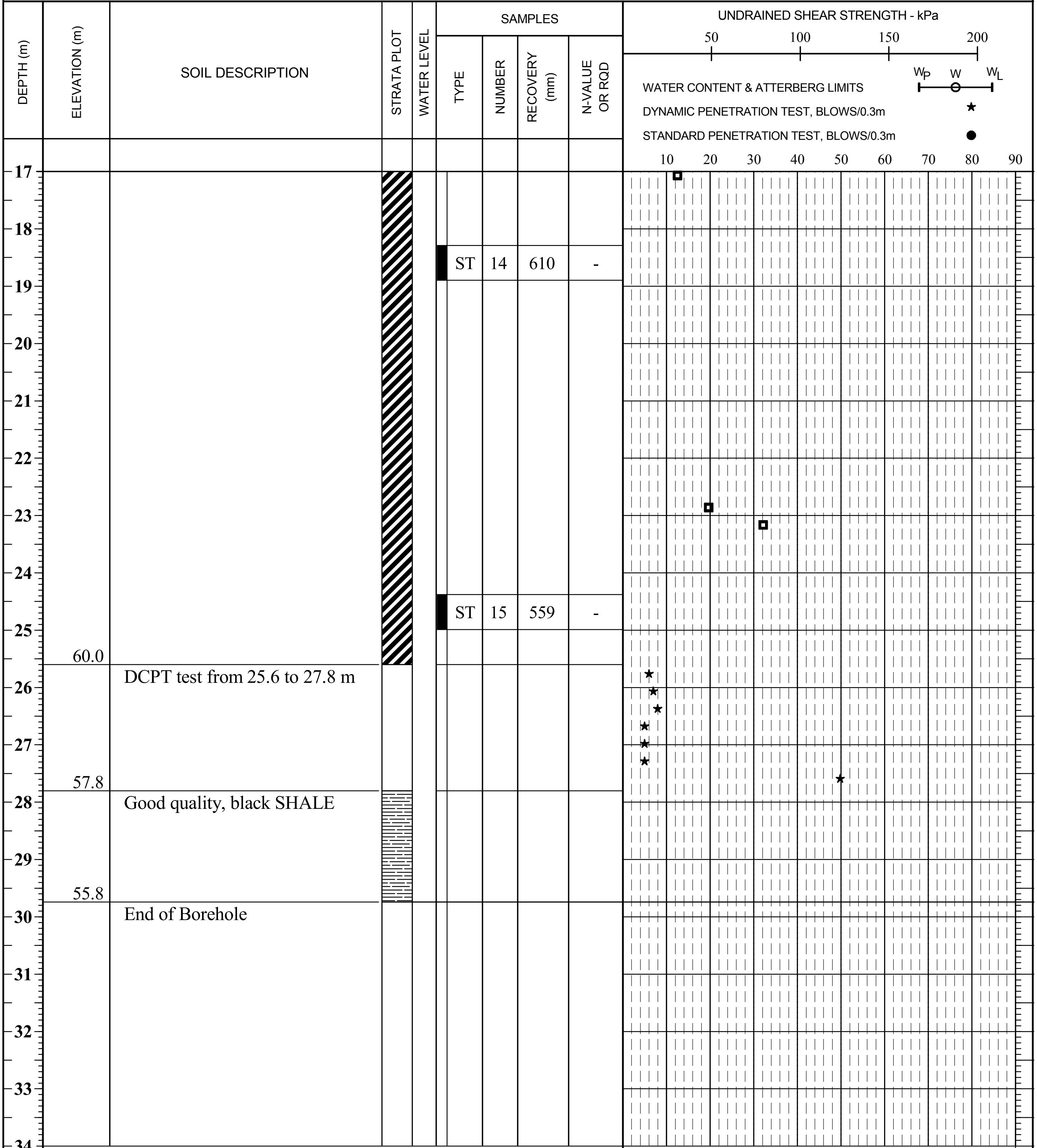
DATES: BORING February 2, 2018 WATER LEVEL N/A

BOREHOLE No. BH18-6

PROJECT No. 160410200

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
			▨		ST	14	610	-
			▨		ST	15	559	-
	60.0	DCPT test from 25.6 to 27.8 m						
	57.8	Good quality, black SHALE	☰					
	55.8	End of Borehole						



⊻ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Field Bedrock Core Log

Client: St George & St Anthony Church
Project: 3856, 3866 and 3876 Navan Road, Ottawa, Ontario
Contractor: George Downing Estate Drilling Limited

Project No.: 160410200
Date: 5-Feb-18
Borehole No.: BH18-6
Logger: JHJ

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES							OCCASIONAL FEATURES	DRILLING OBSERVATIONS
								NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING		
25.6 m	1	100%	89%	27.8 m	Medium Strong, good quality, black SHALE	R4	W2	1	JN	F	C	RU/RP	O	S	Unconfined compressive strength of 35 MPa measured on one core sample collected from a depth of 28 m	
									BD	F	M	RU/RP	O	S		

STRENGTH (MPa)

Grade/Classification	Est. Strength (MPa)
R0 Extremely Week	0.25 - 1.0
R1 Very Weak	1.0 - 5.0
R2 Weak	5.0 - 25.0
R3 Medium Strong	25.0 - 50.0
R4 Strong	50.0 - 100.0
R5 Very Strong	100.0 - 250.0
R6 Extremely Strong	>250.0

JOINT TYPE

BD = Bedding
JN = Joint
FOL = Foliation
CON = Contact
FLT = Fault
VN = Vein

ORIENTATION

F = Flat = 0-20^0
D = Dipping = 20-50^0
V = n-Vertical = >50^0

FILLING

T = Tight, Hard
O = Oxidized
SA = Slightly Altered, Clay Free
S = Sandy, Clay Free
Si = Sandy, Silty, Minor Clay
NC = Non-softening Clay
SC = Swelling, Soft Clay

WEATHERING

Grade/Classification	Description
W1 Fresh	No Visible Signs of Weathering
W2 Slightly	Discoloration, Weathering on Discontinuities
W3 Moderately	<50% of Rock Material is Decomposed, Fresh Core Stones
W4 Highly	>50% Decomposed to soil: Fresh Core Stones
W5 Completely	100% Decomposed to Soil: Original Structure Intact
W6 Residual Soil	All Rock Converted to Soil, Structure and Fabric Destroyed

DISCONTINUITY SPACING

Spacing (mm)	
EW = >6000	Extremely Wide
VW = 2000 - 6000	Very Wide
W = 600 - 2000	Wide
M = 200 - 600	Moderate
C = 60 - 200	Close
VC = 20 - 60	Very Close
EC = <20	Extremely Close

JOINT ROUGHNESS

Jr	Description
4	DJ = Discontinuous Joints
3	RU = Rough, Irregular, Undulating
1.5	SU = Smooth, Undulating
1.5	LU = Slickensided, Undulating
1.0	RP = Rough or Irregular, Planar
0.5	SP = Smooth, Planar
2	LP = Slickensided, Planar

Stantec

Project No.: 160410200

Project Name: St. George and St. Anthony Church

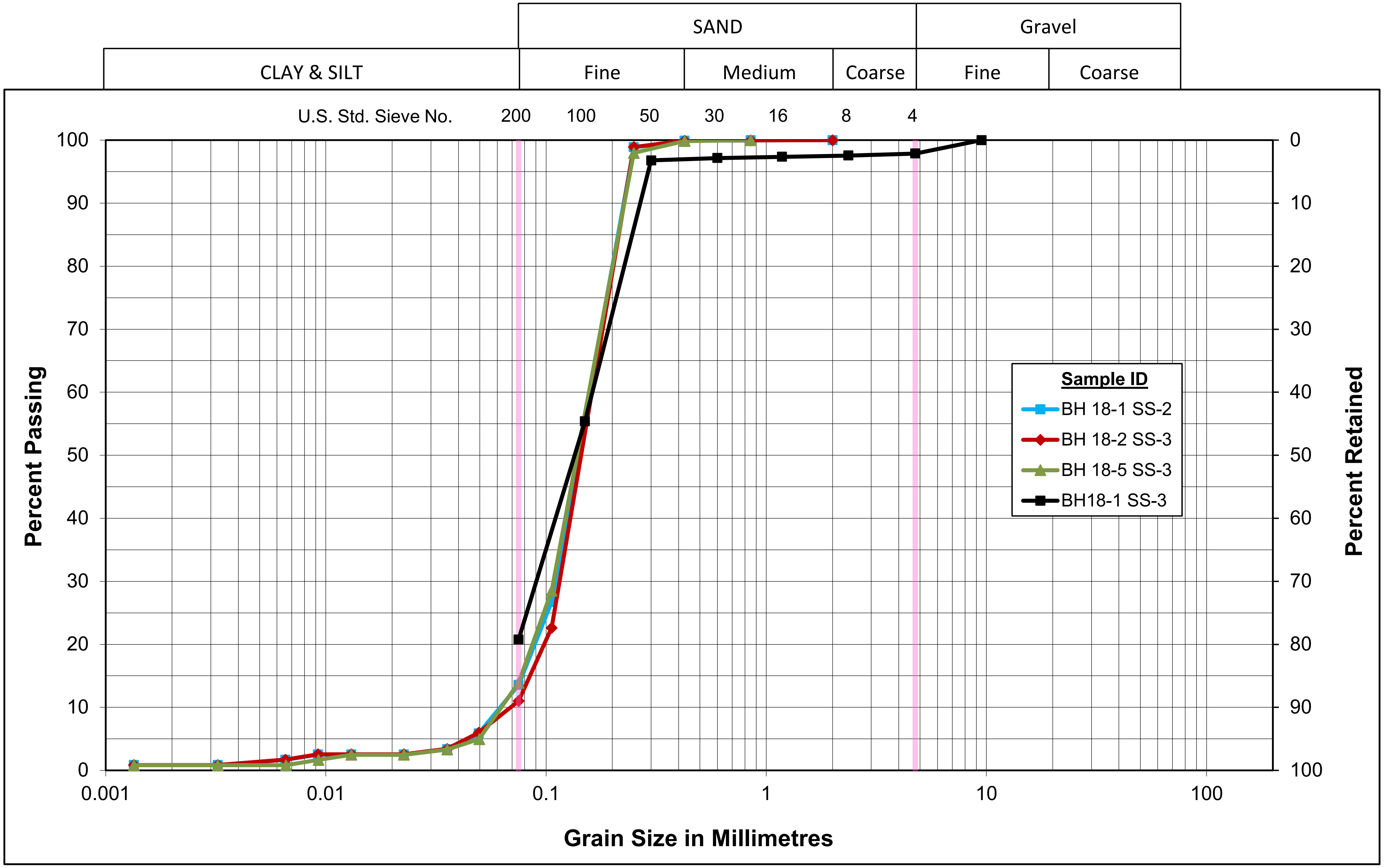
Rockcore Photographs

Rock Core Photo No.:	1	Borehole:	BH 18-6	Depth:	25.6 to 27.8 m

APPENDIX D

Laboratory Test Results

Unified Soil Classification System



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GRAIN SIZE DISTRIBUTION

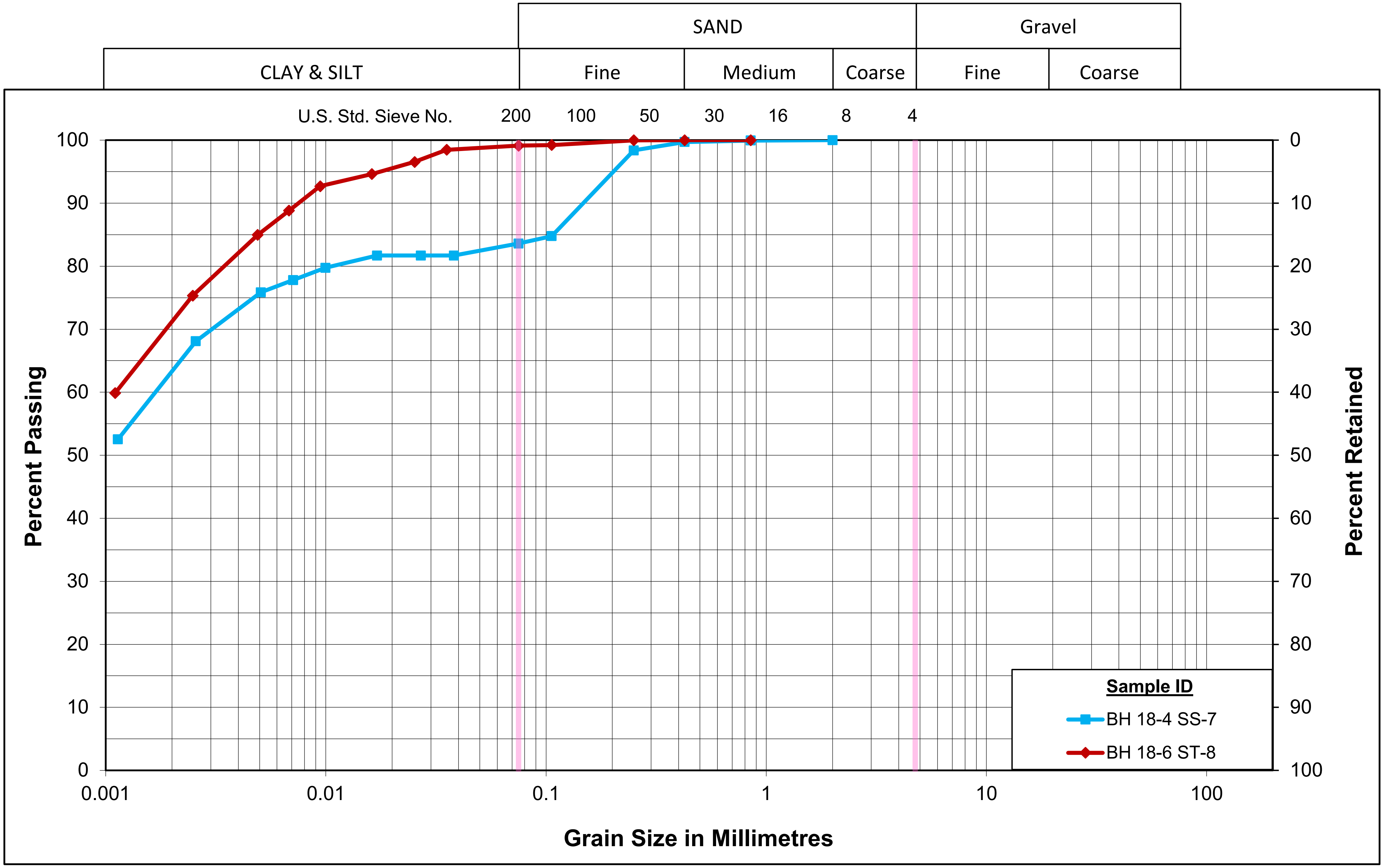
St. George & St. Anthony Coptic Othodox Church

SILTY SAND (SM)

Figure No. D1

Project No. 160410200.101.103

Unified Soil Classification System



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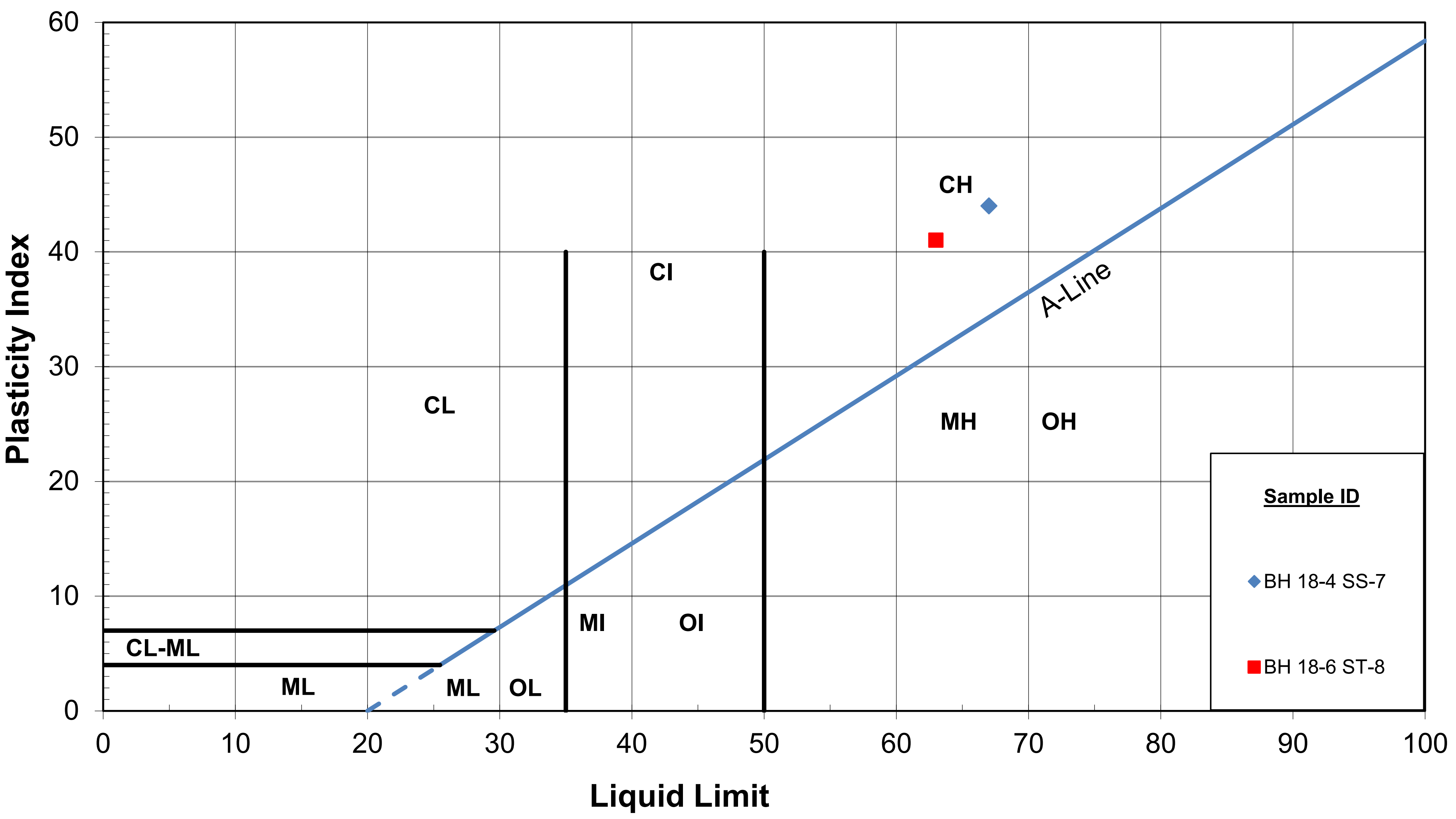
GRAIN SIZE DISTRIBUTION

St. George & St. Anthony Church

SILTY CLAY (CH) - CHAMPLAIN SEA CLAY

Figure No. D2

Project No. 160410200



Stantec

PLASTICITY CHART

St. George & St. Anthony Church

SILTY CLAY (CH) - CHAMPLAIN SEA CLAY

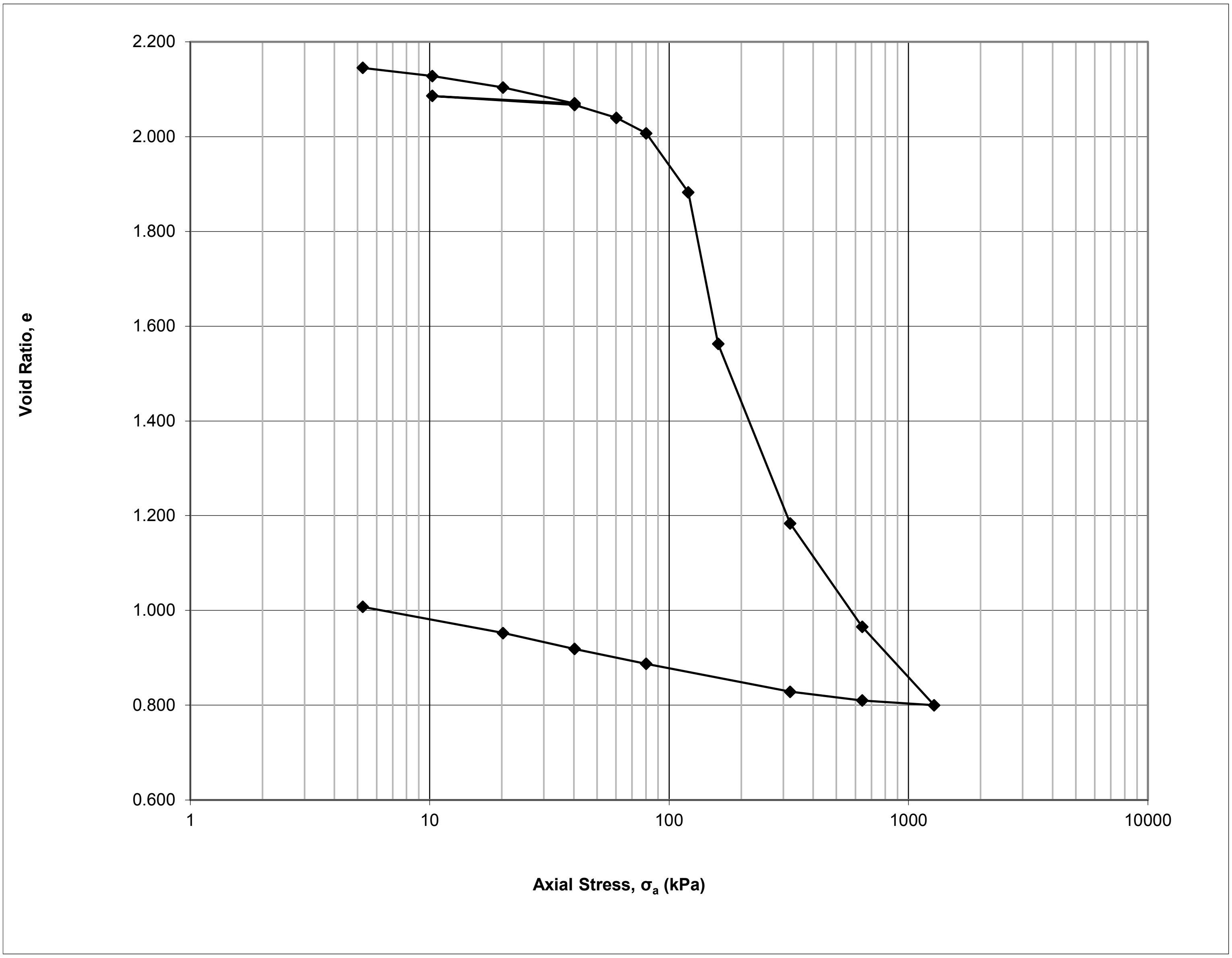
Figure No. D3

Project No. 160410200

One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D2435/D2435M - 11

Project	**3856, 3866 & 3876 Navan Road**
Project No.	**160410200**
Borehole No.	**BH 18-6**
Sample No.	**ST-8**
Sample Depth	**25-27 ft**



One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11

Specimen Details

Project Name	3856, 3866 & 3876 Navan Road
Project Location	Navan, Ottawa
Borehole	BH 18-6
Sample No.	ST-8
Depth	25-27 ft
Sample Date	February 2, 2018
Test Number	One
Technician Name	Daniel Boateng

Soil Description & Classification

Silty Clay, Brown/Grey, Fissured, Very wet - CH

Specific Gravity of Solids		2.771
Liquid Limit	%	63
Plastic Limit	%	22
Plasticity Index	%	41
Average water content of trimmings	%	77

Additional Notes (information source, occurence and size of large isolated particles etc.)

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm^2	1963
Volume	mm^3	39270
Mass	g	60.34
Dry Mass	g	34.16
Density	Mg/m^3	1.537
Dry Density	Mg/m^3	0.870
Water Content	%	76.64
Degree of Saturation	%	97.2
Height of Solids	mm	6.28
Initial Void Ratio		2.186

Final Specimen Conditions

Water Content	%	38.17
Final Void Ratio		1.008
Degree of Saturation	%	105.0
Estimated Preconsolidation Stress	kPa	98

Filename: V:\01216\active\other_pc_projects\16041020\03_(

Checked by: D. Boateng Date: 26-Feb-18

Approved by: R. Hache Date: 26-Feb-18

Filename: V:\01216\active\other_pc_projects\160410200\03_data\laboratory\internal\Consoli
Date: 26-Feb-18

Checked by: D. Boateng Date: 26-Feb-18
Approved by: R. Hache Date: 26-Feb-18

Stantec

Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11

Specimen Details

Project Name	3856, 3866 & 3876 Navan Road
Project Location	Navan, Ottawa
Borehole	BH 18-6
Sample No.	ST-8
Depth	25-27 ft
Sample Date	February 2, 2018
Test Number	One
Technician Name	Daniel Boateng

Test Procedure

Date Started		February 2, 2018
Date Finished		February 3, 2018
Machine Number		Frame D
Cell Number		D
Ring Number		D
Trimming Procedure		Turntable
Moisture Condition		Inundated
Axial Stress at Inundation	kPa	5
Water Used		Distilled
Test Method		B
Interpretation Procedure for c_v		2

All Departures from Outlined ASTM D2435/D2435M-11 Procedure

Calculations

Load Increment	Increment Duration min	Axial Stress σ_a kPa	Corrected Deformation ΔH mm	Specimen Height H mm	Axial Strain ε_a %	Void Ratio e
Seating	0.0	5	0.0000	20.0000	0.00	2.186
1	14.8	5	0.2550	19.7450	1.28	2.145
2	19.8	10	0.3619	19.6381	1.81	2.128
3	23.3	20	0.5140	19.4860	2.57	2.104
4	28.3	40	0.7254	19.2746	3.63	2.070
5	13.3	10	0.6269	19.3731	3.13	2.086
6	15.0	40	0.7464	19.2536	3.73	2.067
7	36.5	60	0.9154	19.0846	4.58	2.040
8	53.3	80	1.1214	18.8786	5.61	2.007
9	193.5	120	1.9030	18.0970	9.52	1.882
10	464.0	160	3.9130	16.0870	19.57	1.562
11	159.0	320	6.2923	13.7077	31.46	1.183
12	110.5	640	7.6585	12.3415	38.29	0.966
13	80.5	1280	8.6986	11.3014	43.49	0.800
14	13.3	640	8.6353	11.3647	43.18	0.810
15	20.0	320	8.5207	11.4793	42.60	0.828
16	43.5	80	8.1511	11.8489	40.76	0.887
17	48.3	40	7.9538	12.0462	39.77	0.919
18	78.5	20	7.7424	12.2576	38.71	0.952
19	117.5	5	7.3943	12.6057	36.97	1.008

Checked by: D. Boateng Date: 26-Feb-18

Approved by: R. Hache Date: 26-Feb-18

Filename: V:\01216\active\other_pc_projects\160410200\03_data\laboratory\internal\Conso

Date: 26-Feb-18

One-Dimensional Consolidation Test using Incremental Loading

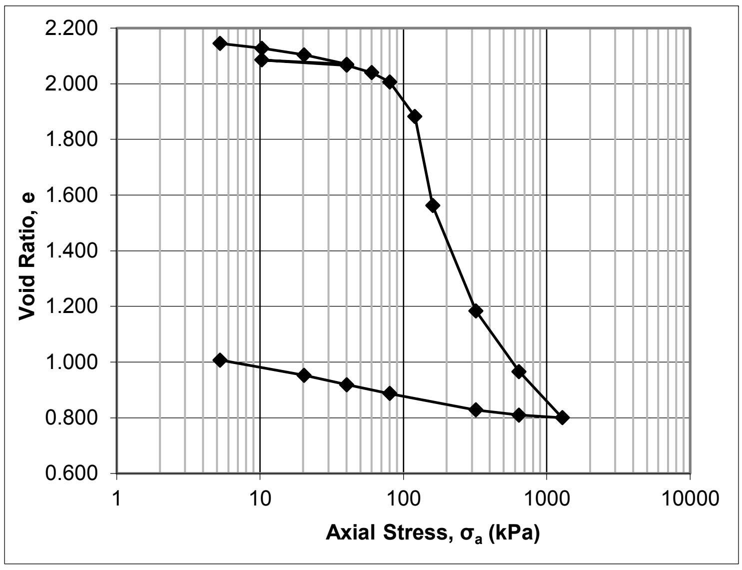
ASTM D2435/D2435M - 11

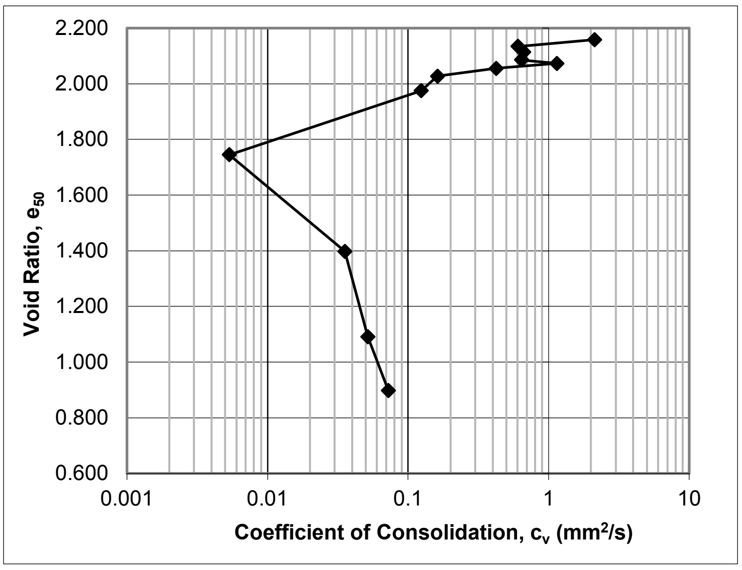
Specimen Details

Project Name	3856, 3866 & 3876 Navan Road
Project Location	Navan, Ottawa
Borehole	BH 18-6
Sample No.	ST-8
Depth	25-27 ft
Sample Date	February 2, 2018
Test Number	One
Technician Name	Daniel Boateng

Calculations

		Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
Load Increment	Axial Stress $\sigma_{a,\ average}$ kPa	Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\varepsilon_{a,50}$ %	Void Ratio e_{50}	Time t_{50} sec	Coeff. Consol. c_v mm^2/s	Time t_{90} sec	Coeff. Consol. c_v mm^2/s
Seating	3								
1	5	0.1688	19.8312	0.84	2.159			39	2.13E+00
2	8	0.3207	19.6793	1.60	2.134			136	6.03E-01
3	15	0.4504	19.5496	2.25	2.114			122	6.63E-01
4	30	0.6262	19.3738	3.13	2.086			124	6.43E-01
5	25	0.6457	19.3543	3.23	2.083				
6	25	0.7042	19.2958	3.52	2.073			69	1.14E+00
7	50	0.8147	19.1853	4.07	2.056			184	4.24E-01
8	70	0.9923	19.0077	4.96	2.027			473	1.62E-01
9	100	1.3219	18.6781	6.61	1.975			598	1.24E-01
10	140	2.7664	17.2336	13.83	1.745			11771	5.35E-03
11	240	4.9475	15.0525	24.74	1.397			1352	3.55E-02
12	480	6.8725	13.1275	34.36	1.091			706	5.17E-02
13	960	8.0794	11.9206	40.40	0.899			417	7.22E-02
14	960	8.6533	11.3467	43.27	0.807				
15	480	8.5616	11.4384	42.81	0.822				
16	200	8.2984	11.7016	41.49	0.864				
17	60	8.1380	11.8620	40.69	0.889				
18	30	7.9464	12.0536	39.73	0.920				
19	13	7.7313	12.2687	38.66	0.954				





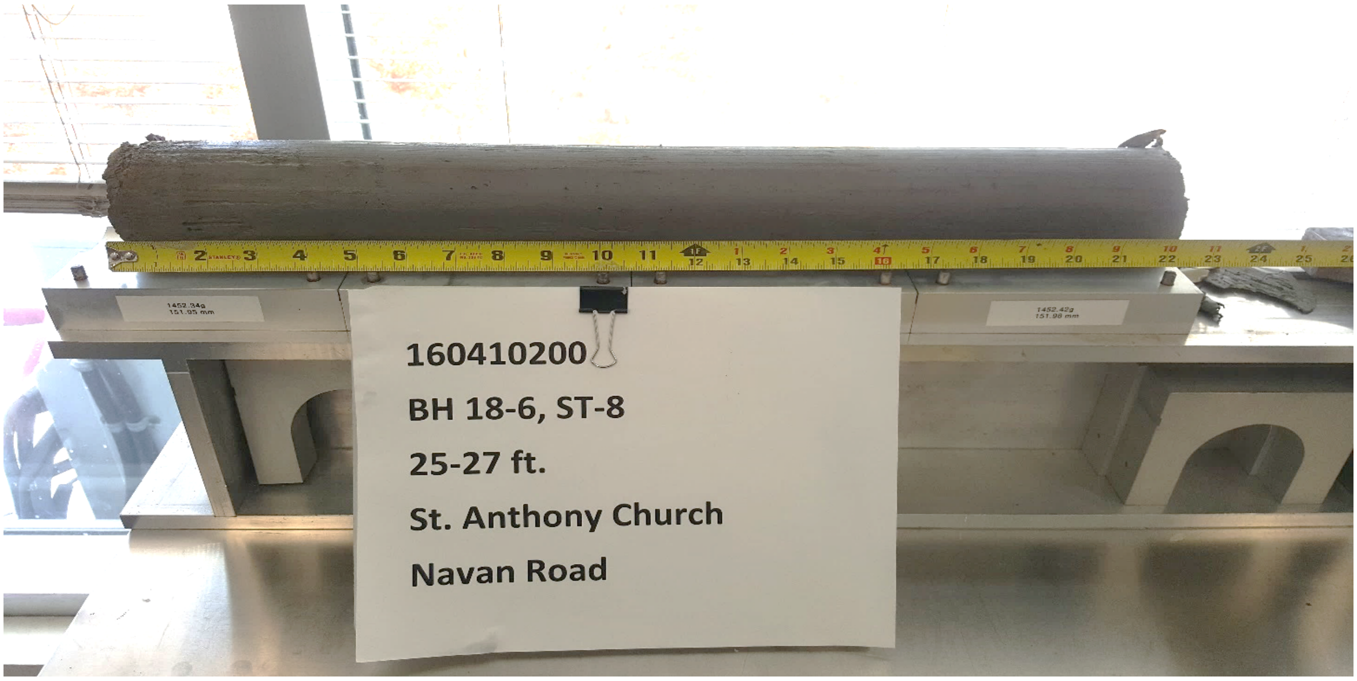


Photo No.:	1	Borehole:	BH 18-6, ST-8	Depth:	25 – 27 ft.

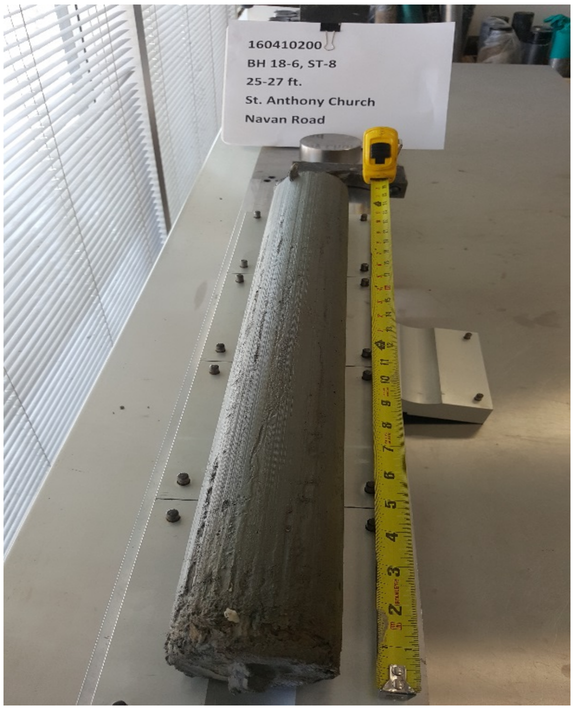


Photo No.:	2	Borehole:	BH 18-6, ST-8	Depth:	25 – 27 ft.

Photo No.: 3 Borehole: BH 18-6, ST-8 Depth: 25 – 27 ft.

Photo No.: 4 Borehole: BH 18-6, ST-8 Depth: 25 – 27 ft.

APPENDIX E

Laboratory Chemical Analysis Results

PARACEL LABORATORIES LTD.
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Certificate of Analysis

Stantec Consulting Ltd. (Ottawa)

2781 Lancaster Road, Suite 101
Ottawa, ON K1B 1A7
Attn: Rami Saadeldin

Client PO:
Project: 160410200
Custody:

Report Date: 22-Feb-2018
Order Date: 15-Feb-2018

Order #: 1807384

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

Paracel ID	Client ID
1807384-01	BH 18-05, SS-4
1807384-02	BH 18-06, SS-1

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	21-Feb-18	21-Feb-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	16-Feb-18	16-Feb-18
Resistivity	EPA 120.1 - probe, water extraction	21-Feb-18	21-Feb-18
Solids, %	Gravimetric, calculation	16-Feb-18	16-Feb-18

	Client ID:	BH 18-05, SS-4	BH 18-06, SS-1	-	-
	Sample Date:	05-Feb-18	02-Feb-18	-	-
	Sample ID:	1807384-01	1807384-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	79.2	80.6	-	-
General Inorganics					
pH	0.05 pH Units	7.55	7.22	-	-
Resistivity	0.10 Ohm.m	49.6	150	-	-
Anions					
Chloride	5 ug/g dry	109	9	-	-
Sulphate	5 ug/g dry	27	29	-	-

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	100	5	ug/g dry	109			8.2	20	
Sulphate	25.2	5	ug/g dry	27.1			7.5	20	
General Inorganics									
pH	7.22	0.05	pH Units	7.24			0.3	10	
Resistivity	4.12	0.10	Ohm.m	4.10			0.5	20	
Physical Characteristics									
% Solids	83.7	0.1	% by Wt.	84.5			0.9	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	211	5	ug/g	109	102	78-113			
Sulphate	135	5	ug/g	27.1	108	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.

APPENDIX F

NBC Seismic Hazard Calculation Data Sheet

Figure F1: Factor of Safety against Liquefaction

2015 National Building Code Seismic Hazard Calculation

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

March 09, 2018

Site: 45.4234 N, 75.4811 W User File Reference:

Requested by: ,

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

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.526	0.608	**0.502**	0.377	**0.264**	**0.128**	**0.060**	**0.016**	**0.0056**	**0.321**	**0.218**

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

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.048	0.169	0.287
Sa(0.1)	0.066	0.210	0.343
Sa(0.2)	0.059	0.178	0.288
Sa(0.3)	0.046	0.136	0.217
Sa(0.5)	0.033	0.095	0.152
Sa(1.0)	0.016	0.047	0.074
Sa(2.0)	0.0063	0.021	0.034
Sa(5.0)	0.0013	0.0049	0.0085
Sa(10.0)	0.0006	0.0019	0.0033
PGA	0.035	0.113	0.185
PGV	0.022	0.073	0.121

References

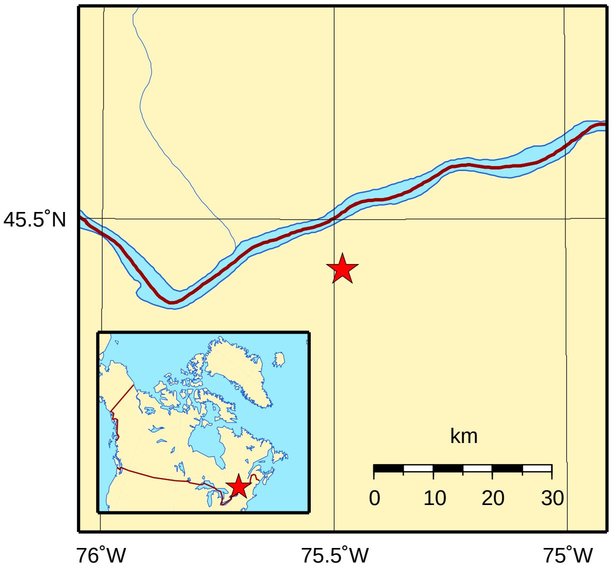
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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

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

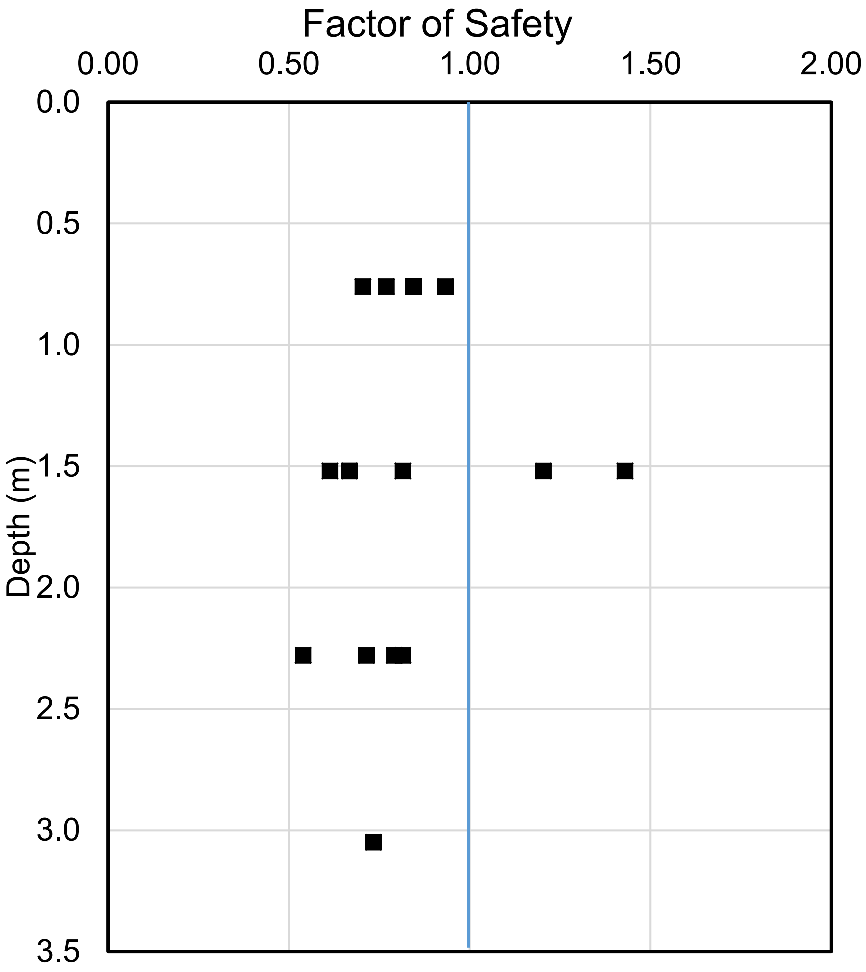
See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français



Natural Resources Canada Ressources naturelles Canada

Canada



St. George and St. Anthony Church
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Factor of Safety against Liquefaction

Figure F1