Geotechnical Investigation Report

Proposed Building
St. Patrick's Home of Ottawa, Ontario

Prepared for:
St. Patrick's Home of Ottawa
2865 Riverside Drive
Ottawa, ON K1V 6M7

Prepared by:
Stantec Consulting Ltd.
1331 Clyde Avenue
Ottawa, ON K2C 3G4

Project No. 121624271

November 2022

Table of Contents

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) has been retained by St. Patrick's Home of Ottawa (the Client) to carry out a geotechnical investigation for a new 7-storey building to be constructed at 2865 Riverside Drive, 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 property is approximately 262,600 square feet (24,400 m^2) and located along the east side of Riverside Dr. The property currently includes an existing building (on the south side of the property), as well as surface parking and greenspace (on the north side of the property). The intent is to replace the existing greenspace area on the north side of the property with a new 7-story apartment building, with a single below grade basement level.

The proposed building footprint is shown on the attached Drawing No. 1 (base plan provided by the client).

South of the current project site is currently home to an existing 5-story building. There are two driveways allowing access to the facility one running along the north of the property and one in the middle between the greenspace and the southwest property line. The existing parking for the facility is situated along the east side of the property and runs from the northern boundary to the existing care facility. The footprint of the proposed development is located between the driveway's, parking lot and Riverside Dr. and has multiple pathways and trees with most of the area being grass.

3.0 BACKGROUND INFORMATION

Prior to 2014, the site was occupied by the former Saint-Patrick's Home building, which consisted of a large multi-storey building with a footprint of approximately 4,500 m^2, including a 1986 expansion near the east end of the property. The original building, prior to the 1986 expansion, was constructed in the early 1960s and was administered by Grey Sisters of the Immaculate Conception, containing a convent on the upper fourth floor. Details of the former four-storey building are not known; however, it is anticipated it would have included a basement level. In 2014, after completion of the current Saint-Patrick's Home to the south of the project site, the former building was removed. Drawing No. 1 in Appendix B includes the outline of the former building and shows part of the currently proposed project being within the footprint of the original building constructed in the early 1960s.

Based on available information obtained from the Geological Survey of Canada (GSC) *Surficial Materials and Terrain Features*, glacial deposits of till (a heterogenous mixture of material ranging from clay to large boulders) can be expected in the area on the west end of the site. On the east end of the site, it is expected to encounter abandoned river channel deposits consisting of silt and silty clay with lenses of sand generally underlain by medium grade sand.

According to the OGS 1:250 000 scale map of the *Bedrock Geology of Ontario,* the bedrock at the site is anticipated to be limestone, dolostone, shale, arkose, or sandstone of the Ottawa Group, Simcoe Group, or Shadow Lake Formation. The bedrock geology map produced in Canadian Geology Society, paper 77-11, by Bélanger and Harrison suggests that the site is underlain by limestone with shaley partings and shows a splay from the Gloucester Fault extending in the north-south direction in the general area of the site.

A review of the geotechnical report provided along with the RFP titled "*Geotechnical Investigation for the Proposed Phase 1 Redevelopment St. Patrick's Home of Ottawa at 2865 Riverside Drive Ottawa Ontario"* and dated "August 2010" was conducted. Based on the review of boreholes 1 to 5, advanced near the proposed building footprint, the following subsurface conditions are expected:

- Topsoil; underlain by
- A weathered crust of Leda clay extending to a depth of 1.1 m to 4.2 m below ground surface (BGS). The clay crust is expected to be stiff (measured undrained shear strength of 58 kPa to 95 kPa); underlain by
- A layer of grey Leda clay extending to 4.9 m to 6.4 m BGS with an expected consistency of firm to stiff (measured undrained shear strength of about 50 kPa); underlain by
- A layer of loose to very dense glacial till with a thickness of 1.0 m to 3.7 m. Seams of sand and sand with gravel should be expected within the glacial till; underlain by
- The bedrock at depths ranging from 5.7 m to 10.2 m BGS, which consisted of horizontally bedded limestone with shale pairings. The core recovered from boreholes were of poor to excellent quality.

The groundwater was measured to be at 2.6 m to 3.8 m depths at installed monitoring wells on October 27, 2008.

As the new seven 7-story structure is sited on top of the previously demolished structure, fill materials is expected to be encountered in the area of the previous structure to either the depth of the previous foundations or to any subgrade material placed when the previous structure was erected.

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 seven boreholes, designated as BH22-1 to BH22-7, was carried out from August 11 to 15, 2022. The approximate borehole locations are shown on Drawing No. 1.

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

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 while conducting Standard Penetration Tests (SPTs) in accordance with the procedures outlined in ASTM specification D1586. In-situ shear vane measurements were conducted within the cohesive soil deposit using a field vane test. Coring was carried out in boreholes BH22-2 and BH22-6 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.

Two Vibrating Wire (VW) piezometers were installed in borehole BH22-1 and BH22-4 to facilitate the measurement of the groundwater level at the site. The 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	
BH22-1	5024344.186	445985.769	81.0
BH22-2	5024349.057	446036.219	80.6
BH22-3	5024330.259	446048.585	81.0
BH22-4	5024313.194	446037.766	80.7
BH22-5	5024328.235	446016.258	81.4
BH22-6	5024324.744	445990.233	81.3
BH22-7	5024322.11	445970.074	81.4

4.2 LABORATORY TESTING

The following geotechnical laboratory testing was performed on selected samples:

- Moisture contents;
- Grain size distribution/hydrometer analyses; and
- Uniaxial Compressive Strength (UCS) tests on bedrock core samples.

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, Atterberg Limits tests, and UCS 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 encountered at the specific locations drilled. 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 fill materials that is underlain by a Champlain Sea clay deposit followed by till materials over shaley limestone bedrock.

5.2.1 Topsoil

The thickness of the topsoil was measured to be approximately 200 mm to 1000 mm at the surface of all borehole locations. The topsoil was generally brown in colour and mixed with gravel.

5.2.2 Fill Material

A layer of fill material was encountered beneath the topsoil and extended to depths of approximately 0.9 m to 3.0 m below ground surface. The nature of the fill is inconsistent, particularly at the boreholes drilled within the former building area.

Boreholes BH22-2 to BH22-5 were drilled within the former building footprint, and the bottom of the fill was encountered at depths ranging from 2.2 m to 3.0 m below ground surface. Boreholes BH22-1, BH22-6, and BH22-7 were drilled west of the former building location, and the bottom of the fill was encountered at depths ranging from 0.9 m to 1.5 m.

The fill was generally granular (non-cohesive) and composed of a brown sand with trace gravel to sand & gravel. Within BH22-5, wood and concrete pieces were observed in the fill material from 1.5 m to 2.9 m depth (SS3 and SS4). Standard Penetration Test (SPT) penetration resistances of 7 to 43 per 0.3 m of penetration were measured within granular fill indicating it to in a loose to dense state. Penetration refusal was encountered within the fill at 1.5 m and 2.1 m in boreholes BH22-3 and BH22-5, suggesting the presence oversize material or an obstruction within the fill.

A cohesive fill layer was encountered from 0.7 m to 1.5 m in BH22-2 and from 2.2 m to 3.0 m in BH22-4. The cohesive fill was described as a brown silty clay/clayey silt, some sand, trace gavel. SPT penetration resistances of 13 and 14 per 0.3 m of penetration were measured within the cohesive portion of fill indicating a stiff consistency.

Laboratory testing carried out on samples of the granular fill measured natural moisture contents of between 3% and 8%, expressed as a percentage of the dry weight of the soil. Natural moisture of a sample of cohesive fill was determined to be 14%.

5.2.3 Champlain Sea Clay

The fill material was underlain by a deposit of sensitive Champlain Sea clay. The base of this deposit extended to depths of approximately 4.5 m to 6.0 m below ground surface, becoming deeper near the northeast corner of the site.

In-situ vane shear tests conducted on the Champlain Sea clay measured undrained shear strength values of about 47 kPa to more than 118 kPa (the maximum value for the equipment used). The sensitivity of the clay is estimated to be 4 to more than 5, and the clay is classified as sensitive in accordance with the errata to the 4th (2006) Edition of the Canadian Foundation Engineering Manual (CFEM). SPT 'N' penetration resistance values ranging from 5 to 29 blows per 0.3 m were measured within these clayey soils. Considering the measured undrained shear strength, encountered shear vane refusals, and recorded SPT N-Values, the clay deposit is generally considered to be stiff to very stiff.

The results of Atterberg limits testing carried out on representative samples of this material are summarized in the following table. The results of this testing are also shown on the Borehole Records included in Appendix C and on Figure D1 in Appendix D, indicate that the Champlain Sea clay samples tested can be classified as Clay of low plasticity (CL).

In addition, the calculated Liquidity Index for the Champlain Sea clay samples were between 0.29 to 0.63 as presented in table below.

Table 5.1: Atterberg Limits Test Results – Champlain Sea Clay (CH)

Borehole	Sample	Depth (m)	Moisture Content, W_n (%)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI	Liquidity Index, LI
BH22-1	SS5	5.6	28	34	18	16	0.63
BH22-3	SS6	4.9	23	28	17	11	0.55
BH22-7	SS4	3.4	24	41	17	24	0.29

Note: PI = (LL-PL) and LI = (W_n-PL)/(LL-PL)

5.2.1 Silty Sand Till

A deposit of silty sand till with trace to some gravel was encountered beneath the Champlain Sea clay deposit at all borehole location expect BH22-7.

The deposit extended to depths of approximately 11.9 m and 5.2 m below ground surface at boreholes BH22-2 and Bh22-6. Other boreholes where terminated within this deposit. Standard Penetration Test (SPT) penetration resistances of 4 to 57 per 0.3 m of penetration were measured within this layer indicating these materials are in a loose to dense state. The deposit was in loose state from 6.8 m to 7.6 m at BH22-3, from 5.3 m to 6.8 in BH22-4, and from 6.0 m to 7.6 m at BH22-5.

Laboratory testing conducted on samples of the till measured natural moisture contents of between 6% and 22%, 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 till. The results of the tests are presented on Figures D2 and D3 in Appendix D and summarized in table below.

Table 5.2: Grain Size Distribution – Silty Sand Till (SM)

Borehole	Sample	Depth (m)	Description	% Gravel	% Sand	% Silt and Clay
BH22-2	SS10	7.7-8.3	SILTY SAND (SM)	11	75	14
BH22-3	SS8	6.9-7.5	SILTY SAND (SM)	7	73	20
BH22-4	SS9	6.1-6.7	SILTY SAND with gravel (SM)	18	46	36
BH22-5	SS9	6.9-7.5	SILTY SAND (SM)	9	75	16

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

5.3 BEDROCK

Bedrock was proven by rock coring at boreholes BH22-2 and BH22-6 at depths of 11.9 m and 5.2 m. Bedrock was inferred by auger refusal at boreholes BH22-1 and BH22-7 at depths of 6.7 m and 4.7 m. Because the shallow refusal to further penetration at BH22-7, a second auger hole was drilled adjacent to the first, resulting in similar refusal depth, suggesting that it most likely corresponds to the bedrock depth.

Split-spoon driving refusal was encountered in boreholes BH22-3 and BH22-5 at depths 8.2 m and 8.5 m, based on refusal driving resistances of 50 blows for 100 mm of penetration and 50 blows for 125 mm of penetration. Split-spoon driving refusal may be due to the presence of cobbles and boulders within the till or due to the presence of bedrock.

The bedrock core obtained from boreholes consisted slightly weathered, very poor to poor quality (very severely fractured with RQD of zero, 7%, and 42%), grey to black shaley limestone. The rock quality designation reflects the degree of fracturing defined as the rock quality designation or RQD, which is an expression of the cumulated length of the rock pieces longer than 100 mm; values of 0%, 7%, and 42% were recorded. 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.

Compressive strength tests conducted on rock core samples collected from a depth of about 13.3 m and 7.5 m in BH22-2 and BH22-6, respectively, showed that the compressive strength of the samples tested were 78.5 MPa and 122.2 MPa. The test result indicates that the bedrock is strong to very strong.

5.4 GROUNDWATER CONDITIONS

Vibrating Wire (VW) piezometers were installed in boreholes BH22-1 and BH22-4 at 6.1 m and 7.6 m depths, respectively, to facilitate the measurement of the groundwater level at the site. The boreholes were filled with a mix of soil cutting and bentonite except for the sections extending from 0.3 m above to 0.3 m below the piezometers, which were filled with sand.

The groundwater levels measured in these VW piezometers and observed during drilling (inferred groundwater level) are summarized in the following table.

Table 5.3: Summary of Groundwater Levels

Borehole No.	Approximate Ground Surface Elevation (m)	Groundwater Depth (m)	Approximate Groundwater Elevation (m)	Date of Measurement
BH22-1	81.4	4.3	77.1	Inferred at the time of drilling (August 15, 2022)
		4.4	**77.0**	**Measured on August 25, 2022**
BH22-3	80.6	6.4	74.2	Inferred at the time of drilling (August 11, 2022)
BH22-4	81.0	4.4	76.6	Inferred at the time of drilling (August 15, 2022)
		4.5	**76.5**	**Measured on August 25, 2022**
BH22-5	80.7	4.9	75.8	Inferred at the time of drilling (August 11, 2022)
BH22-7	81.3	3.7	77.6	Inferred at the time of drilling (August 11, 2022)

Based on the water levels measured at the two VW piezometers described above, the groundwater level at the site was approximately 4.4 m to 4.5 m below the ground surface (or at elevation 76.5 m to 77.0 m). It should be noted that fluctuations in the groundwater levels should be anticipated during and following periods of sustained precipitation and snowmelt as well as throughout the various seasons. As well, lower water levels would be expected during severe drought conditions.

5.5 HYDRAULIC CONDUCTIVITY

For most of the boreholes drilled around the proposed building footprint, the bottom of the clay layer was encountered at depths ranging from 4.5 m and 4.7 m; deeper clay was encountered at BH22-2 and BH22-3, at depths of 5.5 m and 6.0 m, drilled at the east end of the site. Depending on the proposed excavation depth, the construction excavations could be entirely within the clay deposit.

Empirical relationships were used to estimate a range of hydraulic conductivities for the native clay soils encountered at the site. The estimated conductivities are summarized in the following table.

Table 5.4: Estimated Hydraulic Conductivity and Percolation Time for the Clay Soils

Estimation Methods		Hydraulic Conductivity, k (cm/s)	Percolation Time, T (mins/cm)
Typical ranges for Silty Clays in Eastern Canada	Typical Maximum	1×10^{-6}	-
	Typical Minimum	1×10^{-9}	-
Kozeny-Carman method for plastic soils - as presented by Chapuis and Aubertin (2003)	Minimum Value	3.8×10^{-9}	-
	Maximum Value	1.5×10^{-8}	
Based on MMAH Supplementary Standard SB-6 for Clay Soils of high plasticity (CH)	Typical Range	$<10^{-7}$	>50

The hydraulic conductivity of the native till soil was estimated based on an extrapolation of the grain size distribution curves provided on figure D2 in appendix D, following the method recommended by Chapuis (2004) for non-plastic soils. Based on this approach, the following represents the anticipated hydraulic conductivity within the till layer.

Likely upper bound hydraulic conductivity (till) 4×10^{-3} cm/sec (based on D_{10} = 0.06 mm)
Likely lower bound hydraulic conductivity (till) 5×10^{-4} cm/sec (based on D_{10} = 0.04 mm)

The above likely upper bound and lower values consider that estimated values based on grain size distribution are usually half to twice the measured values.

The above indicates that the till is significantly more permeable that the overlying clay layer.

5.6 GEOPHYSICAL TESTING

A Multi-Channel Analysis of Surface Waves (MASW) sounding was performed at the site in order to determine the shear-wave velocity (V_s) and the seismic site classification at the St-Patrick's Home of Ottawa site, Ottawa, Ontario.

The MASW survey was completed in conjunction with the geotechnical investigations. The MASW sounding was carried out on August 02, 2022. The description of the equipment and procedure used to perform the MASW measurements and a summary of the MASW interpretation are provided in a technical memo in Appendix F. The approximate location of the MASW sounding is shown on the MASW location plan provided in Appendix F.

5.7 CHEMICAL ANALYSIS

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 by Paracel Laboratories Inc. on representative samples of soils collected from boreholes.

The analysis results are included in Appendix D and are summarized in the following table.

Table 5.5: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH22-1	SS3 (native)	1.5-2.1	7.33	<5	221	24.7
BH22-3	SS3 (fill)	1.5-2.1	11.67	17	1560	9.53

6.0 DISCUSSION AND RECOMMENDATIONS

This section provides preliminary engineering input related to the geotechnical design aspects of the proposed development based on our interpretation of the available subsurface information described herein and our understanding of the project requirements.

The discussion and recommendations presented in the following sections of this report are intended to provide the designers with preliminary information for planning and design purposes only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities.

The following geotechnical input is based on the information that was available at the time of writing this report. As not all details (e.g., final building configurations and site grades, structural loads etc.) related to the proposed development were available at the time of preparation of this report, all geotechnical comments and input provided herein should be reviewed and revised, as required, as the design progresses and once the final plans become available.

6.1 KEY GEOTECHNICAL ISSUES

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

- The site includes a 0.9 m to 3.0 m topsoil and fill which is not suitable for founding foundation and construction of slab-on-grade. Therefore, as part of the site preparation works, these materials need to be removed from the building footprint. All topsoil and/or organic soils should be removed from the proposed paved areas.
- The site is underlain by 1.5 m to 3.8 m thick, compressible deposit of Champlain Sea clay, typically extending to 4.5 m below the existing ground surface. The clay deposit has a stiff to very stiff 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 shear vane test results suggest that the Champlain Sea clay deposit is sensitive to strength loss when disturbed. This material is not considered suitable for re-use and could require specialized handling procedures (e.g., drying) prior to transport off-site.
- Due to the presence of the clay deposit, it is recommended that the deep foundations be incorporated in the design to support the seven (7) storey building, with basement. Recommendations for the deep foundation options are provided in the following sections.
- The proposed basement floor level is not known at this time; however, it should be anticipated that an underslab drainage system will be required to control groundwater, particularly during wet seasons. The measured water table on August 25, 2022, was 4.4 m to 4.5 m. This would suggest that if the invert of the floor drainage system is kept at least above elevation 77.5 m, the drainage system would only be operating during wet periods and during spring thaw conditions. The potential impacts of locally drawing down the water table would not be an issue since the natural water table would be seasonally lower than the invert level. Assuming the drainage tile invert to

be 0.5 m below the top of floor, would suggest that the top of the basement floor should be above elevation 78.0 m for this design approach.

- The bottom of the clay layer was generally observed at 4.5 m below grade, and generally below elevation 76.9 m. The clay deposit is a low permeability soil and the underlying till layer is a high permeability soil. Depending on the final proposed floor elevation, a limited clay thickness could remain in place below the flow slab, which would have the advantage of reducing the groundwater inflow to be handled by the building drainage system during wet seasons. However, during spring conditions the groundwater pressure from the till could exceed the weight of the remaining clay and the construction pad, which would require that active dewatering using wellpoints could be required to depressurize the till during construction.
- It is understood that the basement floor elevation is proposed at elevation 78.45 m (top of slab). As such, the total pressure at the underside of the clay layer (weight of the floor, drainage layer, and remaining clay) would be about 30 kPa. Should the water level, which was at the base of the clay when measured, raise beyond 3 m during spring thaw, the pressure beneath the clay (within the till) could raise the floor. The clay could be entirely removed to prevent this risk with the understanding that greater drainage system would need to handle larger drainage volumes during spring conditions. It is recommended that a groundwater monitoring program be implemented to help assess variability in the groundwater levels at the site.
- 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.
- 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 Champlain Sea clay deposit is underlain by a silty sand till deposit in a loose to dense state. The liquefaction assessment indicates that a 1.4 m to 1.6 m thick portion of this deposit between 5.2 m to 8.3 m depths is considered susceptible to liquefaction at four borehole locations (BH22-2, BH22-3, BH22-4, and BH22-5).
- Based on the results of the geophysical testing, this Site could be considered as Site Class 'C' based on Table 4.1.8.4.A of the NBCC.

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

6.2 GEOTECHNICAL MODEL

Based on a compilation of all geotechnical data and testing carried out at the site as presented on the Borehole Records and geotechnical laboratory testing (grainsize analyses, Atterberg limits, and moisture contents) carried out at the site. The soil parameters provided in the following table were estimated and were used for geotechnical design in the following section of the report.

Table 6.1: Soil and Bedrock Parameters

Soil/Rock Type	Design Parameters		
	Total Unit Weight, γ (kN/m^3)	Friction Angle, ϕ' (°)	Undrained Shear Strength, S_u (kPa)
Fill	19	31	-
Clay (below elevations 77.7 m to 80.5 m)	17	-	80
Till (below elevations 74.6 m to 76.9 m)	20	32	-
Shaley Limestone Bedrock(1)	26	UCS = 70 MPa	

Notes:

[1] Bedrock was confirmed at elevations 69.2 m and 76.2 m at BH22-2 and BH22-6, respectively, and inferred at elevations 74.6 m and 76.6 m at BH22-1 and BH22-7, respectively.

[2] The groundwater level within the site was approximately 4.4 m to 4.5 m below the ground surface (or at elevation 77.0 m to 76.5 m).

6.3 SEISMIC DESIGN CONSIDERATIONS

6.3.1 Liquefaction Potential

The potential liquefaction of the site soils under seismic loading conditions was assessed using the analysis methodology suggested by Idriss and Boulanger (2008)[4]. The evaluation was completed based on the SPT resistance values (SPT-N values with depth) from the boreholes and based on the following:

- A Site Adjusted PGA of 0.354g.
- An earthquake magnitude, Mw of 6.47.

The formulation by Idriss and Boulanger (2008)[1] compare the earthquake induced cyclic stress ratios (CSR) with the cyclic resistance ratios (CRR) of the soil based on the soil SPT-values. These formulations are discussed in detail in Idriss and Boulanger (2008) with an example illustrated on Page 118 (subsection 3.14). The calculated factor of Safety values based on the recorded SPT-N values within the till from the different boreholes versus depth are presented in Figures F1 to F4 in Appendix F.

The assessment indicates that the Silty Sand Till soils are considered susceptible to liquefaction (factor of safety against liquefaction of less than one) at the following depths and locations:

- From 6.8 m to 8.3 m at BH22-2
- From 6.0 to 7.6 m at BH22-3
- From 5.2 m to 6.8 m at BH22-4
- From 6.0 to 7.6 m at BH22-5

As a result of liquefaction, earthquake-induced settlements in the order of 30 mm to 50 mm should be anticipated. Given that deep foundations are recommended to support the building structure, these settlements would apply only to non-pile supported elements, such as the basement floor slab.

[1] Idriss, I.M. and Boulanger, R.W. (2008). "Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute, Monograph MNO-12, 2008

For clayey soils, it is commonly acknowledged that they cannot reach true-liquefaction condition because of their cohesion and plasticity. For Champlain Sea clay soils, it has been observed through laboratory cyclic simple shear tests carried out on undisturbed soil samples as documented in some geotechnical investigation reports for sites in the Ottawa area (including the publicly available Geotechnical Investigation Report by Golder for the Capital Region Resource Recovery Centre) that the clay may soften considerably, resulting in significant reductions in the undrained shear strength when subjected to a large number of cycles of shear loading in a laboratory environment, however, these stress levels are significantly higher than what is expected for the seismic loading for the Ottawa area.

Section 6.6.3.2(6) from the Canadian Foundation Engineering Manual presents a general method to determine if a clay is susceptible to liquefaction or cyclic mobility. The tested samples from BH22-1 and BH22-3 would be classified as moderately susceptible due to its reduced plasticity index and relatively high moisture content. The tested sample from BH22-7 would be classified as not susceptible to liquefaction.

Table 6.2: Liquefaction Assessment of fine-grained soils (Bray et al., 2004)

Borehole	Sample	Depth (m)	Moisture Content (%)	Liquid Limit, LL	MC/LL	Plasticity Index, PI = (LL-PL)	Liquefaction Assessment
BH22-1	SS5	5.6	28	34	0.82	16	Moderately susceptible to liquefaction or cyclic mobility
BH22-3	SS6	4.9	23	28	0.82	11	
BH22-7	SS4	3.4	24	41	0.58	24	No liquefaction or cyclic mobility, but may undergo significant deformations if cyclic shear stresses> Static undrained shear strength (Su)

The cyclic shear stresses induced in clay deposits considering the site adjusted above-mentioned PGA are estimated to be lower than the measured Su of 47 kPa within the clay deposit and significant deformation of clay deposits is not a concern.

6.3.2 Seismic Class

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 m 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.

Generally, where liquifiable soils are present, such as discussed in the previous section, a Site Class F is applicable to the site. Liquifiable soils were observed in four of the seven boreholes, and the liquifiable thickness was up to 1.6 m. Considering that the thickness and extend of the liquifiable soil is limited, a site-specific response analysis is not necessary.

Geophysical testing was carried out to measure the in-situ shear wave velocity of the subsurface soils and bedrock at the site using the multi-channel analysis of surface waves (MASW) method. The results of the geophysical investigation program can be found in Appendix F of this report.

Based on the results of the geophysical testing, the average shear wave velocity between 0 and 30 m below ground surface ($\bar{V}_{S30}$) was found to be 757 m/s. Based on these measured shear wave velocities, this Site could be considered as Site Class 'C' based on Table 4.1.8.4.A of the NBCC.

A copy of the NBC Seismic Hazard Calculation Data sheet prepared by Natural Resources Canada for this site is provided in Appendix F for reference.

6.4 FROST PENETRATION

The Champlain Sea clay deposit is typically expected to be highly frost susceptible. 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.

6.5 SITE PREPARATION

An approximately 200 to 900 mm thick layer of topsoil containing organic matters was encountered at the surface of the boreholes.

Beneath all building and foundations, all existing surficial topsoil, vegetation, fill material and/or other deleterious materials (e.g., any loose, wet, and/or otherwise disturbed native materials).

Beneath pavement areas, non-clay fill material, free of deleterious material, can be left in place and surface compacted to act as a subgrade for the proposed paved areas. Existing clay fill material should be removed up to 1.5 m from below the top of proposed pavement; clay fill material within 1.5 m from existing surface was observed only within borehole BH22-2.

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

Beneath all buildings and foundations, site grades should then be raised, if needed, 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, excluding basement areas where a drainage system will be required.

Beneath pavement and sidewalks, site grades should be raised using OPSS Select Subgrade Material (SSM) compacted in lifts not exceeding 300 mm to 95% of the material's Standard Proctor Maximum Dry Density (SPMDD)

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 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.

6.5.1 Site Drainage and Subgrade Protection

The clay soils are susceptible to disturbance due to wet weather and/or construction traffic. Therefore, it is critical to control surface water run-off to prevent pounding of water and/or softening of the underlying soils. The prepared subgrade surface for the site should be shaped to prevent pounding of water. Preparation of subgrade should be scheduled such that the protective cover of overlying granular materials or concrete is placed as quickly as possible after subgrade approval by the geotechnical engineer.

The finished grades should provide surface drainage away from all structures. Within 2 m of structures, the exterior should be graded to slope away from the structure at a sufficient gradient. A gradient of 2% should be used wherever possible.

It should be noted that the surface drainage within the site should be collected and directed towards a storm water management system.

6.5.2 Grade Raise Restriction

The site is underlain by a compressible Champlain Sea clay deposit that is approximately 1.5 to 3.8 m thick. Based on the measured in-situ undrained shear strength and plasticity index of tested soil samples, the pre-consolidation pressure of the clay deposit could be as low as 210 kPa at an approximate depth of 4.9 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 or yield stress) 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. Based on the results of the completed settlement analyses, a maximum grade raise restriction of 2 m is, therefore, recommended for the development due to the compressible soils encountered at the site.

6.6 FOUNDATION DESIGN

Considering the presence of the compressible clay deposit at the site and relatively high load expected for the multi-story building, shallow foundation is not an option. 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).

The following deep foundation options could be considered.

Driven piles:	applicable to the middle portion and east end of the building (for axially loaded piles, the minimum driven length is typically considered to be 4 m)
Micro-piles:	applicable throughout
Caissons:	applicable throughout

Driven piles are discussed in the following section, micro-piles and caisson options in later sections.

6.6.1 Piled Foundations

Due to the variable depth to bedrock at the site, particularly at the west end of the proposed building, piled foundations are considered suitable only for a portion of the building area. Driven piles are applicable to the middle portion and east end of the building (for axially loaded piles, the minimum driven length is typically considered to be 4 m)

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 was confirmed in boreholes BH22-2 and BH22-6 at depths of 11.9 m and 5.2 m, respectively (corresponding to elevations of 68.7 m and 76.1 m, respectively). 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.

Because of the presence of boulders within the till and the poor quality of the bedrock, it is recommended that rock-points, such as the Titus rock injector points be included to protect the pile tips.

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 140 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 140 N/mm^2 against steel cross-sectional area, the following ULS geotechnical resistances may be considered.

HP 310x110	1975 kN at ULS
Pipe 324 mm diameter, 11 mm thick wall	1530 kN at ULS

Note:

Section 7.4 provides recommendations to include a sacrificial steel thickness when evaluating the structural capacity of the pile due to a potentially corrosive overburden soil. The sacrificial thickness does not apply to the geotechnical resistance which will be provided by the bedrock.

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

The piles recommended to be spaced at least three diameters apart. Considering that the piles will be on bedrock surface, no group effects is required to be considered in assessment of geotechnical vertical resistance of piles.

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

Downdrag due to potential soil liquefaction

The till which underlies the clay is sporadically considered potentially susceptible to liquefaction during a design seismic event. Based on the conducted liquefaction analyses, settlements associated with liquefaction could reach 30 mm to 50 mm. Therefore, drag loads should be incorporated in the design. For design, the following can be considered for a pile (up to 11 m long).

$$D_L = P_p \times 320 \text{ kN/m}$$

where:

D_L = Drag load in kN

P_p = Perimeter of pile in metres

For longer piles the above D_L value should be proportionally adjusted.

The structural capacity of the pile would need to account for drag load imposed during a seismic event. The geotechnical capacity is not affected by the drag loads. These values are only to be used to validate the structural capacity of the pile.

As discussed elsewhere in this report, a grade-raise restriction of 2 m is required at the site to prevent soil consolidation at the edges of footprint of the proposed building. Therefore, it has been assumed that drag loads due to soil settlements may not be considered in the design.

6.6.2 Micropile Foundation System

The elevation of the bedrock surface encountered at the site is highly variable. Therefore, the consideration could be given to using a micropile foundation system as an alternative to the piled foundation design.

The following conditions have been assumed in assessing the micropile capacities:

- Assumed Rock Unconfined Compressive Strength 70 MPa
- f'c = 35 MPa for concrete
- Pile capacity calculated strictly based on shaft resistance

For Ultimate Limit States (ULS) design, the unfactored bond strength at the grout/rock interface may be taken as 1,500 kPa. Using a resistance factor of 0.4, the factored ULS bond strength is 600 kPa. If higher factored resistance values are required, on-site testing of the micropiles should be carried out. Based on these values, the factored bearing resistances in the following table may be used for micropile design. As the uppermost 1 m of the bedrock mass is often more heavily fractured and less competent, the first metre of rock should not be included as part of the socket length.

Table 6.3: Micropile Axial Capacities

Pile Diameter (m)	Socket length in Competent Bedrock[1] (m)	Factored Bearing Resistance at ULS[2] (kN) Socket Friction
0.150	1.00	285
	2.00	565
	3.00	850
0.175	1.00	330
	2.00	660
	3.00	990
0.200	1.00	375
	2.00	750
	3.00	1125
0.225	1.00	425
	2.00	850
	3.00	1275

Notes:

[1] Micropiles should be socketed into competent bedrock. The socket length in the table above represents the depth socketed into competent bedrock; for design purposes, it should be assumed that uppermost metre of the bedrock is not included in the socket length.
[2] The above geotechnical resistances at ULS include a resistance factor of 0.4 in compression.
[3] Very little axial deformation would occur and therefore, reactions at SLS are not expected to govern.

The following provides additional considerations that should be accounted for in the design and construction of the micropile foundation system:

- The micropiles should be designed and constructed in accordance with standard practices such as those identified in the US Department of Transportation – Federal Highway Administration Publication No. FHWA NHI-05-039 (Micropile Design and Construction Reference Manual).
- Micropiles intended as permanent structural elements should be provided with double corrosion protection.
- In order to limit the potential for differential foundation settlement, all foundations should for the building addition should consist of either shallow foundations bearing on bedrock or micropile foundations socketed into bedrock (i.e. shallow foundations bearing on overburden materials should not be used). In this regard, a micropile supported grade beam is expected be required around the perimeter of the building.
- The resistance values provided above represent the geotechnical capacity of the micropiles; an assessment should be completed to confirm if the geotechnical or structural capacity of the micropiles will govern. Similarly, the structural design of micropiles should take into account other potential failure mechanisms (e.g. buckling).
- Full-time inspection should be carried out by qualified geotechnical personnel during micropile installation. Additionally, sufficient materials testing (e.g. grout compressive strength testing) should be completed to monitor conformance to the pertinent project specifications.
- Stantec's geotechnical group should review the final drawings and specifications for this project prior to tendering/construction to ensure that the guidelines in this report have been adequately interpreted.

6.6.3 Rock Socketed Caissons

Rock socketed caissons constructed using a steel liner, combined with the tremie technique to place concrete may be considered for design. The use of a steel liner and the tremie technique would be required due to the presence of the highly permeable till deposit.

Given the fracture nature of the bedrock at the site, the following should be considered.

- That the top 1.0 m of the rock socket is not to be included in the calculated capacity
- That the rock socket length, within the calculated zone, be at least three (3) times the caisson diameter
- A minimum caisson diameter of 0.9 m be considered
- A factored geotechnical resistance at the concrete-rock shaft interface at ULS of 700 kPa, which includes a resistance factor of 0.4
- A factored geotechnical resistance at the concrete-rock shaft interface at SLS of 600 kPa, corresponding to less than 10 mm of settlement

<u>Construction Inspection</u>

It is anticipated that contractor would use flight augers to construct the caissons. The following should be anticipated.

- That caissons would need be to clean and dewatered to allow for inspection to ensure that all loose materials are removed and that the sidewalks are free of debris
- That concrete should not be placed within a dewatered caisson since waterflow from the fractured bedrock would wash out the cement paste from the concrete
- The caissons would need to be filled with water prior to concreting to allow for use of the tremie method where concrete is pumped underwater, from the bottom of the caisson, while displacing the overlying water
- That full time inspection by a geotechnical engineer's representative would be required while constructing caissons, including placement of concrete by the tremie method

6.7 ROCK ANCHORS

Considering placement of underslab drainage system described in Section 7.1, rock anchors are not expected for this project. However, recommendation related to rock anchors are provided in this section for the sake of completeness.

For rock anchor design, there are several possible failure modes. Failure may occur in the steel tendon, in the bond at either the rock-grout or grout-steel interfaces, or rock mass conical failure. The structural failure modes i.e., failures in the steel tendon and in the grout-steel bond should be reviewed by a structural engineer.

The rock parameters presented in the following table were considered to develop the anchor design recommendations provided herein.

Table 6.4: Parameters for Rock Anchor Design

RQD* (%)	RMR**	GSI***	Hoek and Brown Parameters			Unconfined Compressive Strength of Intact Rock (MPa)	Apex Angle of Failure Cone (degrees)
			m_b	s	a		
7-42	26	30	0.821	0.0004	0.522	70	60

* Rock Quality Designation
** Rock Mass Rating
*** Geological Strength Index

6.7.1 Rock-Grout Failure Mode

When considering the rock-grout failure mode the following should be considered:

- A rock to grout interface bond strength of 800 kPa at ULS, assuming grout with an unconfined compressive strength of 30 MPa. The ULS value provided includes a resistance factor of 0.5.
- The upper 1.0 m of bedrock should not be included as bonded length when calculating the anchor capacity – i.e. it should be considered a no-load zone.
- Minimum bonded anchor length of 3 m and a maximum bonded length of 8 m;
- The unbonded length of anchor should be equal to the height of the rock cone and less half the bonded length. Based on the FHWA guideline (Publication No. FHWA-IF-99-015) titled "Ground Anchors and Anchored Systems" a minimum unbonded length of 3.0 m for bar tendons and 4.5 m for strand tendons is required for rock anchors;
- Grouting of the unbonded length after the anchor has been pre-stressed; and,
- A minimum center-to-center spacing of four times the diameter of the bored hole should be used to prevent or reduce excessive stress concentrations being developed around the anchors.

The above applies for both vertical and inclined rock anchors.

6.7.2 Rock Mass Failure

To minimize the possibility of a rock mass failure, the following approach is recommended:

- For a single anchor, use the calculation method provided on the sheet titled "Rock Anchor: Resistance to Rock Mass Failure" presented in Appendix G.
- The strength developed on the surface of the pull-out cone is best determined from the results of full-scale uplift tests. Where load tests are not possible, the factored tensile strength, σt, of the fractured rock is estimated to be

4 kPa, considering a geotechnical resistance factor of 0.5. The recommended σt value is a conservative estimate based on the rock parameters presented in Table 6.4.

- Where the center-to-center spacing of adjacent rock anchors is less than twice the height of the rock cone, the anchor group resistance to rock mass failure should be reduced to reflect the theoretical rock cone overlap.
- A 60o apex angle should be used to calculate the rock volume within the theoretical cones and the apex should be located in the middle of the bonded length as shown on the sheet titled "Rock Anchor: Resistance to Rock Mass Failure" in Appendix E.
- A submerged unit weight of rock = 16.2 kN/m3 is recommended.

6.7.3 Rock Anchor Testing

Proof testing should be carried out on 100% of production anchors to confirm the design criteria. In accordance with the Canadian Foundation Engineering Manual, proof tests should be taken to the maximum test load of 1.33 times the working (service) load.

6.8 EXCAVATIONS AND RETAINING WALLS

6.8.1 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 to extend to depths of 3 m or less 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 of existing site grades are expected to be within the fill layers or the clay 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.

6.8.2 Dewatering

Based on the water levels measured at the two VW piezometers described above, the average water level within the site was approximately 4.4 m to 4.5 m below the ground surface (or at elevation 76.5 m to 77.0 m). As such, groundwater inflows into small and shallow excavations of less than 3.0 m deep developed within the fill material and clay deposit could be handled by pumping from filtered sumps within the excavation areas.

More significant groundwater inflows should be expected for deeper excavations, especially extending below the prevailing groundwater level at site at the time of excavation. Therefore, more extensive dewatering systems could be required for such conditions requiring Ministry of the Environment and Climate Change (MOECC) permitting.

6.8.3 Earth Pressures on Retaining Walls

Earth pressures will need to be considered in the design of the foundation and basement walls. Any retaining walls should be backfilled with non-frost susceptible granular fill meeting the gradation requirements of OPSS Granular B Type I materials.

The total active (P_A), passive (P_P), and at-rest (P_O) thrusts acting on the walls can be calculated using the following equations:

$$P_A = \tfrac{1}{2} K_a \gamma H^2$$
$$P_P = \tfrac{1}{2} K_p \gamma H^2$$
$$P_O = \tfrac{1}{2} K_o \gamma H^2$$

where;

H = height of the wall
γ = unit weight of the backfill soil

Values for K_a, K_p, K_o and γ for granular backfill material are provided in the table below. These values are based on the assumption that a horizontal back slope is present behind and adjacent to the wall system(s). The earth pressure coefficients need to be adjusted (i.e., increased) where sloping backfill will be present behind the walls.

At-rest earth pressures should be used in the design of walls that are restrained from movement. The thrust acts at a point one third up the height of the wall.

Table 6.5: Non-Seismic Lateral Earth Pressure Parameters (Horizontal Backfill)

Parameter	**OPSS Granular B - Type I**
Bulk Unit Weight, γ (kN/m^3)	22
Effective Friction Angle	32°
Coefficient of Earth Pressure at Rest (K_o)	0.47
Coefficient of Active Earth Pressure (K_a)	0.31
Coefficient of Passive Earth Pressure (K_p)	3.25

The total active and passive thrusts under earthquake conditions can be calculated using the following equations:

$$P_{AE} = \tfrac{1}{2} K_{AE} \gamma H^2$$
$$P_{PE} = \tfrac{1}{2} K_{PE} \gamma H^2$$

where;

K_{AE} = active earth pressure coefficient (combined static and seismic)
K_{PE} = passive earth pressure coefficient (combined static and seismic)
H = height of wall
γ = total unit weight

The recommended seismic earth pressure parameters are provided in table below. The angle of friction between the soil and the wall has been assumed to be 0° to provide a conservative estimate.

Table 6.6: Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I
Bulk Unit Weight, γ (kN/m^3)	22
Effective Friction Angle	32º
K_{AE} (Non-Yielding Wall)	0.37
Height of Application of P_{AE} from base as a ratio of wall height, (H) – Non-Yielding Wall	0.381
Active Earth Pressure (K_{AE}) – Yielding Wall	0.42
Height of Application of P_{AE} from base as a ratio of wall height, (H) - Yielding Wall	0.406
Passive Earth Pressure, (K_{PE})	2.92
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.303

In order to use the coefficients of active and at-rest pressures for the granular materials presented in the tables above, the granular backfill must be provided within a wedge extending out from the base of the wall at 45 degrees (or smaller) to the horizontal. The coefficient of passive earth pressure applicable to wall design should be confirmed during detailed design when additional information on wall configuration and depths/founding elevations are determined.

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 PAVEMENT DESIGN RECOMMENDATIONS

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 6.3 below may be used for design. Where required, site grades below pavement structures are to be raised using imported soils meeting the requirements of OPSS Select Subgrade Material (SSM).

Table 6.7: 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 above pavement structure assumes that the subgrade will consist of either the existing granular fill materials or OPSS SSM material, and that all areas where clay fill subgrade is present, it will be sub excavated to at least 1.5 m below the proposed pavement level, and replaced with compacted OPSS SSM material.
- The pavement subgrade must be proof rolled under the supervision of geotechnical personnel prior to subbase or engineered fill placement. Any soft areas identified during proof rolling may require subexcavation and replacement with additional Granular 'B'. Where required, site grades below pavement structures are to be raised using subgrade fill.
- The finished subgrade surface and the pavement surface should be crowned and graded to direct runoff water away from the development and associated infrastructure.
- Given the low permeability of the native subgrade soils, perimeter drains and pavement subdrains connected to catch basins are recommended to promote drainage of the pavement structure. The subdrains should comprise 100 mm or 150 mm diameter perforated corrugated pipes with filter socks bedded in sand. The top of pipe should be below the lower limit of the granular subbase.
- Asphalt performance grade PG 58-34 should be specified.
- Based on the Ontario Provincial Standard Specification "Material Specification for Superpave and Stone Mastic Asphalt Mixtures" OPSS.MUNI 1151 (April 2018) a Superpave Traffic Category of A is suitable.
- 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.

<u>Compaction Requirements:</u>

- 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.

- 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.
- The compaction of the asphalt layers should be to at least 92.5% Maximum Theoretical Relative Density (MTRD) in accordance with OPSS 310.

7.0 CONSTRUCTION CONSIDERATIONS AND CONSTRAINTS

7.1 UNDERFLOOR DRAINAGE

The proposed development is to include a basement level; therefore, it is recommended that both a perimeter drainage and an under-slab drainage system be included in the design. The following is recommended for the underslab drainage system.

- Concrete floor
- Vapour barrier
- 50 mm of compacted OPSS Granular A, as a working surface
- 250 mm of 19 mm clearstone
- 100 mm perforated drains placed up to 6 m apart
- Filtering, non-woven geotextile between the clearstone and the native soil

The underfloor drainage system should be designed to accommodate the highwater levels associated with spring conditions. Unless seasonal water levels are taken, it should be assumed that the water level could be as high as 1 m below ground surface for brief periods of time.

The required capacity of the groundwater handling system will need to be assessed by a hydrogeologist or a geotechnical engineer once the final basement elevations are confirmed. Significantly different volumes would be anticipated for a shallower basement floor resting on clay, compared to a deeper basement floor resting on the till. The proposed basement floor level is not known at this time; however, the required capacity of the groundwater handling system is estimated to 75,000 L/day based on the following assumptions:

- The basement floor at elevation 78.45 m (top of slab);
- The invert of the floor drainage system at elevation 77.95 m (0.5 m below the basement top of slab);
- The water level could be as high as 1 m below ground surface for brief periods of time (during wet seasons);
- A hydraulic conductivity of 10^{-4} m/s for fill soils at the site.

It should be anticipated the drainage system would only be operating during wet periods and during spring thaw conditions.

7.2 REUSE OF ON-SITE MATERIALS

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

The fill material 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 site are not considered to be suitable for foundation backfill due to its poor free-draining and frost susceptible characteristics. It may, however, be reused as grading fill for landscaped areas if the moisture content permit. These materials could behave like a fluid once excavated/disturbed and could require drying of the soil prior to transport.

7.3 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/pile caps/slabs 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/pile caps/slabs, 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/pile caps 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 100 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.

7.4 CEMENT TYPE AND CORROSION POTENTIAL

Two soil samples (one from native soils and one from fill material) were submitted to Paracel Laboratories Ltd. in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The results of the analysis are summarized in Table 5.5 in a preceding section of this report.

The concentration of soluble sulphates provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater. The soluble sulphate concentrations for the native and fill samples tested is 221 and 1560 µg/g, respectively. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Therefore, based on the soil testing results, Type GU (General Use) Portland Cement should therefore be suitable for use in concrete buried in native soils.

The pH, resistivity, and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The native soil and fill samples pH values were 7.33 and 11.67, respectively. The normal range for soil pH is considered to be between 5.5 to 9.0.

The resistivity of the tested native soil and fill samples is reported 24.7 and 9.5 (ohm-m) suggests a low corrosive environment. A comparison of the resistivity test results to literature references indicate a highly (10-30 ohm-m) and extremely (<10 ohm-m) corrosive environment for the tested native and fill material samples. The additional test results provided in Table 5.6 may be used to aid in the selection of coatings and corrosion protection systems for buried infrastructure incorporating steel components.

Based on the above results and the fact that piles will be driven through native soils, to account for long term corrosion in steel, the following sacrificial thicknesses are recommended in determining the piles steel cross section area:

- For open ended pipe piles, 2 mm on the external and internal steel faces of the pile.
- For close ended pipe filled with concrete, 2 mm on the outside perimeter face of the pile.
- For other H-piles, 2 mm against the steel perimeter.
- Steel pile must have a minimal effective thickness of 10 mm.

8.0 CLOSURE

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of the St. Patrick's Home of Ottawa, 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 Ramin Ghassemi, Ph.D., P.Eng. and reviewed by Raymond Haché, M.Sc., P.Eng., ing.

Respectfully submitted,

STANTEC CONSULTING LTD.

LICENSED PROFESSIONAL ENGINEER
R. GHASSEMI
100536678
2022/11/25
PROVINCE OF ONTARIO

Ramin Ghassemi

Ramin Ghassemi, Ph.D., P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
17713504
2022/11/25
PROVINCE OF ONTARIO

Raymond Haché, M.Sc., P.Eng., ing.
Senior Principal, Geotechnical Engineering

rg \\ca0218-ppfss01\work_group2\01216\active\1216242xx\121624271 - st patricks home\05_report_deliv\deliverables\report\121624271_fnl_rpt_20221121.docx

APPENDIX A

A.1 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.

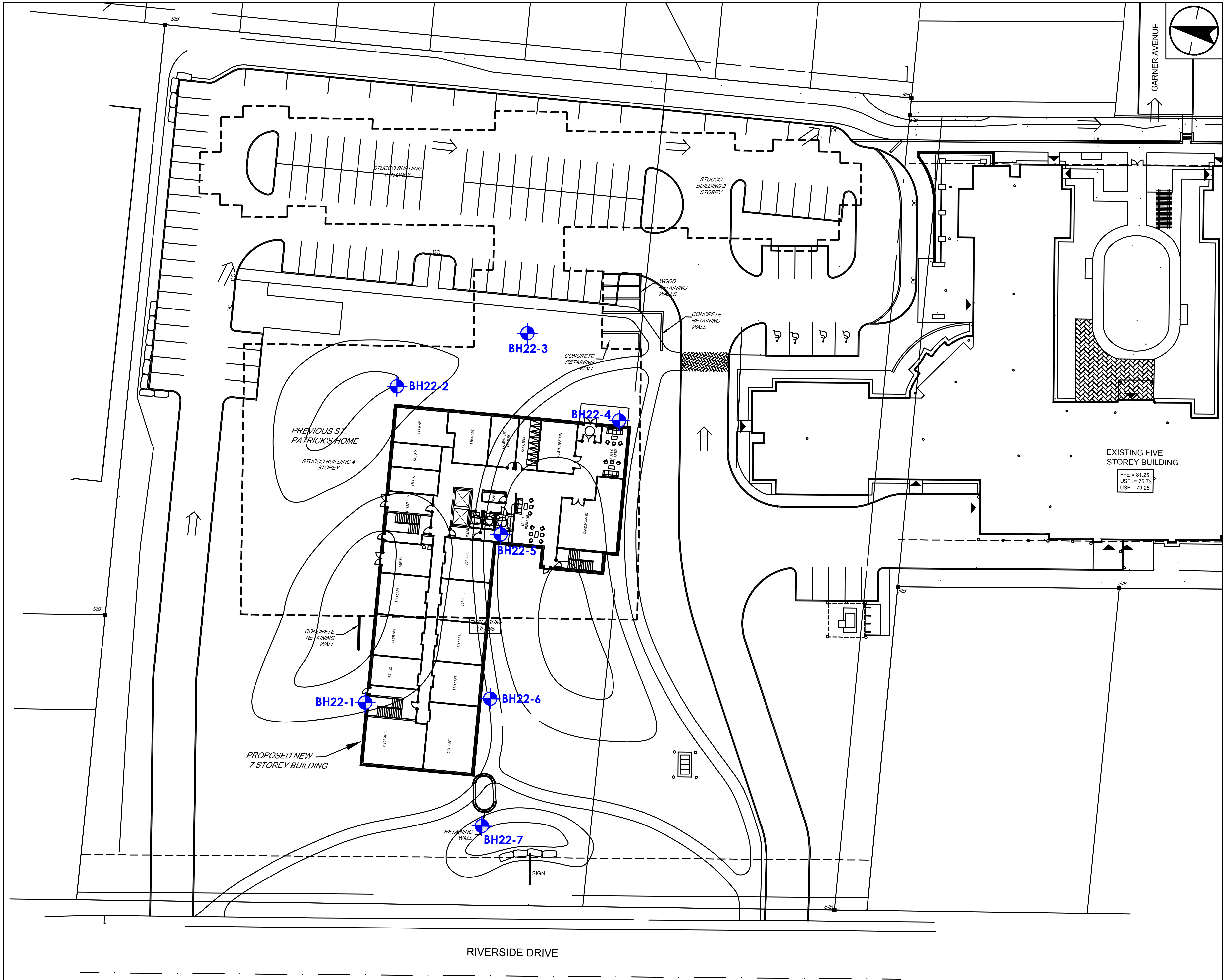
INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications 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

B.1 DRAWING NO. 1 – BOREHOLE LOCATION PLAN

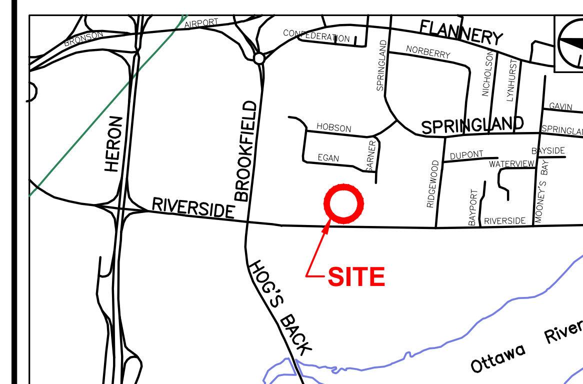


Stantec

300 - 1331 Clyde Avenue
Ottawa, ON, Canada K2C 3G4
www.stantec.com

LEGEND

APPROXIMATE BOREHOLE LOCATION



KEY PLAN 1 : 25 000

NOTES

1. COORDINATE SYSTEM: NAD 1983 UTM ZONE 18N.
2. BASEPLAN PROVIDED BY EDWARD J CUJACI, JULY 19, 2022.

6m 0 6 12m

1 : 600

SEPTEMBER 2022
Project No. 121624271

Client/Project

ST. PATRICK'S HOME OF OTTAWA
GEOTECHNICAL INVESTIGATION
2865 RIVERSIDE DRIVE, OTTAWA, ONTARIO

Drawing No.
1

Title

BOREHOLE LOCATION PLAN

C:\CAD Drawings\Acad2019 Drawings\2022\121624271\121624271_St Patrick's Home_BH Locations.dwg
Printed: Sep 30, 2022 By: G. Briones

APPENDIX C

C.1 SYMBOLS & TERMS USED ON THE BOREHOLE RECORDS

C.2 BOREHOLE RECORDS

C.3 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.

Boulders Cobbles Gravel | Sand | Silt | Clay | Organics | Asphalt | Concrete | Fill | Igneous Bedrock | Metamorphic Bedrock | Sedimentary Bedrock

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

Stantec

BOREHOLE RECORD

N: 5 024 344 E: 445 986

BH22-1

CLIENT St. Patrick's Home of Ottawa

BOREHOLE No. BH22-1

LOCATION 2865 Riverside Drive, Ottawa, Ontario

PROJECT No. 121624271

DATES: BORING August 15, 2022 WATER LEVEL August 25, 2022

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	81.36	TOPSOIL - 700 mm	SS	1	360	8
0.7	80.7	FILL: Compact, Brown, Moist SAND (SP), trace gravel	SS	2	300	11
1.3	80.1	Stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL), trace gravel	SS	3	300	14
			SS	4	560	11
			SS	5	610	9
		*Undrained Strength>118 kPa	SS	6	540	9
4.5	76.9	Dense to very dense, Brown, Moist SILTY SAND TILL (SM), trace gravel	SS	7	430	50
			SS	8	410	36

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Groundwater Level in Open Borehole

Groundwater Level Measured in Standpipe

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 024 344 E: 445 986

BH22-1

CLIENT: St. Patrick's Home of Ottawa

BOREHOLE No.: BH22-1

LOCATION: 2865 Riverside Drive, Ottawa, Ontario

PROJECT No.: 121624271

DATES: BORING: August 15, 2022 WATER LEVEL: August 25, 2022

DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
6	74.6	-Wet, grey below 6.1 m depth			SS	9	360	51
		End of Borehole - Augur Refusal						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

End of Borehole - Augur Refusal

Borehole was open and dry upon completion of drilling.

A vibrating wire piezometer was installed at 6.1 m depth. Groundwater was measured to be at 4.3 m at the time of installation and at 4.4 m on August 25, 2022.

*Implied from field vane test refusal at this depth.

Depth scale: 6, 7, 8, 9, 10, 11, 12

∇ Groundwater Level in Open Borehole

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

STN13-STAN-GEO 121624271 - ST.PATRICK'S_HOME_20220911.GPJ SMART.GDT 10/12/22

Stantec

BOREHOLE RECORD

N: 5 024 349 E: 446 036

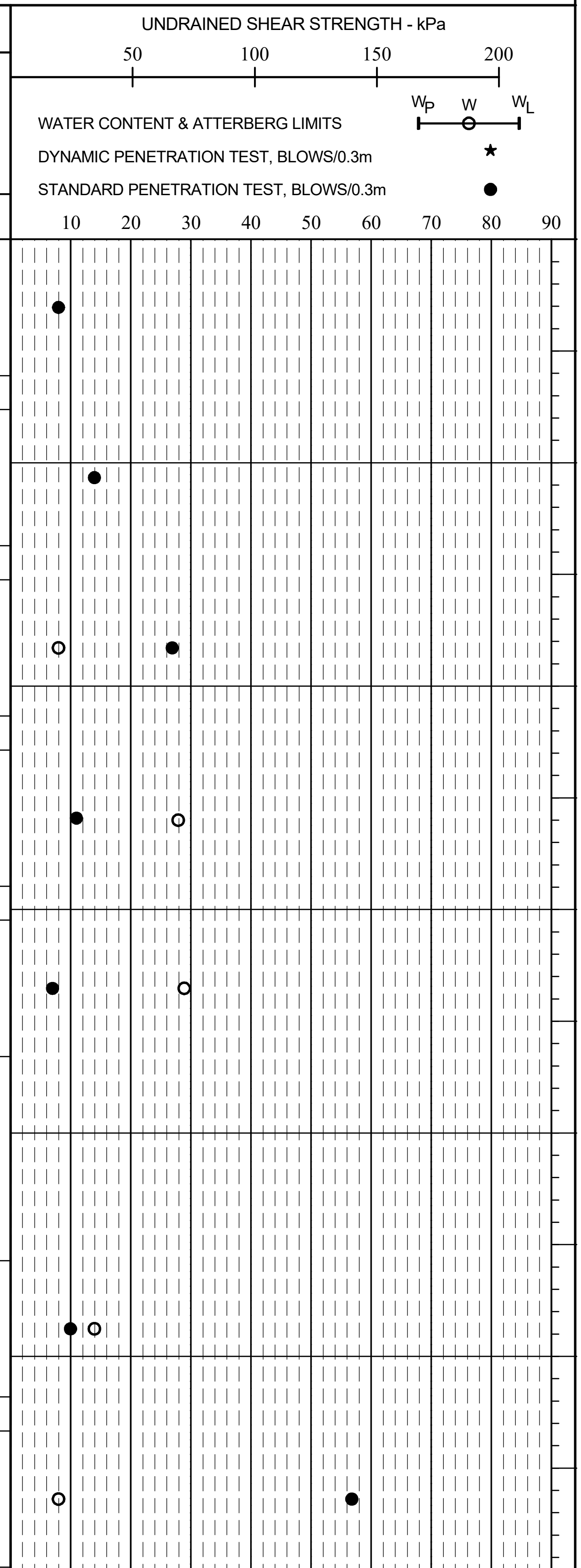
BH22-2

CLIENT: St. Patrick's Home of Ottawa
BOREHOLE No.: BH22-2

LOCATION: 2865 Riverside Drive, Ottawa, Ontario
PROJECT No.: 121624271

DATES: BORING: August 12, 2022 WATER LEVEL: n/a
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	81.02	TOPSOIL - 700 mm	SS	1	230	8
0.7	80.3	FILL: Stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL), trace gravel	SS	2	280	14
1.5	79.5	FILL: Compact, Brown, Moist SAND (SP), trace gravel	SS	3	530	27
2.5	78.5	Stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL), trace gravel	SS	4	530	11
			SS	5	610	7
		*Undrained Strength>118 kPa				
		-Grey below 4.6 m depth	SS	6	560	10
5.5	75.5	Compact to very dense, Brown, Moist SILTY SAND TILL (SM), trace	SS	7	380	57



Groundwater Level in Open Borehole
Groundwater Level Measured in Standpipe

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

Stantec

BOREHOLE RECORD

N: 5 024 349 E: 446 036

BH22-2

CLIENT St. Patrick's Home of Ottawa
BOREHOLE No. BH22-2
LOCATION 2865 Riverside Drive, Ottawa, Ontario
PROJECT No. 121624271
DATES: BORING August 12, 2022 WATER LEVEL n/a
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
6		gravel -Wet below 6.1 m depth	SS	8	480	22
7			SS	9	200	13
8			SS	10	610	13
			SS	11	610	26
9			SS	12	610	47
	71.3	Advanced through the deposit by coring upon refusal at 9.7 m depth				
10						
11						
	69.2	SHALEY LIMESTONE				
12						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Groundwater Level in Open Borehole
Groundwater Level Measured in Standpipe

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

Stantec

BOREHOLE RECORD

N: 5 024 349 E: 446 036

BH22-2

CLIENT: St. Patrick's Home of Ottawa

BOREHOLE No.: BH22-2

LOCATION: 2865 Riverside Drive, Ottawa, Ontario

PROJECT No.: 121624271

DATES: BORING: August 12, 2022 WATER LEVEL: n/a

DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
12		BEDROCK Slightly weathered, very poor quality, strong, grey/black Recovered cores were entirely fractured (Refer to Bedrock Core Log for Details) *UCS = 78.5 MPa			NQ	13	100%	0
13					NQ	14	100%	7%
	67.3	End of Borehole Borehole was open upon completion of drilling; Groundwater could not be measured due to use of water for coring. *Implied from field vane test refusal at this depth. *UCS = Uniaxial Compressive Strength						
14								
15								
16								
17								
18								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Groundwater Level in Open Borehole

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 024 330 E: 446 049

BH22-3

CLIENT St. Patrick's Home of Ottawa — BOREHOLE No. BH22-3

LOCATION 2865 Riverside Drive, Ottawa, Ontario — PROJECT No. 121624271

DATES: BORING August 11, 2022 WATER LEVEL n/a — DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	80.60	TOPSOIL - 1000 mm	SS	1	360	12
1	79.6	FILL: Brown, Moist SAND & GRAVEL (SP/GP)	SS	2	300	5
			SS	3	330	83/280
2.2	78.4	Stiff to very stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL), trace sand, trace gravel	SS	4	510	17
			SS	5	610	5
		-Wet, grey below 4.9 m depth	SS	6	360	8
		*Undrained Strength>118 kPa				
6	74.6					

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Groundwater Level in Open Borehole

Groundwater Level Measured in Standpipe

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

STN13-STAN-GEO 121624271 - ST.PATRICK'S_HOME_20220911.GPJ SMART.GDT 10/12/22

Stantec

BOREHOLE RECORD

N: 5 024 330 E: 446 049

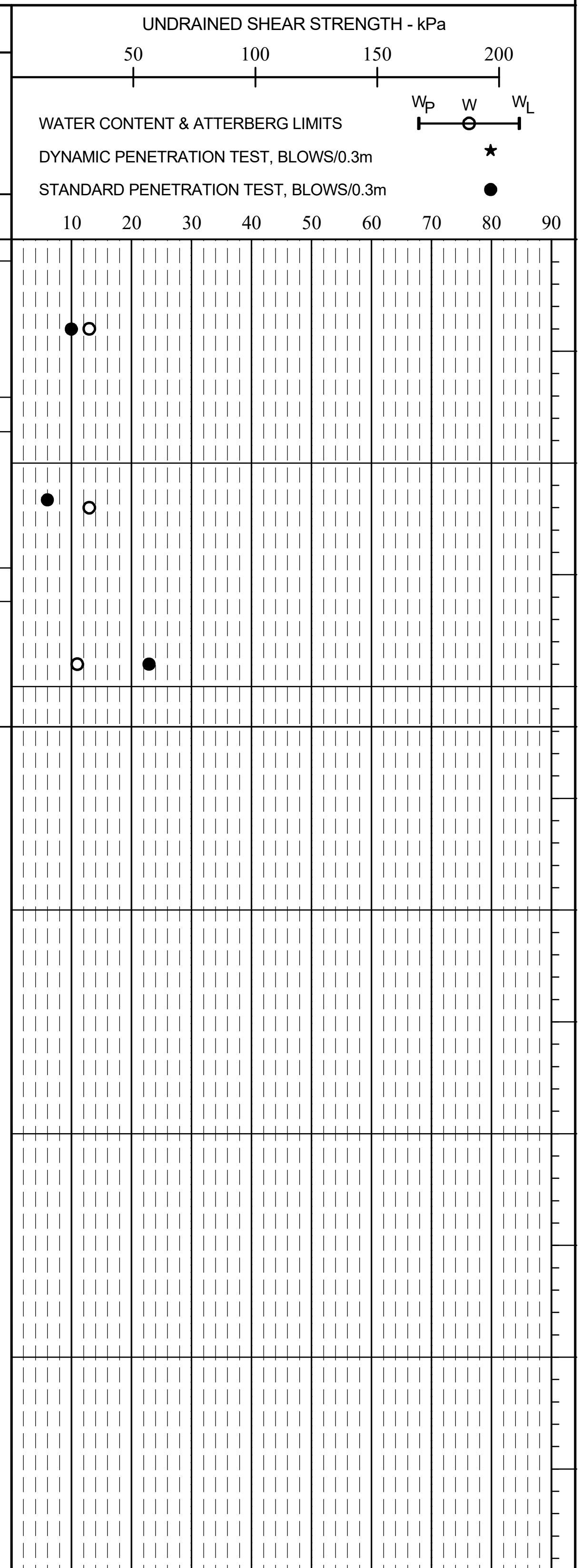
BH22-3

CLIENT St. Patrick's Home of Ottawa
BOREHOLE No. BH22-3

LOCATION 2865 Riverside Drive, Ottawa, Ontario
PROJECT No. 121624271

DATES: BORING August 11, 2022 WATER LEVEL n/a
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
6		Loose to compact, Brown, Wet SILTY SAND TILL (SM), trace gravel		∇	SS	7	250	10
7					SS	8	330	6
8					SS	9	560	23
	72.4	SPT refusal at 8.1 m depth (50 blows for 100 mm of penetration)						
		End of Borehole						
9		Borehole caved in to 7.3 m depth and groundwater was measured at 6.4 m on completion of drilling.						
		*Implied from field vane test refusal at this depth.						
10								
11								
12								



∇ Groundwater Level in Open Borehole
▼ Groundwater Level Measured in Standpipe

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

Stantec

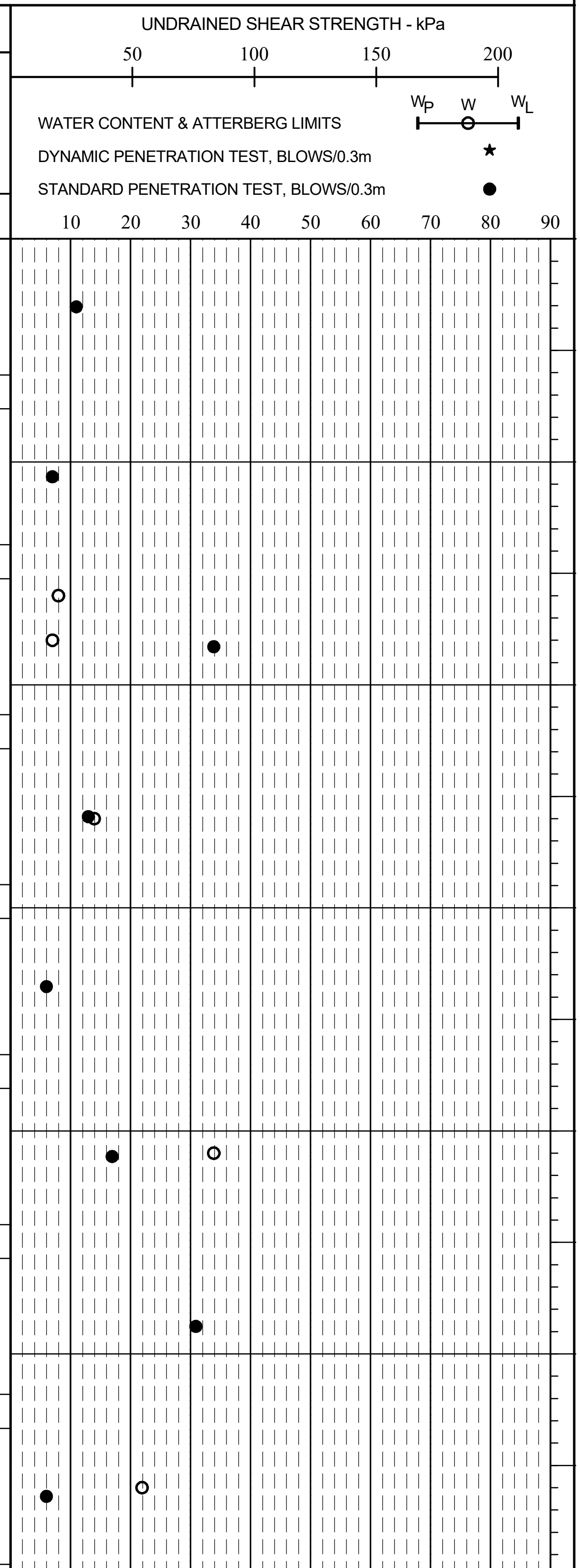
BOREHOLE RECORD

N: 5 024 313 E: 446 038

BH22-4

CLIENT	St. Patrick's Home of Ottawa	BOREHOLE No.	BH22-4
LOCATION	2865 Riverside Drive, Ottawa, Ontario	PROJECT No.	121624271
DATES: BORING	August 15, 2022 — WATER LEVEL August 25, 2022	DATUM	Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	81.04					
	80.8	TOPSOIL - 200 mm				
		FILL: Loose to compact, Brown, Moist SAND (SP), some gravel	SS	1	480	11
1			SS	2	330	7
			SS	3	360	34
2	78.8					
		FILL: Stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL), some sand	SS	4	360	13
3	78.0					
		Stiff to very stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL), trace gravel	SS	5	410	6
4		-Wet below 3.8 m depth	SS	6	130	17
	76.5					
		Loose to dense, Grey, Moist SILTY SAND/SANDY SILT TILL (SM/ML), trace clay	SS	7	510	31
5			SS	8	410	6
6	75.0					



∇ Groundwater Level in Open Borehole
▼ Groundwater Level Measured in Standpipe

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

Stantec

BOREHOLE RECORD

N: 5 024 313 E: 446 038

BH22-4

CLIENT St. Patrick's Home of Ottawa
LOCATION 2865 Riverside Drive, Ottawa, Ontario
DATES: BORING August 15, 2022 WATER LEVEL August 25, 2022

BOREHOLE No. BH22-4
PROJECT No. 121624271
DATUM Geodetic

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●
10 20 30 40 50 60 70 80 90

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
6		Loose to compact, Grey, Moist SILTY SAND with gravel TILL (SM)			SS	9	360	5
7					SS	10	300	15
8					SS	11	610	22
					SS	12	610	25
9	72.1	End of Borehole Borehole was open and dry upon completion of drilling. A vibrating wire piezometer was installed at 7.6 m depth. Groundwater was measured to be at 4.4 m at the time of installation and at 4.5 m on August 25, 2022.						
10								
11								
12								

∇ Groundwater Level in Open Borehole
▼ Groundwater Level Measured in Standpipe

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

Stantec

BOREHOLE RECORD

N: 5 024 328 E: 446 016

BH22-5

CLIENT St. Patrick's Home of Ottawa

BOREHOLE No. BH22-5

LOCATION 2865 Riverside Drive, Ottawa, Ontario

PROJECT No. 121624271

DATES: BORING August 11, 2022 WATER LEVEL n/a

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
	80.75							
0		TOPSOIL - 700 mm			SS	1	410	14
	80.0	FILL: Dense, Brown, Moist SAND (SP), some gravel						
1					SS	2	230	43
		Wood and concrete pieces were observed in the sample taken from 1.5 m to 2.1 m depth (SS3)			SS	3	150	50/250
2		Wood and concrete pieces were observed in the sample taken from 2.3 to 2.9 m depth (SS4); the sample was wet possibly due perched water condition			SS	4	250	36
3	77.7	Stiff to very stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL), trace gravel			SS	5	430	7
4		*Undrained Strength>118 kPa						
	76.2	Compact, Grey, Wet SILTY SAND/SANDY SILT TILL (SM/ML), trace gravel		∇	SS	6	430	12
5	75.4	Loose to compact, Grey, Wet SILTY SAND TILL (SM), trace gravel			SS	7	50	18
6								

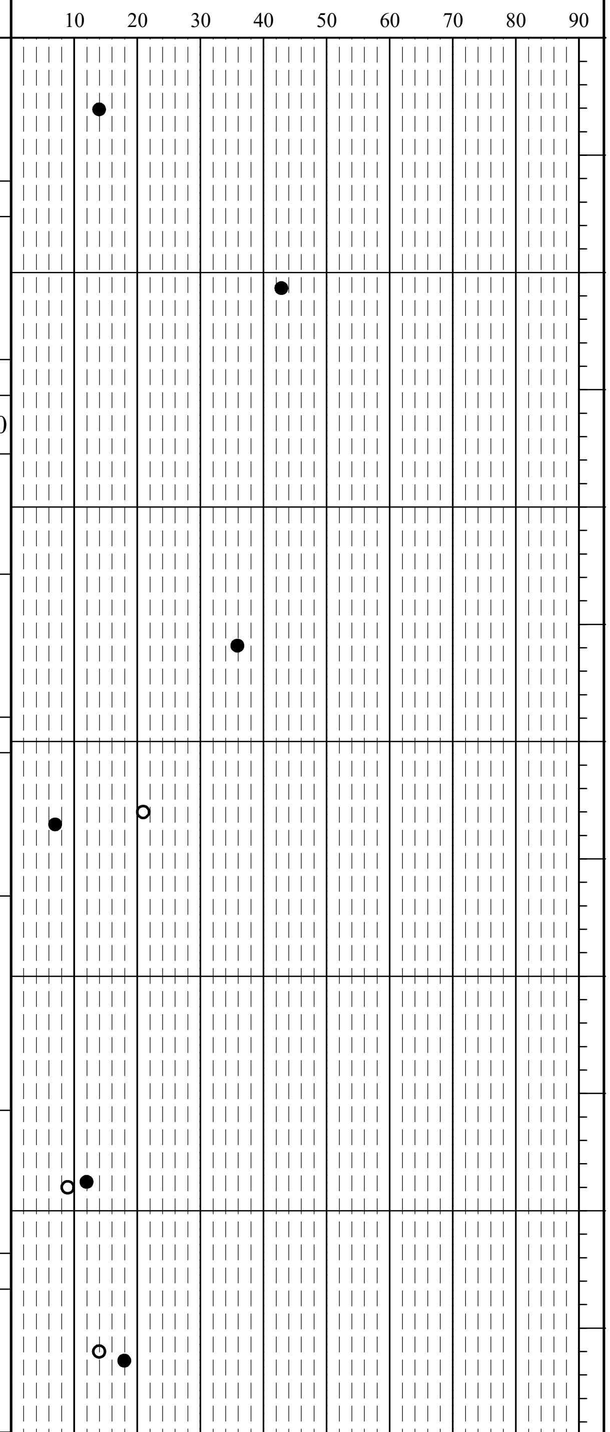
UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●



∇ Groundwater Level in Open Borehole

▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 024 328 E: 446 016

BH22-5

CLIENT St. Patrick's Home of Ottawa
BOREHOLE No. BH22-5
LOCATION 2865 Riverside Drive, Ottawa, Ontario
PROJECT No. 121624271
DATES: BORING August 11, 2022 WATER LEVEL n/a
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
6						
			SS	8	150	4
7			SS	9	610	8
8			SS	10	360	23
	72.2	SPT refusal at 8.4 m depth (50 blows for 125 mm of penetration)	SS	11	130	50/130
		End of Borehole Borehole caved in to 6.7 m depth and groundwater was measured at 4.9 m on completion of drilling. *Implied from field vane test refusal at this depth.				
9						
10						
11						
12						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Groundwater Level in Open Borehole

Groundwater Level Measured in Standpipe

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 024 325 E: 445 990

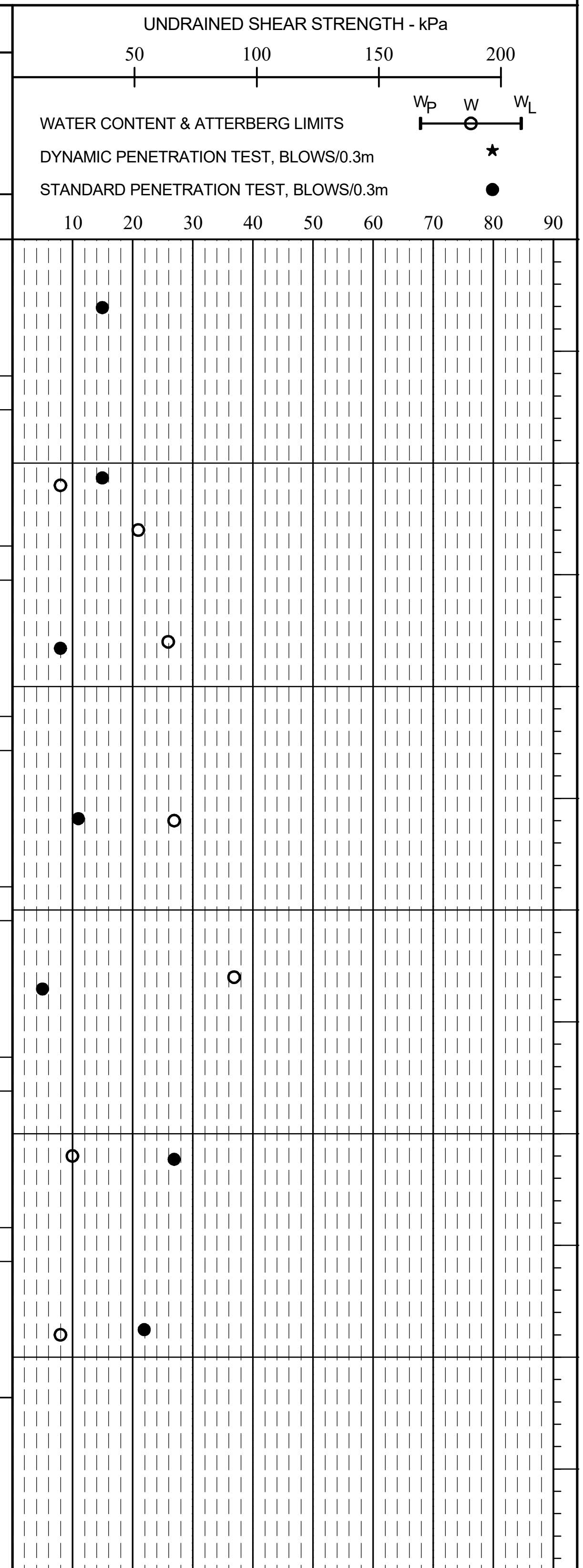
BH22-6

CLIENT St. Patrick's Home of Ottawa
BOREHOLE No. BH22-6

LOCATION 2865 Riverside Drive, Ottawa, Ontario
PROJECT No. 121624271

DATES: BORING August 12, 2022 WATER LEVEL n/a
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	81.41	TOPSOIL - 300 mm	SS	1	430	15
	81.1	FILL: Compact, Brown, Moist SAND & GRAVEL (SP/GP)				
1	80.5	Stiff to very stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL) 100 mm thick sand seam at 1.1 m depth	SS	2	360	15
2			SS	3	510	8
			SS	4	610	11
3			SS	5	610	5
4		*Undrained Strength>118 kPa	SS	6	460	27
5	76.9	Compact, Grey, Moist SILTY SAND/SANDY SILT TILL (SP/SM), trace gravel	SS	7	530	22
6	76.2	SHALEY LIMESTONE BEDROCK Moderately weathered, very poor quality, medium strong, grey Recovered cores were entirely fractured	NQ	8	61%	0



∇ Groundwater Level in Open Borehole
▼ Groundwater Level Measured in Standpipe

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

Stantec

BOREHOLE RECORD

N: 5 024 325 E: 445 990

BH22-6

CLIENT St. Patrick's Home of Ottawa

LOCATION 2865 Riverside Drive, Ottawa, Ontario

DATES: BORING August 12, 2022 WATER LEVEL n/a

BOREHOLE No. BH22-6

PROJECT No. 121624271

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD	UNDRAINED SHEAR STRENGTH - kPa (50, 100, 150, 200) / WATER CONTENT & ATTERBERG LIMITS / DYNAMIC PENETRATION TEST, BLOWS/0.3m / STANDARD PENETRATION TEST, BLOWS/0.3m (10–90)
6	75.3	SHALEY LIMESTONE BEDROCK Slightly weathered, poor quality, very strong, grey/black Recovered cores were fractured to 6.9 m depth (Refer to Bedrock Core Log for Details) *UCS = 122.2 MPa			NQ	9	96%	42%	
7	73.8	End of Borehole - Augur Refusal Borehole was open upon completion of drilling; Groundwater could not be measured due to use of water for coring. *Implied from field vane test refusal at this depth. *UCS = Uniaxial Compressive Strength							
8									
9									
10									
11									
12									

Groundwater Level in Open Borehole
Groundwater Level Measured in Standpipe

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

Stantec

BOREHOLE RECORD

N: 5 024 322 E: 445 970

BH22-7

CLIENT: St. Patrick's Home of Ottawa

LOCATION: 2865 Riverside Drive, Ottawa, Ontario

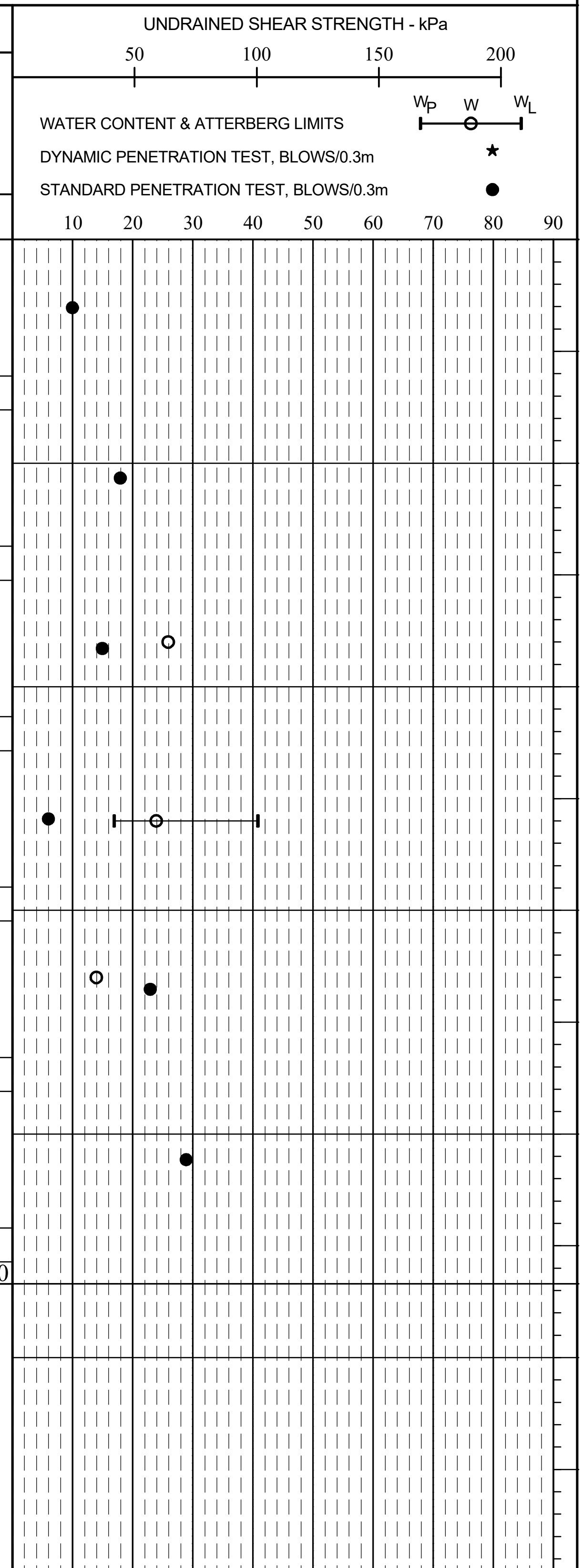
DATES: BORING August 11, 2022 WATER LEVEL n/a

BOREHOLE No. BH22-7

PROJECT No. 121624271

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	**81.27**	TOPSOIL - 1000 mm			SS	1	330	10
1	80.3	FILL: Compact, Brown, Moist SAND (SP), some gravel			SS	2	380	18
	79.8	Stiff to very stiff, Brown, Moist SILTY CLAY/CLAYEY SILT (CL), trace gravel			SS	3	510	15
2					SS	4	610	6
3		*Undrained Strength>118 kPa			SS	5	530	23
4				∇	SS	6	30	29
	76.6	-Wet, grey below 4.5 m depth			SS	7	30	50/100
5		End of Borehole - Auger refusal Borehole caved in to 4.0 m depth and groundwater was measured at 3.7 m on completion of drilling. Upon refusal, another borehole was advanced at about 1.5 m offset to the original borehole						
6								



∇ Groundwater Level in Open Borehole

▼ Groundwater Level Measured in Standpipe

▫ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 024 322 E: 445 970

BH22-7

CLIENT St. Patrick's Home of Ottawa

LOCATION 2865 Riverside Drive, Ottawa, Ontario

DATES: BORING August 11, 2022 WATER LEVEL n/a

BOREHOLE No. BH22-7

PROJECT No. 121624271

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD	UNDRAINED SHEAR STRENGTH - kPa (50, 100, 150, 200)
6		location; auger refusal was encountered at 4.6 m depth. *Implied from field vane test refusal at this depth.							
7									
8									
9									
10									
11									
12									

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Groundwater Level in Open Borehole

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Field Bedrock Core Log

Stantec

Client:	St. Patrick's Home of Ottawa	**Project No.:**	121624271
Project:	Geotechnical Investigation for the Proposed Building	**Date:**	12-Aug-22
Contractor:	Downing	**Borehole No.:**	BH22-2
		Logger:	Ben Heyl

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES: NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING	OCCASIONAL FEATURES	DRILLING OBSERVATIONS
11.86	13	100	0	12.14	SHALEY LIMESTONE BEDROCK Slightly weathered, very poor quality, strong, grey/black	R4	W2	2	BD	F	C	RU	G	S/O	-	Recovered cores were entirely fractured
									JN	V	-	RU	G	S		
12.14	14	100	7	13.72	SHALEY LIMESTONE BEDROCK Slightly weathered, very poor quality, strong, grey/black	R$	W2	2	BD	F	VC	RU	G	Si	UCS = 78.5 MPa	Recovered cores were entirely fractured
									JN	V	-	RU	G	O		

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

JOINT APERTURE

C = Closed = < 0.5 mm
G = Gapped = 0.5 to 10 mm
O = Open = > 10 mm

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

Field Bedrock Core Log

Stantec

Client:	St. Patrick's Home of Ottawa	**Project No.:**	121624271
Project:	Geotechnical Investigation for Proposed Building	**Date:**	12-Aug-22
Contractor:	Downing	**Borehole No.:**	BH22-6
		Logger:	Ben Heyl

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES: NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING	OCCASIONAL FEATURES	DRILLING OBSERVATIONS
5.18	8	610	0	6.15	SHALEY LIMESTONE BEDROCK Moderately weathered, very poor quality, very strong, grey	R5	W3	-							No solid pieces. Discolored/ oxidized.	Recovered cores were entirely fractured
6.15	9	96	42	7.65	SHALEY LIMESTONE BEDROCK Slightly weathered, poor quality, very strong, grey/black	R5	W2	1	BD	F	C	RU	G	SA	UCS = 122.2 MPa	Recovered cores were fractured to 6.9 m depth

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}

JOINT APERTURE

C = Closed = < 0.5 mm
G = Gapped = 0.5 to 10 mm
O = Open = > 10 mm

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

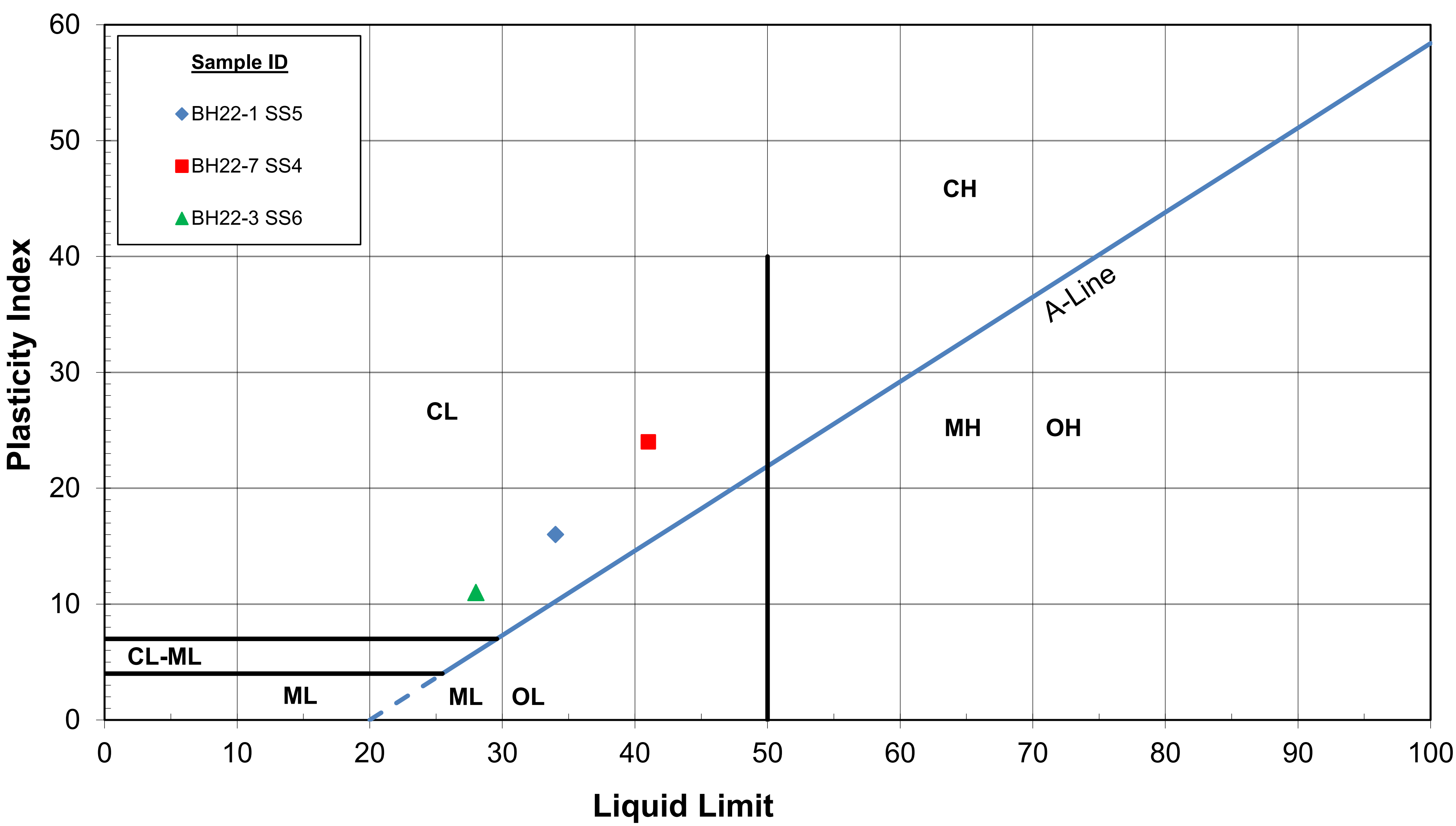
V:\01216\active\1216242XX\121624271 - St Patricks Home\05_report_deliv\deliverables\report\app-c3_bedrock_core_log.xlsx

Photo No. 1: BH22-2 / NQ13 and NQ14 / 11.86 m to 13.72 m depths

Photo No. 2: BH22-6 / NQ8 and NQ9 / 5.18 m to 7.65 m depths

APPENDIX D

D.1 GEOTECHNICAL LABORATORY TEST RESULTS



Stantec

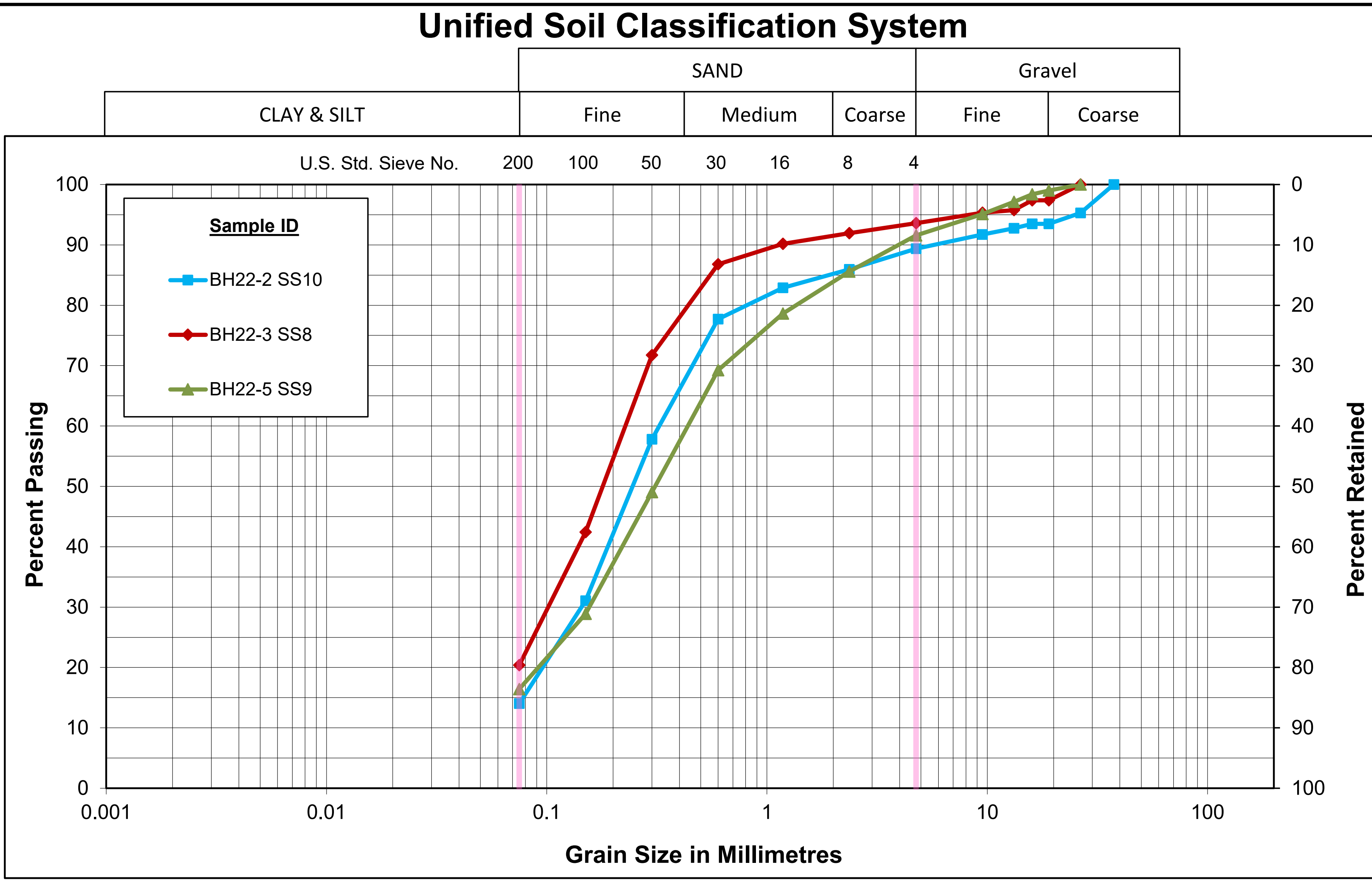
St. Patrick's Home of Ottawa

Champlain Sea Clay (CL)

PLASTICITY CHART

Figure No. D1

Project No. 121624271



Stantec

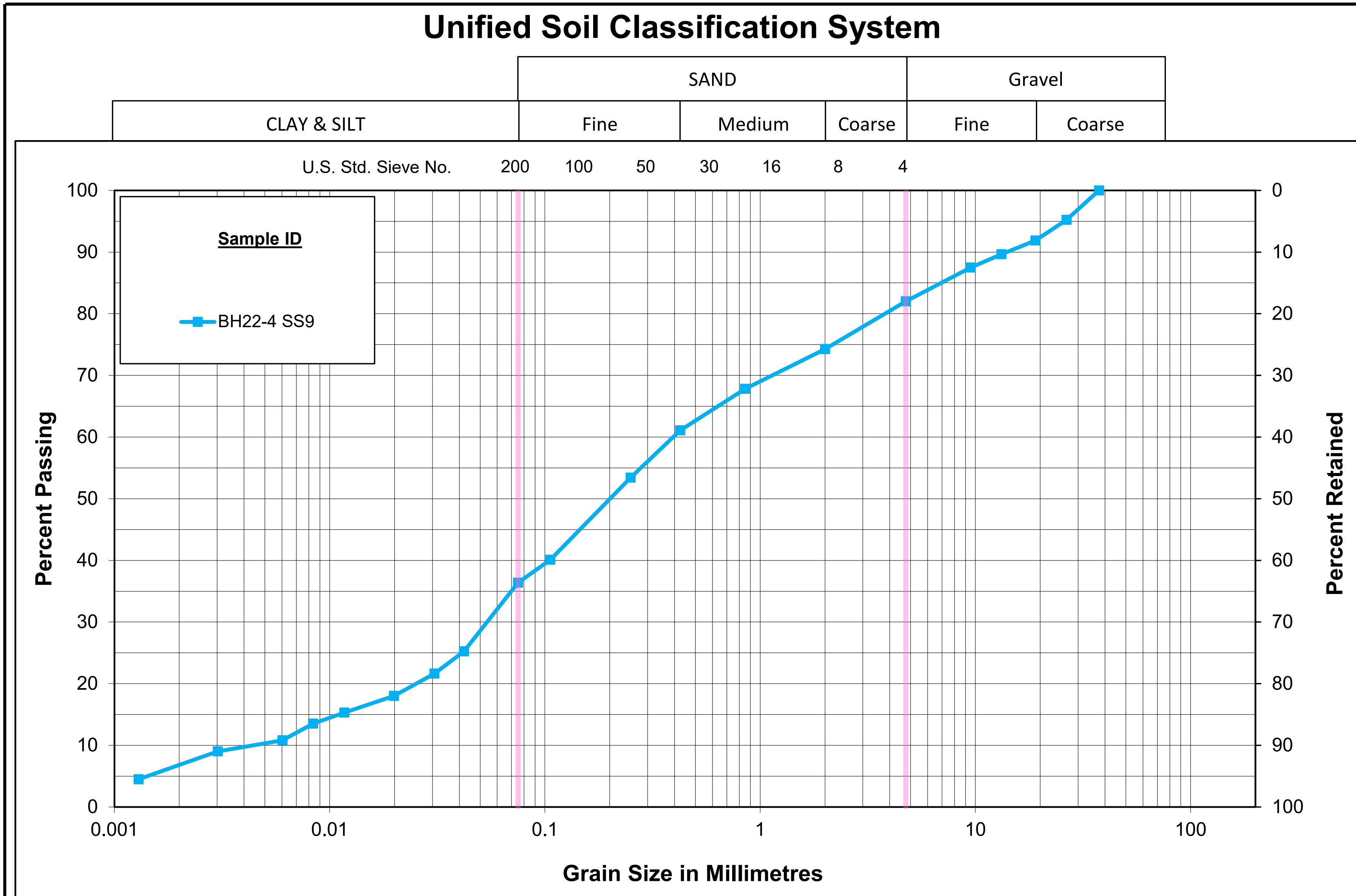
GRAIN SIZE DISTRIBUTION

Silty Sand Till (SM)

St. Patrick's Home of Ottawa

Figure No. D2

Project No. 121624271



Stantec

GRAIN SIZE DISTRIBUTION

SILTY SAND with gravel TILL (SM)

St. Patrick's Home of Ottawa

Figure No. D3

Project No. 121624271

Stantec

Compressive Strength & Elastic Moduli of Intact Rock Core Speciments under Varying States of Stress and Temperatures
Method C
ASTM D7012 & D4543

Client:	St. Patrick's Home of Ottawa	Project No.:	121624271
Project:	GeoInv_St.Patrick'sHome		
Material Type:	Rock Core; Diameter ≥ 47.0 mm	Date Received:	August 17, 2022
Sampled By:	Stantec- BH	Tested By:	Moe Komaiha
Date Sampled:	August 12, 2022	Date Tested:	August 30, 2022

Sample Information				
Borehole Location	**BH22-2**	**BH22-6**		
Sample Number	RC2	RC2		
Sample Depth	43'9" TO 44'2"	24'6" TO 25'1"		
Compressive Strength Test Data				
Physical Description	As per Geotechnical Report	As per Geotechnical Report		
Average Sample Diameter (mm) (≥47.0)	48	47		
Average Sample Length (mm)	116	115		
Density (kg/m^3)	2684.62	2700.75		
Unit Weight (kN/m^3)	26.34	26.49		
L/D Ratio (2.0-2.5)	2.45	2.42		
Failure Load (lbs)	31430	48560		
Compressive Strength (MPa)	**78.5**	**122.2**		
Straightness by Procedure S1 (≤0.02inch)	<0.02	<0.02		
Flatness by Procedure FP2 (≤0.001inch)	<0.001	<0.001		
Parallelism by Procedure FP2 (≤0.25°)	0.038	0.069		
Perpendicularity by Procedure P2 (≤0.0043)	<0.0043	<0.0043		
Moisture Condition	As-Received	As-Received		
Description of Break D7012/11.1.13	Well formed cones at both ends	Vertical Cracking throughout, no well formed cones		
Note	-	-		

Remarks:

Reviewed by: ____________________ Date: September 15, 2022

APPENDIX E

E.1 LABORATORY CHEMICAL ANALYSIS RESULTS

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

Certificate of Analysis

Stantec Consulting Ltd. (Ottawa)
1331 Clyde Avenue Suite 400
Ottawa, ON K2C 3G4
Attn: Brian Prevost

Report Date: 6-Sep-2022
Order Date: 30-Aug-2022

Client PO: St. Patrick Home
Project: 121624271
Custody:

Order #: 2236112

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

Paracel ID	Client ID
2236112-01	BH22-1, SS3, 5'-7'
2236112-02	BH22-3, SS3, 5'-7'

Approved By:

Milan Ralitsch, PhD

Senior Technical Manager

PARACEL
LABORATORIES LTD.

Order #: 2236112

Certificate of Analysis

Report Date: 06-Sep-2022

Client: **Stantec Consulting Ltd. (Ottawa)**

Order Date: 30-Aug-2022

Client PO: St. Patrick Home

Project Description: 121624271

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	1-Sep-22	1-Sep-22
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	31-Aug-22	1-Sep-22
Resistivity	EPA 120.1 - probe, water extraction	2-Sep-22	2-Sep-22
Solids, %	Gravimetric, calculation	31-Aug-22	1-Sep-22

Summary of Criteria Exceedances

(If this page is blank then there are no exceedances)

Only those criteria that a sample exceeds will be highlighted in red

Regulatory Comparison:

Paracel Laboratories has provided regulatory guidelines on this report for informational purposes only and makes no representations or warranties that the data is accurate or reflects the current regulatory values. The user is advised to consult with the appropriate official regulations to evaluate compliance. Sample results that are highlighted have exceeded the selected regulatory limit. Calculated uncertainty estimations have not been applied for determining regulatory exceedances.

Sample	Analyte	MDL / Units	Result	-	-

Certificate of Analysis

Report Date: 06-Sep-2022

Client: **Stantec Consulting Ltd. (Ottawa)**

Order Date: 30-Aug-2022

Client PO: St. Patrick Home

Project Description: 121624271

	MDL/Units						
Client ID:		BH22-1, SS3, 5'-7'	BH22-3, SS3, 5'-7'	-	-	-	-
Sample Date:		15-Aug-22 09:00	11-Aug-22 09:00	-	-		
Sample ID:		2236112-01	2236112-02	-	-		
Matrix:		Soil	Soil	-	-		
Physical Characteristics							
% Solids	0.1 % by Wt.	83.9	95.6	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.33	11.67	-	-	-	-
Resistivity	0.1 Ohm.m	24.7	9.53	-	-	-	-
Anions							
Chloride	5 ug/g	<5	17	-	-	-	-
Sulphate	5 ug/g	221	1560	-	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%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					

PARACEL LABORATORIES LTD.

Certificate of Analysis

Order #: 2236112

Report Date: 06-Sep-2022

Client: **Stantec Consulting Ltd. (Ottawa)**

Order Date: 30-Aug-2022

Client PO: St. Patrick Home

Project Description: 121624271

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	5.43	5	ug/g	5.63			3.7	20	
Sulphate	53.7	5	ug/g	57.6			7.0	20	
General Inorganics									
pH	6.95	0.05	pH Units	6.98			0.4	10	
Resistivity	69.3	0.10	Ohm.m	69.3			0.1	20	
Physical Characteristics									
% Solids	91.1	0.1	% by Wt.	96.4			5.7	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	111	5	ug/g	5.63	106	82-118			
Sulphate	154	5	ug/g	57.6	96.3	80-120			

Qualifier Notes:

Login Qualifiers :

Container and COC sample IDs don't match - ID reads BH22-3, SS3, 5'-7' and coc reads BH22-6, SS3, 5'-7'

Applies to Samples: BH22-3, SS3, 5'-7'

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.

NC: Not Calculated

Soil results are reported on a dry weight basis unlesss otherwise noted.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Any use of these results implies your agreement that our total liabilty in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstances be liable to you in connection with this work.

PARACEL LABORATORIES LTD. | TRUSTED. RESPONSIVE RELIABLE.

Paracel ID: 2236112

Chain of Custody (Lab Use Only)

OTTAWA · KINGSTON · NIAGARA · MISSISSAUGA · SARNIA www.paracellabs.com

Page ___ of ___

Client Name:	Stantec Consulting Ltd.	Project Reference:	St. Patrick Home
Contact Name:	Brian Prevost, Ramin Ghassemi	Task #:	
Address:	100A&B-2781 Lancaster Rd. Ottawa ON. K1B-1A7	PO #	121624271
Telephone:	(613) 612-5860	Email Address:	brian.prevost@stantec.com , ramin.ghassemi@stantec.com

TAT: [x] Regular [ ] 3 Day [ ] 2 Day [] 1 Day

Date Required: ______

Criteria: [ ] O. Reg. 153/04 Table __ [ ] O. Reg. 153/11 (Current) Table __ [ ] RSC Filing [ ] O. Reg. 558/00 [ ] PWQO [ ] CCME [ ] SUB (Storm) [ ] SUB (Sanitary) Municipality:______ [ ] Other: ____

Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm/Sanitary Sewer) P (Paint) A (Air) O (Other)

Paracel Order Number: 82368112

Required Analyses

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	Resistivity	PH	Sulphate & Chloride								
1	BH22-1, SS3, 5'-7'	Soil		1	15-Aug-22		x	x	x								
2	BH22-6, SS3, 5'-7'	Soil		1	11-Aug-22		x	x	x								
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

Comments:

Method of Delivery: Swift

Relinquished By (Print & Sign): Moe Komaiha- Stantec	Received by Driver/Depot:	Received at Lab: [signature]	Verified By: [signature]
	Date/Time:	Date/Time: Aug 30/22 9:45	Date/Time: AUG 30, 22 10:55
Date/Time: 30-Aug-22	Temperature: ____ °C	Temperature: 22.3 °C	pH Verified [ ] By:

APPENDIX F

F.1 GEOPHYSICAL INVESTIGATION REPORT

F.2 NBC SEISMIC HAZARD CALCULATION DATA SHEET

F.3 FIGURES F1 TO F4: FACTOR OF SAFETY AGAINST LIQUEFACTION

Stantec

Memo

To:	**Janet Morris** **President & CEO** St. Patrick's Home of Ottawa 2865 Riverside Drive Ottawa ON K1V 6M7	From:	Abderrezak Bouchedda, Ph. D. 110-100 Alexis-Nihon Boulevard Saint-Laurent QC H4M 2N6
File:	**121624271**	Date:	November 14, 2022

Reference: MASW Measurements for Seismic Site Classification of the St. Patrick's Home of Ottawa site, Ottawa

1. INTRODUCTION

Stantec Consulting Ltd. (Stantec) performed a Multi-Channel Analysis of Surface Waves (MASW) sounding in order to determine the shear-wave velocity structure and the seismic site classification at the St. Patrick's Home of Ottawa site, Ottawa, Ontario.

This technical memo describes the equipment and procedure used to perform the MASW measurements and provides a summary of the MASW interpretation.

The MASW survey was completed in conjunction with the geotechnical investigations. The MASW sounding was carried out on August 02, 2022. The approximate location of the MASW sounding is shown on the MASW location plan provided in Appendix 1.

2. GEOLOGICAL BACKGROUND AND SURVEY LOCALIZATION

The studied site, identified as St. Patrick's Home of Ottawa site, is located in 2865 Riverside Drive, Ottawa, Ontario. It is bounded to the west by the Riverside Drive, to the east by a parking and to the south by the existing St. Patrick's Home of Ottawa building.

Based on the geotechnical report, the observed stratigraphy mainly consisted of topsoil (thickness of 0.2 m to 1 m), over sand fill and/or a silty clay fill (thickness of 0.6 m to 2.8 m), over a native silty clay/clayey silt deposit (thickness of 1.1 m to 4.2 m), over a glacial till deposit (thickness of 0 m to 3.7 m) over bedrock. The glacial till deposit consisted of a loose to very dense silty sand with some gravel. The bedrock consisted of shaley Limestone. It was encountered in boreholes BH22-06 and BH22-02 at depths of 5.2 m and 11.8 m.

In order to determine the shear wave velocity structure and seismic site classification of St. Patrick's Home site, a MASW sounding using passive and active measurements was conducted on the site as shown on location map of Appendix 1. Table 1 gives positions of active and passive MASW profiles. Active MASW measurements were carried out using 3 m and 1 m receiver spacings. The passive MASW measurements were achieved using L-shape profile using 5 m spacing.

Reference: MASW Measurements for Seismic Site Classification of the St. Patrick's Home of Ottawa site, Ottawa

Table 1: MASW sounding position

Active MASW profile (NAD83-UTM zone 18N)			
Position	**Start**	**Center**	**End**
EAST (m)	445978.00	446013.00	446048.00
NORTH (m)	5024337.00	5024341.00	5024345.00
Passive MASW L-shape profile (NAD83-UTM zone 18N)			
Position	**Start**	**Corner**	**End**
EAST (m)	446049.00	446040.00	445980.00
NORTH (m)	5024316.00	5024370.00	5024365.00

3. MASW METHOD

The multichannel analysis of surface waves (MASW) method deals with surface waves in the lower frequencies (e.g., 1-30 Hz) and uses a much shallower depth range of investigation (e.g., a few to a few tens of meters). The active MASW method generates surface waves through an impact source like a sledgehammer, whereas the passive method uses surface waves generated passively by cultural (e.g., traffic) or natural (e.g., thunder and tidal motion) activities.

In some cases, the energy generated by a sledgehammer impact source could be insufficient to reach a depth of investigation of more than 30 m. Consequently, it is recommended to perform passive measurements, in addition to active measurements, to improve the depth of investigation of active MASW data (> 20 m). In our case, passive measurements are always performed. During the data processing step, passive data are used only when the resolution of the low frequency part of dispersion image is improved.

In the case of active MASW measurements, the length of the receiver spread is directly related to the longest wavelength that can be analyzed, which determines the maximum depth of investigation, while receiver spacing is related to the shortest wavelength and therefore determines the shallowest resolvable depth of investigation.

The entire procedure for MASW usually consists of three steps as illustrated in Figure 1 (Park et al., 2007): (1) acquiring multichannel field records (or shot gathers); (2) extracting dispersion curves; and (3) inverting the dispersion curve to obtain 1D (depth) Vs sounding.

To process active and passive data, we used ParkSeis software which uses an effective way of combining active and passive dispersion images as described in Park et al. (2005). In addition, passive data are processed using dynamic azimuth detection algorithm of Park (2010).

Because all surface-wave methods, in theory, are based on a layered earth model, the data analysis steps inevitably apply lateral averaging of subsurface conditions along the surface distance occupied by the receiver array. As a result, the interpreted MASW sounding can best represent the subsurface velocity (Vs) model below the center of the profile.

Reference: MASW Measurements for Seismic Site Classification of the St. Patrick's Home of Ottawa site, Ottawa

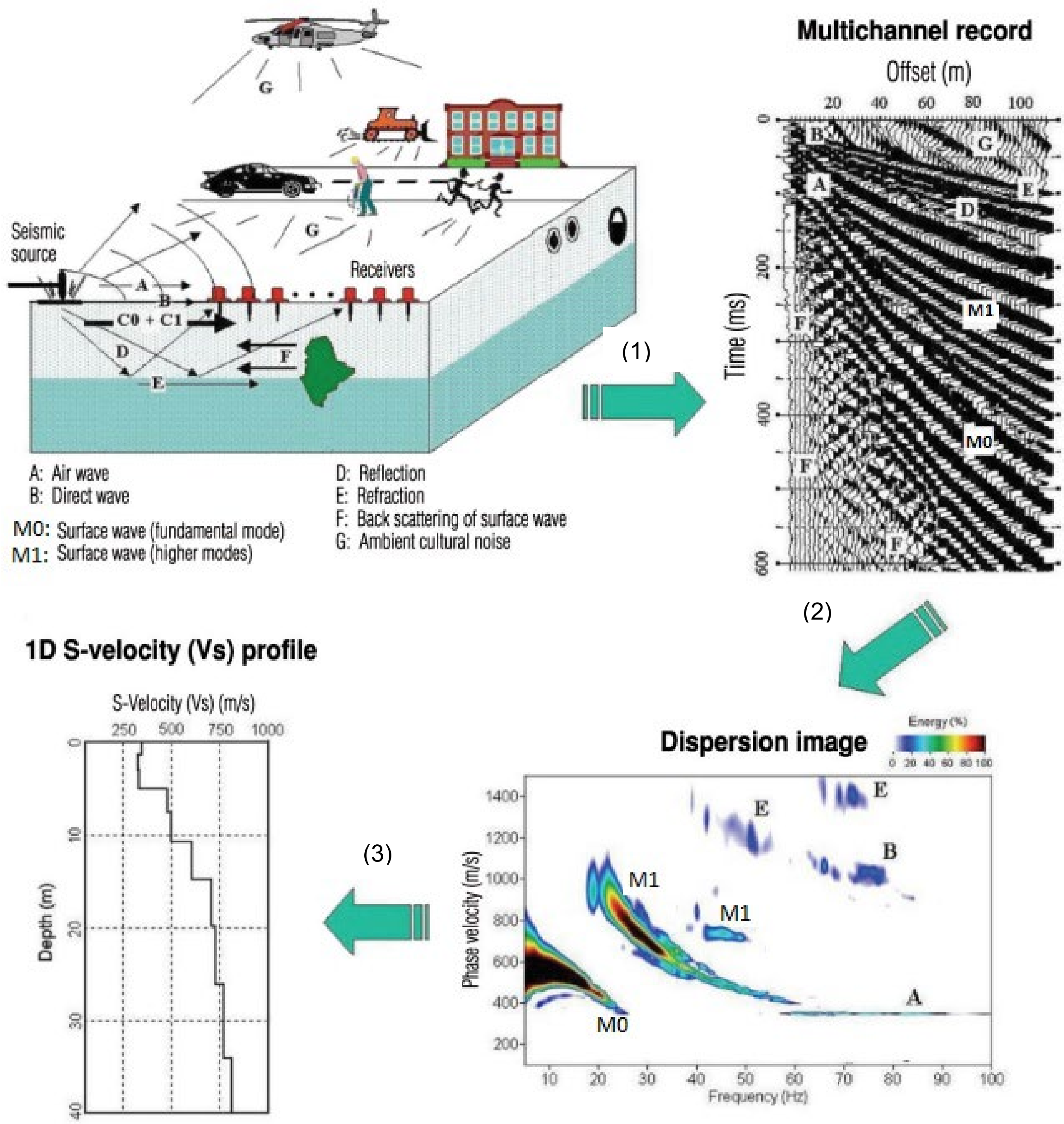
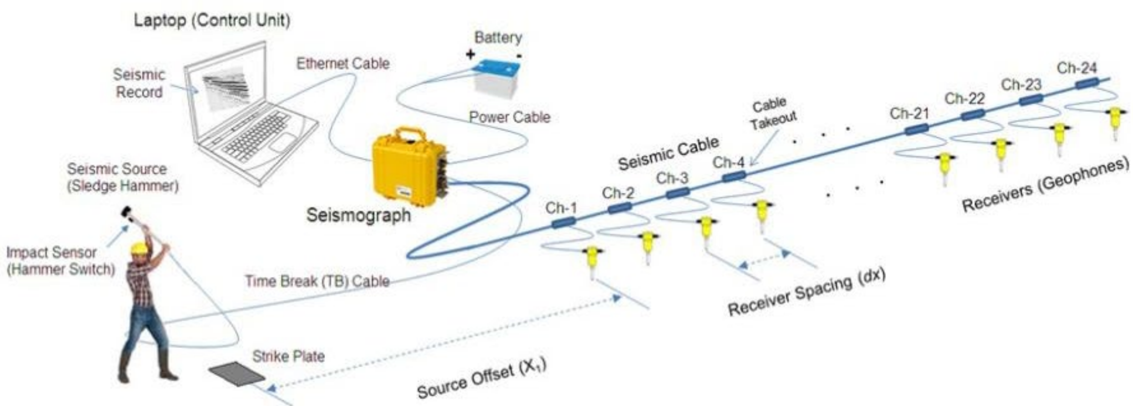


Figure 1: Illustration of active and passive MASW methods (Park et al., 2007). (1) data acquisition, (2) dispersion image generated using seismic data and (3) Vs profile obtained using 1-D inversion of dispersion curve of fundamental mode M0.

Reference: MASW Measurements for Seismic Site Classification of the St. Patrick's Home of Ottawa site, Ottawa

4. DATA ACQUISTION

MASW data acquisition was carried out using Geometrics MASW kit system (USA) which consists of a 24-channel seismograph (Geode), a laptop, a seismic cable with 24 hookups (takeouts), 24 low-frequency 4.5-Hz geophones with tripods, a 18lb sledgehammer and aluminum strike plate. The figure 2 illustrates a typical



configuration of MASW data acquisition.

Figure 2: MASW data acquisition setup using 24-Channel seismic acquisition system (From: www.masw.com). The seismic cable is connected to the geode which is controlled by a laptop. An impact sensor is fixed on the hammer and connected to the geode using a trigger cable.

The following acquisition parameters were used for:

- Active MASW measurements:
  - Array length = 69 m and 23 m.
  - Source: 18 lb sledgehammer.
  - Receiver Spacing = 3 m and 1 m.
  - Number of receivers = 24.
  - Stacking: 2 to 5.
  - Source positions:
    69 m array length: direct shots at 3, 18 and 36 m; reverse shots at 3, 18 m.
    23 m array length: shots on both sides at 3, 6, 12 m.

- Passive MASW measurements:
  - Array type = L-shape
  - Array length = 115 m.
  - Receiver Spacing = 5 m.
  - Number of receivers = 24.
  - Time window: 3 records of 4 min length.

Reference: MASW Measurements for Seismic Site Classification of the St. Patrick's Home of Ottawa site, Ottawa

Quality of data was "EXCELLENT" for all obtained records with very high signal-to-noise ratios (generally SNR > 0.8) for the fundamental-mode dispersion energy as shown in the appendix 2.

5. RESULTS

For the MASW sounding, all active records' dispersion images are stacked (active measurements with 3 m and 1 m receiver spacing), and a one fundamental-mode (M0) dispersion curve is extracted from the stacked image (Figure 3). Note that, the passive MASW measurements were not used to produce the stacked dispersion image because the low frequency phase velocity between 10 Hz and 20 Hz is well defined by active data. A 1-D shear-velocity (Vs) profile of 10-layers model is obtained by inversion of the extracted dispersion curve as shown in Figure 4. Table 2 summarizes the obtained Vs-30 and the corresponding site class.

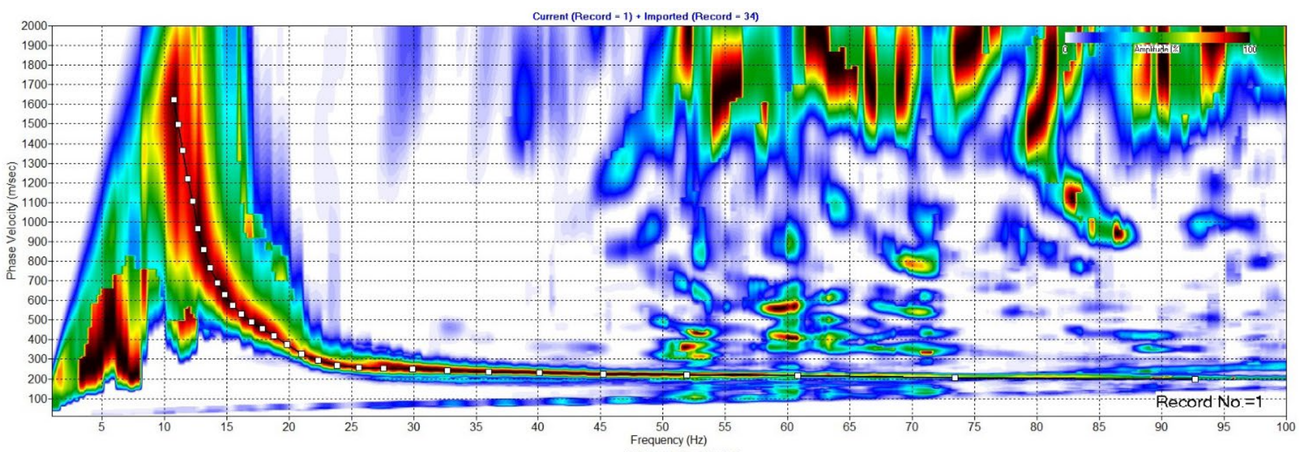


Figure 3: Fundamental-mode (M0) dispersion curve extracted from the stacked dispersion image of active MASW measurements.

Reference: MASW Measurements for Seismic Site Classification of the St. Patrick's Home of Ottawa site, Ottawa

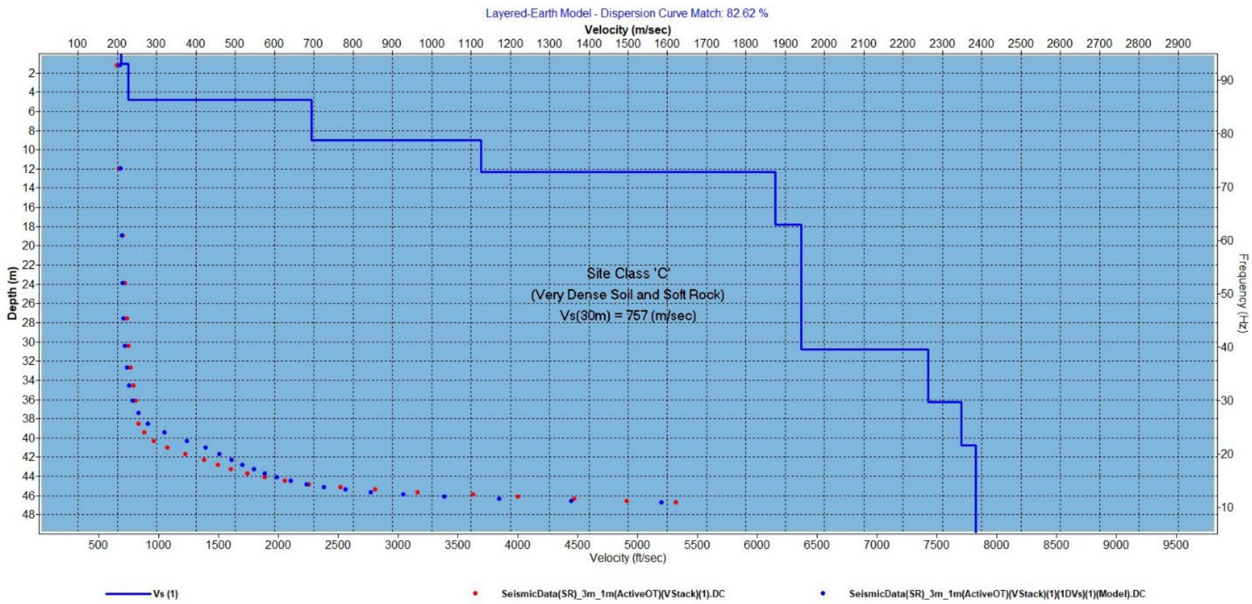


Figure 4: Shear-velocity model obtained by inverting the fundamental mode (M0) dispersion curve of figure 3.

According to the analyzed 1-D Vs profile, top 9 m of subsurface consists of stiff soil with velocities (Vs's) in 209-228 m/s (topsoil, sand and silty clay fill, and silty clay native soil deposit), over very dense soil with higher velocity of 694 m/s corresponding to the glacial till. These materials are then followed by stiffer materials at about 9m depth that have velocities in 1125-2384 m/s, indicating bedrock.

From MASW 1-D Vs profile, the average Vs for top 30 m depths (i.e., Vs30-m) is calculated. The value as Vs-30 of MASW is 757 m/s, which puts the site into class C (Very Dense Soil and Soft Rock) according to the Table 4.1.8.4-A of National Building Code of Canada 2015 (please see Appendix 4).

6. CONCLUSIONS

According to the MASW measurements and the National Building Code of Canada (2015), the site class of St. Patrick's Home of Ottawa site is class C (Very Dense Soil and Soft Rock).

Table 2: 1-D Vs model as obtained from inversion of the extracted dispersion curve of fundamental mode.

St. Patrick's Home of Ottawa site			
Layer #	Depth(m)	Thickness(m)	Vs(m/s)
1	1,02	1,02	209,7
2	3,558	2,538	228,34
3	4,805	1,247	228,56
4	9,009	4,204	694,21
5	12,331	3,32	1125,57
6	17,803	5,472	1874,56
7	30,746	12,943	1940,64
8	36,261	5,515	2263,78
9	40,8	4,539	2348,23
10	Half-Space	N/A	2384,96
		Vs-30 (m/s)	756,7

7. LIMITATIONS

- The estimation of the shear wave velocity profile from surface wave analyses requires the solution of an inverse problem. The result is affected by solution non-uniqueness as several different models may provide similar goodness of fit with the experimental data.
- The resolution markedly decreases for increasing depth. Therefore, relatively thin deep layers cannot be identified at depth and the accuracy of the location of layer interfaces is poor at large depth.
- Only 1D models of the subsurface are considered, hence the outlined procedures are used by considering no significant lateral variations of the seismic properties with flat or mildly inclined ground surface.

Reference: MASW Measurements for Seismic Site Classification of the St. Patrick's Home of Ottawa site, Ottawa

8. REFERENCES

NBCC, 2015. National Building Code of Canada, National Research Council of Canada, Ottawa, Ontario.

Houle and Chevrier Engineering, 2010, Geotechnical Investigation, Proposed PHASE I Redevelopment, ST. PATRICK'S HOME OF OTTAWA, 2865 RIVERSIDE DRIVE, OTTAWA, ONTARIO.

Park, C, B, R, D, Miller, J, H, Xia, and J, Ivanov, 2007, Multichannel analysis of surface waves (MASW)-active and passive methods: The Leading Edge, 26, 60-64.

Park, C, B, R, D, Miller, and J, H, Xia, 1999, Multichannel analysis of surface waves: Geophysics, 64, 800-808.

Xia, J, H, R, D, Miller, and C, B, Park, 1999, Estimation of near-surface shear-wave velocity by inversion of Rayleigh waves: Geophysics, 64, 691-700.

Stantec Consulting Ltd.

Abderrezak Bouchedda, Ph. D.
Geophysical Specialist

Phone: 514 315 1124
Abderrezak.Bouchedda@stantec.com

APPENDICES

Appendix 1: MASW SOUNDING LOCATION.

Appendix 2: MASW DATA QUALITY.

Appendix 3: PARKSEIS COLOR CODE USED FOR SEISMIC SITE CLASSIFICATION (VS30-M OR VS100-FT).

Appendix 4: SITE CLASSIFICATION FOR SEISMIC SITE RESPONSE (THE NATIONAL BUILDING CODE OF CANADA, 2015).

Appendix 5: GEOMETRICS MASW MEASUREMENT KIT

APPENDIX 1

MASW SOUNDING LOCATION

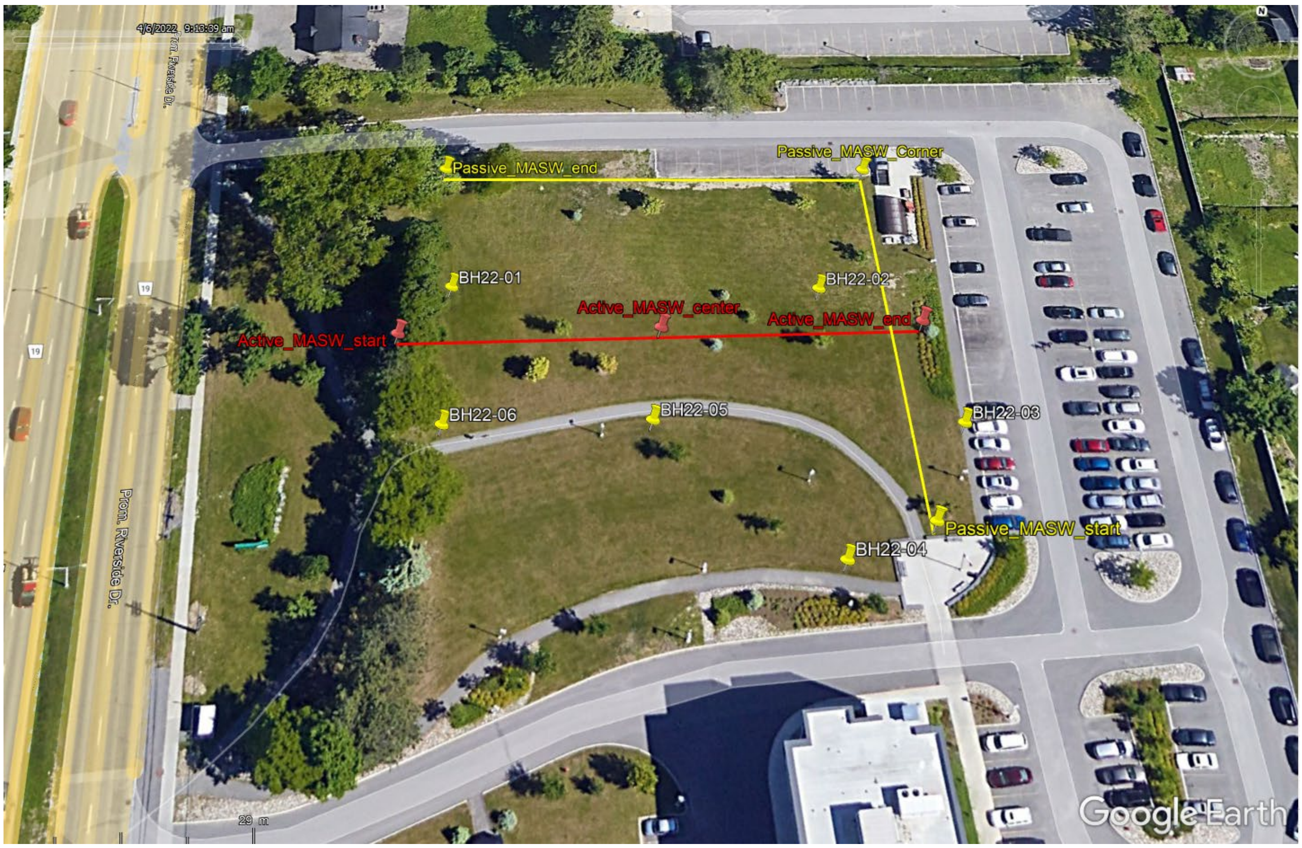


Figure 1A: Google Earth image of active (in red) and passive (in yellow) MASW profiles location.

APPENDIX 2

MASW DATA QUALITY

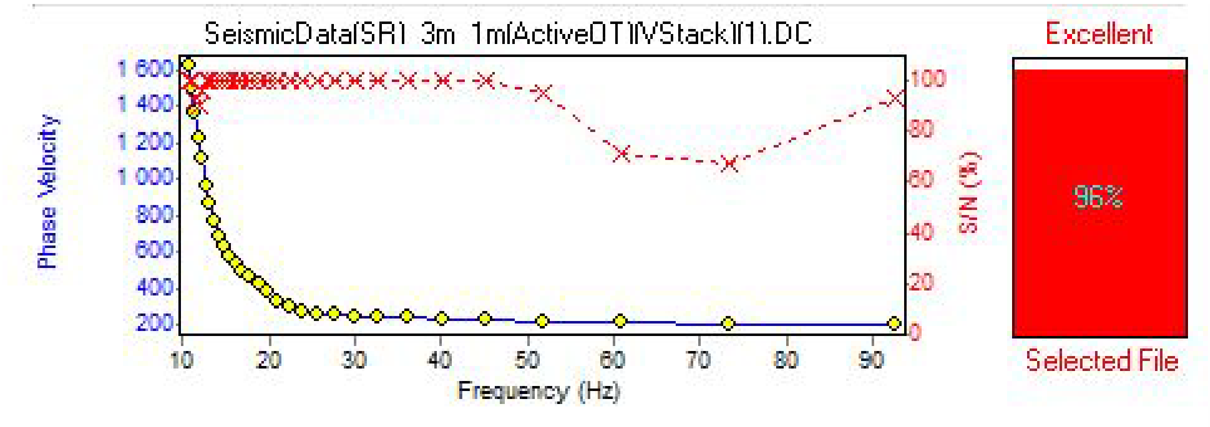


Figure 2A: Dispersion curve (yellow diamond) and signal to noise ratio curve (S/N) in % (red cross). Mean value of S/N (in right).

APPENDIX 3

PARKSEIS COLOR CODE USED FOR SEISMIC SITE CLASSIFICATION (VS30-M OR VS100-FT)

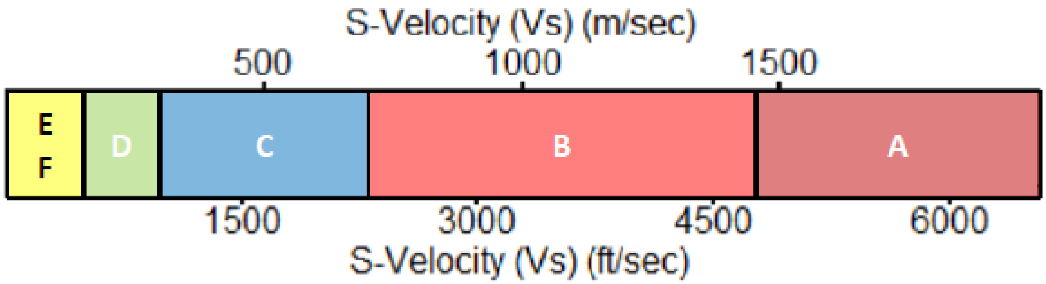


Table 1A: NBCC pour la classe sismique des sites basé sur la vitesse de cisaillement (Vs),

Site Class	S-Velocity (Vs) (ft/sec)	S-Velocity (Vs) (m/sec)
A (Hard Rock)	> 5,000	> 1500
B (Rock)	[2500, 5000]	[760, 1500]
C (Very Dense Soil and Soft Rock)	[1200, 2500]	[360, 760]
D (Stiff Soil)	[600, 1200]	[180, 360]
E (Soft Clay Soil)	< 600	< 180
F (Soils Requiring Add'l Response)	Vs < 600 and meeting some additional conditions,	Vs < 180 and meeting some additional conditions,

* National Building Code of Canada 2015

APPENDIX 4

SITE CLASSIFICATION FOR SEISMIC SITE RESPONSE (THE NATIONAL BUILDING CODE OF CANADA, 2015)

Table 4.1.8.4.-A
Site Classification for Seismic Site Response
Forming Part of Sentences 4.1.8.4.(1) to (3)

Site Class	Ground Profile Name	Average Properties in Top 30 m, as per Note A-4.1.8.4.(3) and Table 4.1.8.4.-A		
		Average Shear Wave Velocity, $\bar{V}_{s30}$, m/s	Average Standard Penetration Resistance, $\bar{N}_{60}$	*Soil* Undrained Shear Strength, s_u
A	Hard *rock*(1)(2)	$\bar{V}_{s30} > 1500$	n/a	n/a
B	*Rock*(1)	$760 < \bar{V}_{s30} \leq 1500$	n/a	n/a
C	Very dense *soil* and soft *rock*	$360 < \bar{V}_{s30} < 760$	$\bar{N}_{60} > 50$	$s_u > 100$ kPa
D	Stiff *soil*	$180 < \bar{V}_{s30} < 360$	$15 \leq \bar{N}_{60} \leq 50$	50 kPa $< s_u \leq$ 100 kPa
E	Soft *soil*	$\bar{V}_{s30} < 180$	$\bar{N}_{60} < 15$	$s_u < 50$ kPa
		Any profile with more than 3 m of *soil* with the following characteristics: • plasticity index: PI > 20 • moisture content: $w \geq 40\%$, and • undrained shear strength: $s_u < 25$ kPa		
F	Other *soils*(3)	Site-specific evaluation required		

Notes to Table 4.1.8.4.-A:

(1) Site Classes A and B, hard *rock* and *rock*, are not to be used if there is more than 3 m of softer materials between the *rock* and the underside of footing or mat *foundations*. The appropriate Site Class for such cases is determined on the basis of the average properties of the total thickness of the softer materials (see Note A-4.1.8.4.(3) and Table 4.1.8.4.-A).

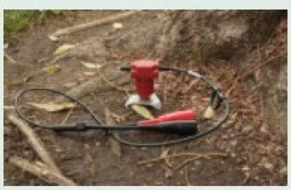
(2) Where $\bar{V}_{s30}$ has been measured in-situ, the F(T) values for Site Class A derived from Tables 4.1.8.4.-B to 4.1.8.4.-G are permitted to be multiplied by the factor $0.04 + (1500/\bar{V}_{s30})^{1/2}$.

(3) Other *soils* include:
- (a) liquefiable *soils*, quick and highly sensitive clays, collapsible weakly cemented *soils*, and other *soils* susceptible to failure or collapse under seismic loading,
- (b) peat and/or highly organic clays greater than 3 m in thickness,
- (c) highly plastic clays (PI > 75) more than 8 m thick, and
- (d) soft to medium stiff clays more than 30 m thick.

APPENDIX 5

GEOMETRICS MASW MEASUREMENTS KIT

MASW Kit

	1 CT: SPREAD CABLE, 24 TAKEOUTS AT 5-M INTERVAL
	26 CT: VERTICAL GEOPHONE, 4.5 HZ
	26 CT: GEOPHONE TRIPODS
	1 CT: HAMMER SWITCH

	2 CT: 300 FT (92 M) TRIGGER EXTENSION CABLE
	1 CT: 12"X12"X2" POLYETHYLENE STRIKER PLATE
	1 CT: 16 LB SLEDGEHAMMER
	**1 CT: GEODE**

2020 National Building Code of Canada Seismic Hazard Tool

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

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_C
Latitude (°)	45.37
Longitude (°)	-75.689

Please select one of the tabs below.

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

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

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

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

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

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

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	$PGA(X_C)$	$PGV(X_C)$
0.661	0.393	0.21	0.0964	0.0255	0.00841	0.354	0.269

The log-log interpolated 2%/50 year $S_a(4.0, X_C)$ value is : **0.0353**

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

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

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	$PGA(X_C)$	$PGV(X_C)$
0.383	0.227	0.116	0.0523	0.0132	0.00441	0.208	0.147

The log-log interpolated 5%/50 year $S_a(4.0, X_C)$ value is : **0.0185**

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

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	$PGA(X_C)$	$PGV(X_C)$
0.243	0.143	0.0716	0.0313	0.00753	0.00252	0.131	0.0888

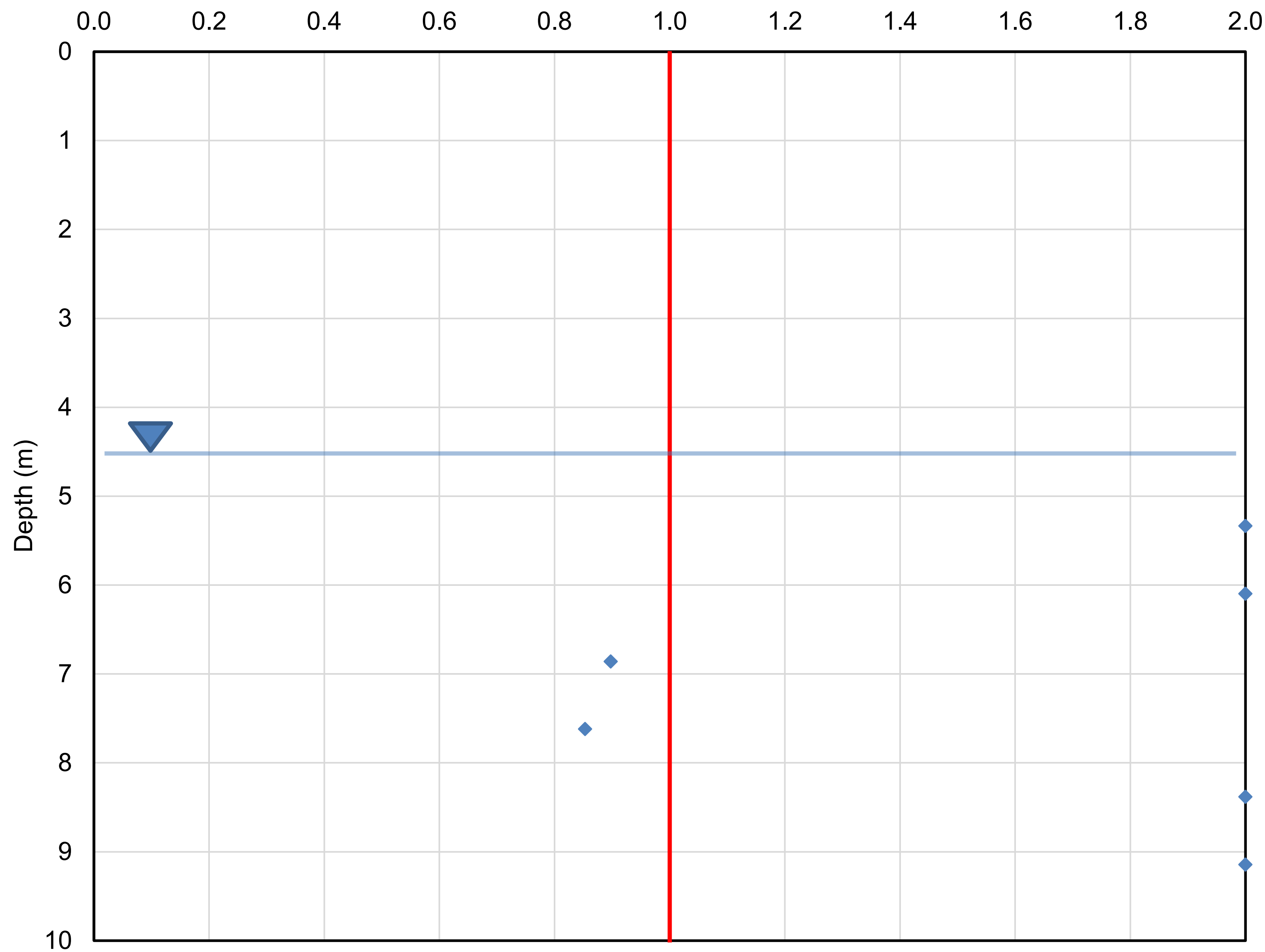
The log-log interpolated 10%/50 year $S_a(4.0, X_C)$ value is : **0.0107**

Download CSV

Go back to the seismic hazard calculator form

Date modified: 2021-04-06

Fig F1. Factor of Safety against Liquefaction - BH22-2



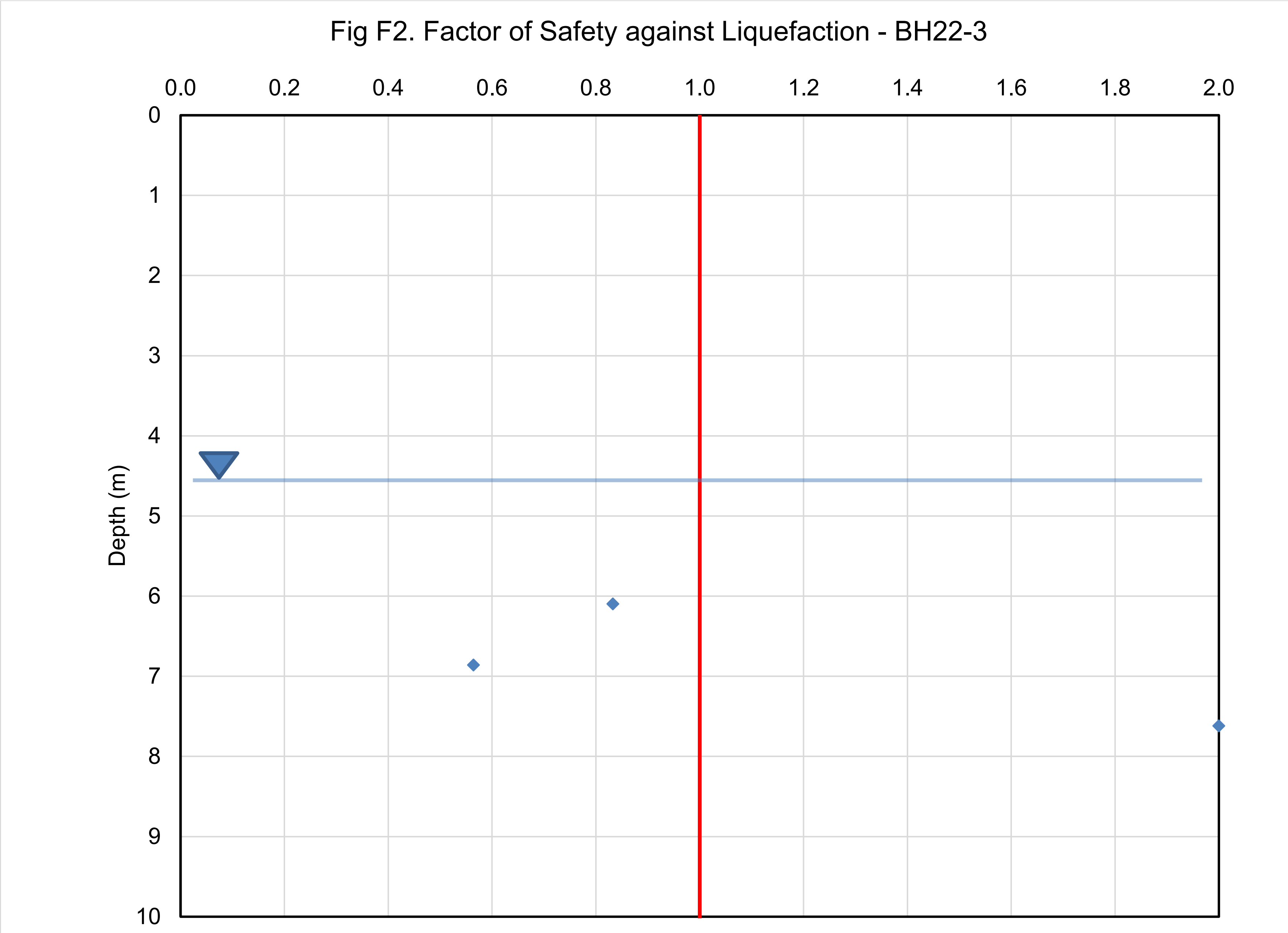


Fig F2. Factor of Safety against Liquefaction - BH22-3
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
0
1
2
3
4
5
6
7
8
9
10
Depth (m)

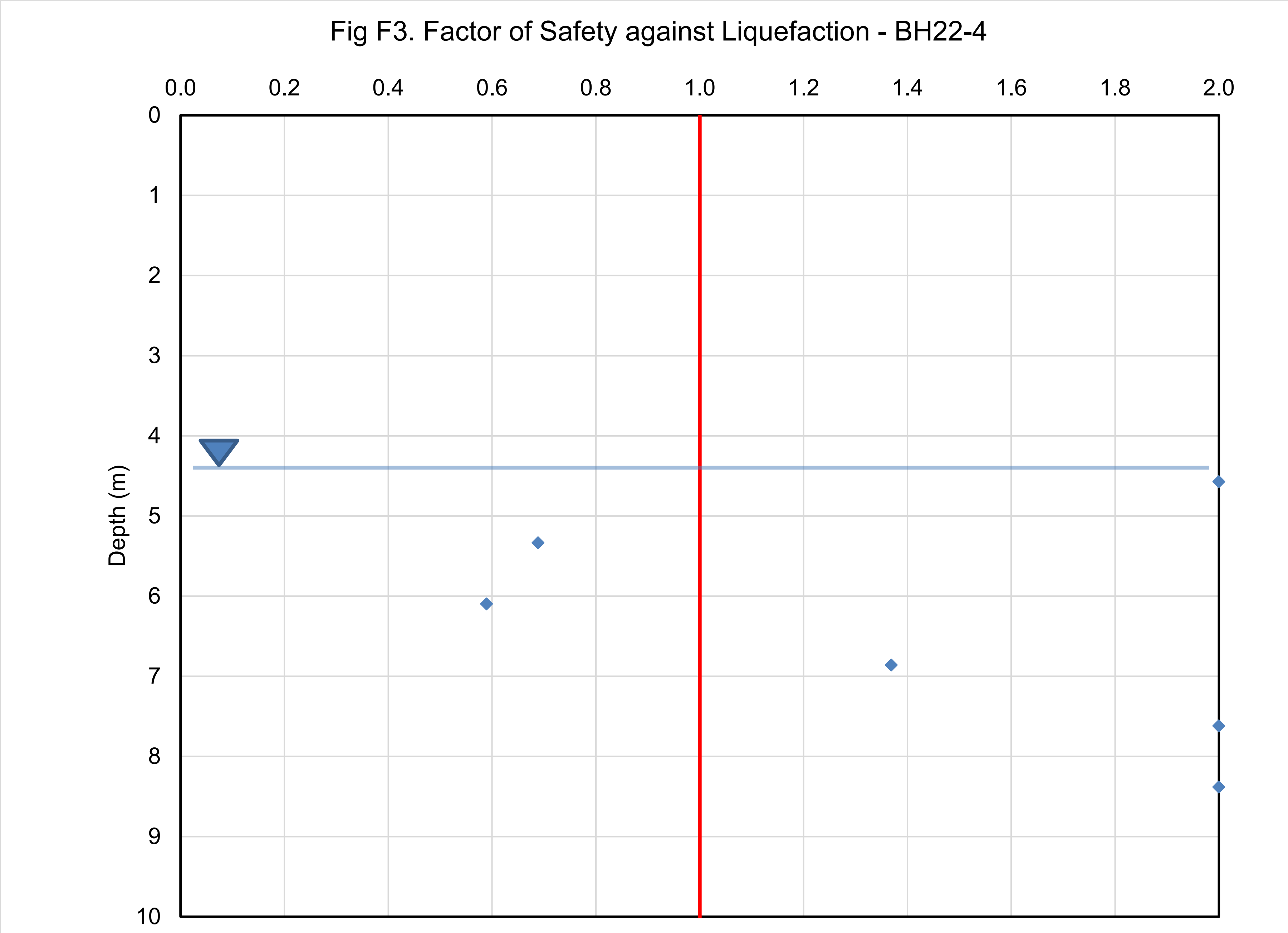
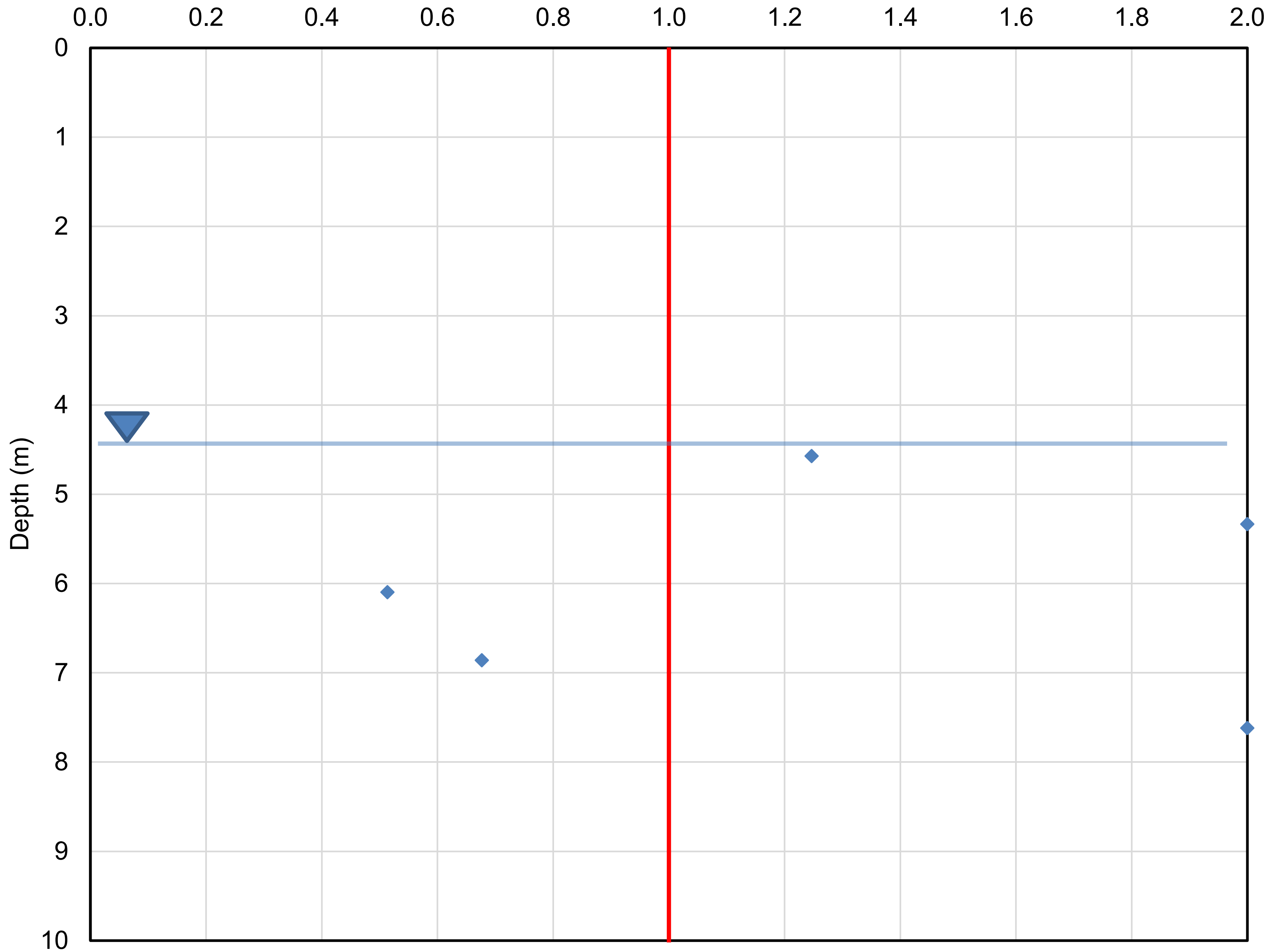


Fig F3. Factor of Safety against Liquefaction - BH22-4
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Depth (m)
0
1
2
3
4
5
6
7
8
9
10

Fig F4. Factor of Safety against Liquefaction - BH22-5



APPENDIX G

G.1 ROCK ANCHOR: RESISTANCE TO ROCK MASS FAILURE

Rock Anchor

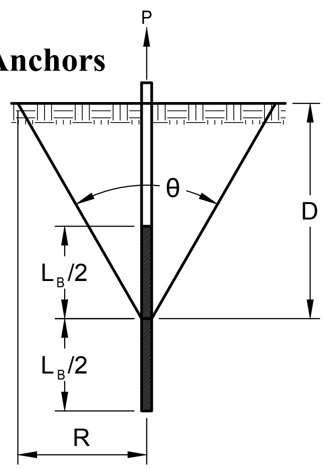
Resistance to Rock Mass Failure

Required Safety Factor for Resistance to Rock Mass Failure: $W_R / P \geq 2.0$

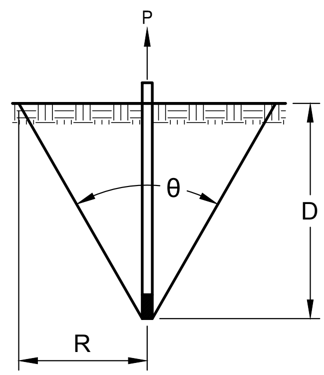
Design Considerations:

1. Use 60° or 90° apex angle as per recommendations in the geotechnical report

Vertical Anchors

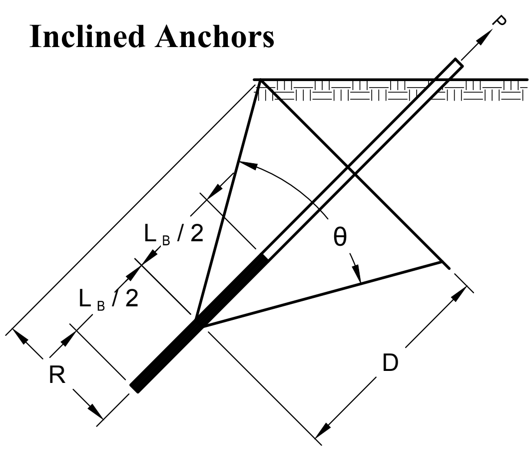


Grouted Rock Anchors

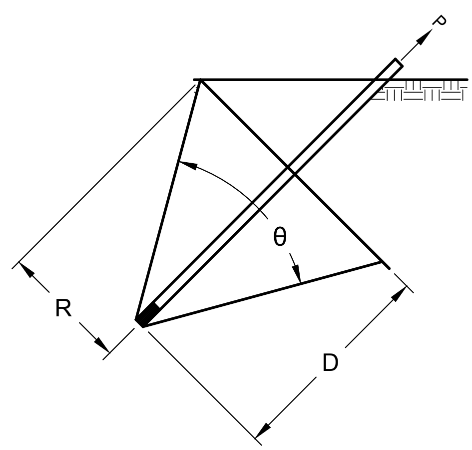


Mechanical Rock Anchors

Inclined Anchors



Grouted Rock Anchors



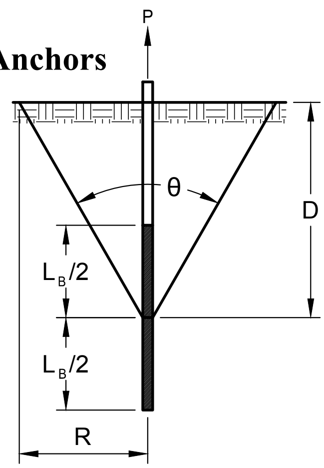
Mechanical Rock Anchors

P	=	**Resultant of maximum anchor forces**
D	=	**Height of rock cone**
R	=	**Radius of rock cone**
θ	=	**Apex angle**
L_B	=	**Bond Length**
Y_R	=	**Submerged unit weight of bedrock**
W_R	=	**Weight of rock cone ($W_R = 1/3\pi R^2 D Y_R$)**

Rock Anchor

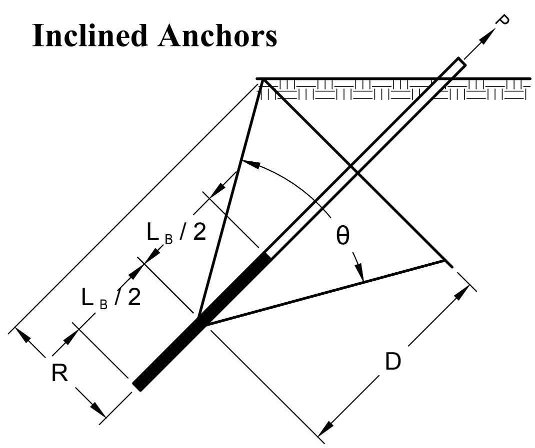
Resistance to Rock Mass Failure

Vertical Anchors



Grouted Rock Anchors

Inclined Anchors



Grouted Rock Anchors

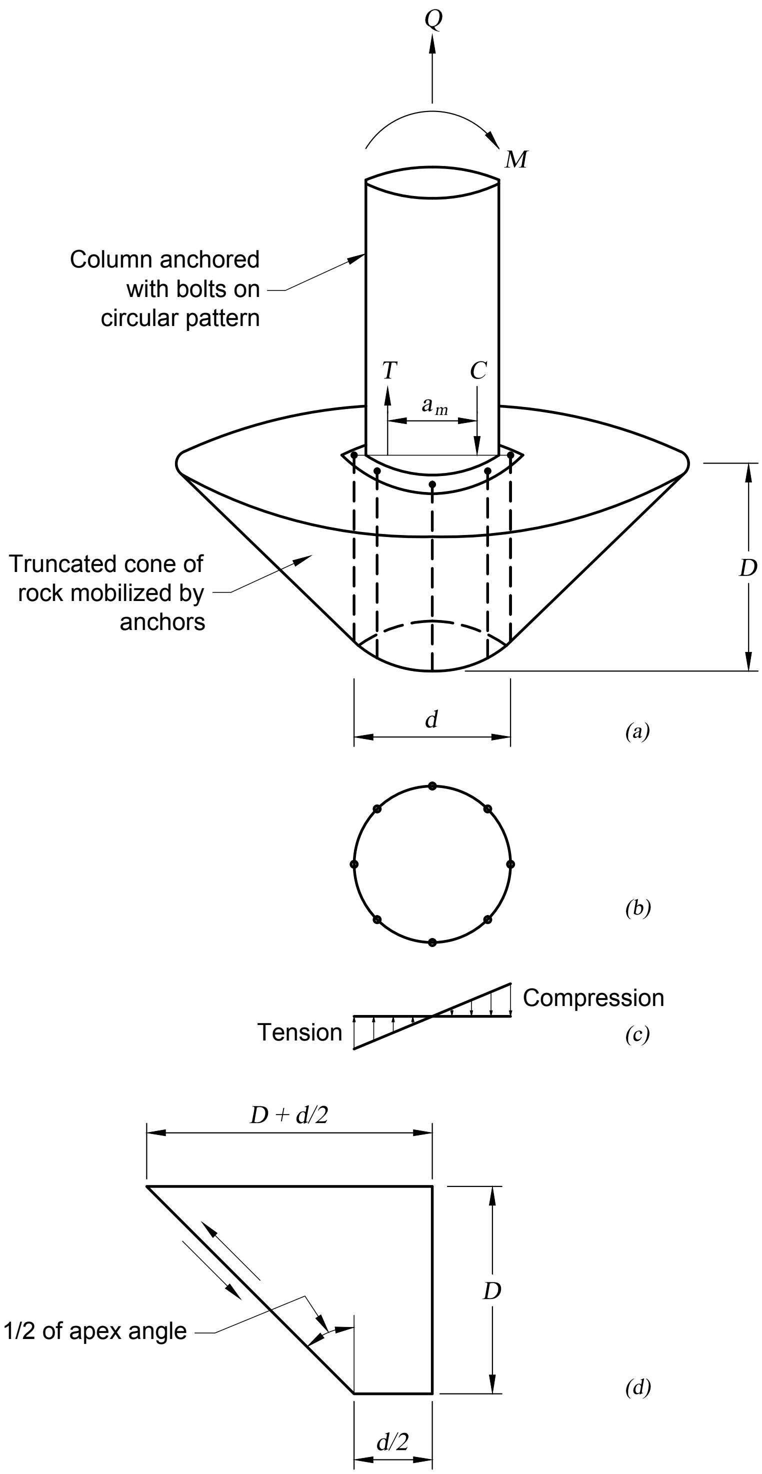
$$P = \pi/3 \tan^2(\theta/2)\, D^3\, \gamma_{R'} + \frac{\sigma_t\, \pi\, D^2 \tan(\theta/2)}{\cos(\theta/2)}$$

P	=	**Ultimate resistance to rock mass failure**
D	=	**Height of rock cone**
θ	=	**Apex angle**
L_B	=	**Bond Length**
$\gamma_{R'}$	=	**Submerged unit weight of bedrock**
σ_t	=	**Tensile strength of rock on cone surface**

Stantec

Group of Anchors

Combined Uplift and Moment Loading



Design Considerations:

1. Use 60° or 90° apex angle as per recommendation in the geotechnical report to calculate the following:

   W''_c The buoyant weight of the truncated rock cone

   A'_c The surface area of one half of the truncated cone, ignoring the horizontal base of the cone

   $$A'_c = \frac{\pi}{\sqrt{2}} (D^2 + dD)$$

2. Only the rock on the surface of the uplift half of the cone is used to calculate the mobiled tensile resistance force on the surface of the rock cone.

3. The resisting force developed on the curved surface area of one half of the cone is defined as follows:

   $$F'_{(r)} = \delta_t A'_c$$

   δ_t The tensile strength of the rock on the surface of the cone as provided in the geotechnical report

4. The factored axial resistance of the group of anchors is defined as follows:

   $$Rf = \frac{W'_c}{FOS(1.5)} + \frac{F'_{(r)}}{FOS\ (3.0)}$$

5. The factored resistance, Rf, should be compared to the sum of "the axial force" and "the tensile force induced by the moment".

Reference: Wyllie, D.C. (1999) Foundations on Rocks, Second Edition E & FN Spon (Routhledge), New York.