Geotechnical Investigation Report
788 March Road, Kanata (Ottawa), Ontario

Revision: 3

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1 INTRODUCTION

Geofirma Engineering Ltd. (Geofirma) was retained by Omnipex Real Estate Inc. to complete a geotechnical investigation of the property located at 788 March Road, Kanata, herein referred to as `the site`. The geotechnical assessment has been completed to support development of the site.

The report presents the factual results of the geotechnical investigation and provides geotechnical recommendations for the proposed development of the site. Work has been completed in accordance with the *Geotechnical Investigation and Reporting Guidelines for Development Applications in the City of Ottawa* and the Canadian Foundation Engineering Manual, 4th Ed. (2006).

1.1 Project and Site Description

The site is currently a vacant parcel of land located at the southeast corner of March Road and Klondike Road, and bordered by Shirley's Brook to the northeast. The total area of the parcel is approximately 1.2 hectares (~3 acres). Land use surrounding the site is primarily low-rise commercial, with residential subdivisions located south of the site.

Site topography is moderately sloping downward to the northeast (toward Shirley's Brook). Elevation of the site adjacent to March Road is approximately 78 mASL, with the elevation at Shirley's Brook of approximately 72 mASL. The site is well vegetated with established grass and shrubs on the southern portion the site and mature trees on the north portion of the site.

It is understood that the proposed development of the site will include a multi-level residential complex, with one level of underground parking. The exact depth of footing for the proposed structure was not known at the time of submitting this report but it is understood that the structure will be founded at an elevation of about 73 mASL.

The location of the site is shown on Figure A.1, the proposed building footprint is illustrated on Figure A.2, and site topography is presented on Figure A.3, Appendix A.

1.2 Regional Geological Setting

The site is located within the Clay Plains physiographic region (Chapman and Putnam, 1984) and consists of fine-texture glaciomarine deposits, including silt and clay with minor sand and gravel (OGS, 2011). Bedrock at the site is mapped as dolostone and sandstone of the Beekmantown Group (OGS, 2011).

1.3 Background

There are no available geotechnical reports for the site; however, a review of the Ontario Geotechnical Borehole Database was completed prior commencing field activities to provide some indication on site specific stratigraphic conditions. Three boreholes are mapped within 200 m of the site: ID 609813, 609814 and 609816 with bedrock depths of 6.4 m, 5.5 m and 9.1 m, respectively.

1.4 Purpose and Scope

The purpose of the geotechnical investigation at the site was to characterize the engineering properties of the subsurface materials, and provide recommendations for site development based on the soil, rock and groundwater conditions.

The scope of work for the geotechnical investigation included the following:

- Review of existing background geological information, primarily data from Ontario Geotechnical Borehole Database.
- Completion of a borehole drilling program to provide spatial coverage and to characterize material properties of the key stratigraphic units.
- Completion of a geotechnical laboratory testing program to characterize the engineering properties of site soil and bedrock.
- Collection of representative soil and groundwater samples to characterize corrosion potential and recommend cement type; and
- Preparation of a geotechnical report providing a summary of field observations and subsurface conditions, laboratory results and geotechnical recommendations.

1.5 Report Organization

The Geotechnical Report is organized into six Sections and eight Appendices. Section 1 provides an overview of the site, and the purpose and scope of the project. The investigation methodology is outlined in Section 2. Section 3 presents a summary of the factual site data collected during the geotechnical investigation. Section 4 provides an assessment of the geotechnical properties of the subsurface materials and geotechnical recommendations. Limitations of this report are outlined in Section 5 and references are included in Section 6.

Appendices include report figures and borehole logs, as well as supporting laboratory results, hydraulic testing results and a copy of the seismic information for the site.

2 METHODOLOGY

Project activities were conducted by Geofirma between May 18 and June 15, 2018. This included the following:

- Slope stability assessment (site inspection)
- Underground utility locates
- Borehole drilling
- Piezometer installation and collection of groundwater elevations
- Surveying
- Geotechnical laboratory analysis; and
- Geochemical laboratory analysis.

2.1 Slope Stability Assessment

A site inspection was completed by Steve Gaines, M.A.Sc., P.Eng., on May 18, 2018 to visually inspect the slope along Shirley's Brook. Specifically, the site inspection was completed to observe current site conditions, slope characteristics, vegetative cover, as well as drainage and water course characteristics.

A preliminary assessment of the slope was completed in accordance with the guidelines provided in the Ontario Ministry of Natural Resources (MNR) Geotechnical Principles for Stable Slopes (MNR, 1998). Based on the observed site features, a slope rating of 22 was assigned and it was concluded that there was no significant toe erosion at the slope toe, the vegetation is well established, and there is no evidence of past instability. A copy of the preliminary inspection report is included in Appendix B. Site photographs collected during the inspection are included as Appendix C.

Based on MNR guidelines, a rating of less than 24 requires no further investigation; however, given the proposed development, an assessment of the post-development conditions (i.e. structural loading and grade-raises) is evaluated and discussed in Section 4.11.

2.2 Underground Utility Locates

Ontario OneCall was contacted to identify the location of all underground buried utilities at the site. Utilities including telephone, gas, hydro, cable/fiber and municipal services were cleared through these services.

2.3 Borehole Drilling

A total of eight (8) boreholes were drilled using a CME-75 track-mounted drilling rig, operated by Aardvark Drilling Inc. (ADI) of Carleton Place, Ontario. Drilling was completed between June 5 and 7, 2018. Boreholes were identified sequentially from BH18-01 to BH18-08 and presented on Figure A.2, Appendix A.

BH18-01 and BH18-02 were advanced using 203 mm hollow stem augers, while advancing a 51 mm diameter split spoon sampler. The remaining boreholes were advanced using a 152 mm diameter solid stem auger, also advancing a 51 mm diameter split spoon. Standard Penetration Tests (SPT) N-values were recorded by the supervising Geofirma technician. Field shear vane tests were collected, where appropriate, using a standard N-sized vane.

Table 1 summarizes the borehole drilling program, including ground surface elevation, depth to bedrock, and total drilled depth.

Table 1 Summary of Bedrock and Total Borehole Depth and Elevations

Borehole ID	Ground Surface Elevation (mASL)	Interpreted Bedrock Depth (Elevation)	Borehole Depth (Elevation)
BH18-01	74.86	5.6 (69.7)	9.3 (66.0)
BH18-02	77.07	6.2 (70.5)	6.2 (70.5)
BH18-03	77.26	6.2 (71.0)	6.2 (71.0)
BH18-04	76.57	5.9 (70.3)	5.9 (70.3)
BH18-05	77.06	5.6 (71.4)	6.6 (70.5)
BH18-06	76.46	5.6 (70.8)	5.6 (70.8)
BH18-07	77.03	5.5 (71.6)	5.5 (71.6)
BH18-08	76.35	5.3 (70.7)	5.3 (70.7)

Note: depth is referenced in metres below ground surface (mBGS) and elevation as metres above sea level (mASL)

Soil samples were collected from each borehole in 0.6 m intervals, and are identified as BH18-XX-Y, where XX is the borehole identifier and Y is the sequential sample interval. For example, BH18-02-3 indicates the third sample from borehole number 2. Samples were inspected in the field by an experienced field technician and collected in plastic freezer bags to preserve moisture conditions. Samples were brought back to the Geofirma office for detailed inspection by a geotechnical engineer. Selected samples were submitted for additional laboratory testing, as discussed in Section 2.7.

Complete borehole logs are included in Appendix D.

2.4 Groundwater Investigations

Piezometers were installed in BH18-01 and BH18-06 during the drilling investigation and subsequently renamed BH/MW18-01 and BH/MW18-06. The piezometer installed in BH/MW18-01 is screened in the upper bedrock surface, while the piezometer in BH/MW18-06 is screened above the bedrock surface. Both piezometers were constructed using 51 mm diameter PVC pipe with a 3.0 m slotted screen.

Water levels were measured using an electronic water level tape on June 8, 12 and 15, 2018 to determine static groundwater elevations. The elevation of Shirley's Brook was also collected on June 15, 2018 to establish groundwater flow direction and approximate horizontal groundwater flow gradient.

Monitoring well instrumentation details, including static water level elevations, are included on the borehole stratigraphic logs in Appendix D.

2.5 Soil Classification

Soils were classified in the field based on visual and tactile examination by Geofirma technical staff based on accepted methods of classification used in geotechnical engineering practice. Laboratory testing of subsurface units was completed and incorporated into the finalized borehole logs and soil classification. Boundaries between stratigraphic units are generally transitional in nature and subsurface conditions represent the conditions in the borehole only. Conditions between boreholes represent an interpretation of the subsurface geology and should be confirmed during construction activities.

2.6 Surveying

A site topographical survey, referenced to a geodetic datum, was completed by J.D. Barnes Ltd. and provided to Geofirma. A supplemental survey was completed to tie in top of riser elevations for each of the piezometers, as well as to correct the ground surface elevation for boreholes that were adjusted during the drilling program (i.e. after the J.D. Barnes survey).

2.7 Laboratory Testing

Soil samples identified for additional characterization were submitted to the materials testing laboratory at Cambium Inc. in Peterborough, Ontario. Rock core samples were submitted to GEMTEC in Kanata. Both Cambium and GEMTEC labs are certified by the Canadian Council of Independent Laboratories (CCIL) for the completion of standard geotechnical laboratory testing.

In total, the following number of samples were submitted for geotechnical laboratory analysis:

- 6 x sieve/hydrometer
- 5 x atterberg
- 2 x unconfined compressive strength (rock core)

Two soil samples and one groundwater sample were collected and submitted to Paracel Laboratories in Ottawa to measure pH, resistivity, chloride and sulphate, for the purpose of determining the corrosion potential and recommended cement type based on the geochemistry of soil and groundwater.

Results of geotechnical and geochemical laboratory testing are discussed in Section 3. Complete laboratory reports are included in Appendix E.

2.8 Geophysical Testing

A geophysical testing program was completed by Geophysics GPR International Inc., of Longueil, Quebec on June 8, 2018. The geophysical survey was completed to determine the time-averaged shear wave velocity (Vs) in the upper 30 metres in order to determine the appropriate seismic site class as per the National Building Code of Canada and Ontario Building Code requirements.

A copy of the geophysics report is included in Appendix F and outlines the testing methodology and results.

3 SUBSURFACE CONDITIONS

All boreholes were completed to a minimum of spoon refusal, interpreted to be the bedrock surface. In general, site stratigraphy consists of topsoil, underlain by a clay and silt to approximately 5-6 mBGS. A thin (less than 0.5 m), discontinuous layer of till (clayey to sandy silt) was observed in some boreholes across the site overlying bedrock.

3.1 Overburden

A description of the key overburden units are described in the following sub-sections. A summary of the soil geotechnical laboratory testing is presented in Table 2. This includes grain size analysis (sieve/hydrometer), atterberg limits, and associated USCS classification and soil description. Moisture content for each sample is included on the borehole logs.

Complete laboratory reports are included in Appendix E.

Table 2 Summary of Sieve/Hydrometer Laboratory Analysis

Sample ID	Depth (mBGS)	Gravel (%)	Sand (%)	Silt / Clay (%)	LL / PL	USCS Classification - Description
BH18-01-3	1.5 - 2.1	0	1	99	56.5 / 23.6	CH-MH – Clay and Silt
BH18-03-7	4.6 - 5.2	0	0	100	56.4 / 20.0	CH-MH – Clay and Silt
BH18-04-8	5.5 – 5.9	0	2	98	42.6 / 19.4	CL-ML – Clay and Silt
BH18-05-8	5.3 – 5.9	2	13	85	32.2 / 15.6	ML – Clayey Silt some Sand trace Gravel
BH18-08-5	3.0 – 3.7	0	1	99	48.9 / 24.9	CL-ML – Clay and Silt
BH18-08-7	4.7 – 5.3	8	24	68	--	ML – Clayey Sandy Silt trace Gravel

3.1.1 Topsoil

Brown sandy silt topsoil, measuring approximately 0.3 m thick, was encountered across the site.

3.1.2 Clay and Silt

A deposit of clay and silt of low to high plasticity (CL-ML to CH-MH) was encountered across the entire site underlying the topsoil unit. The clay and silt was generally observed to extend to a depth of approximately 5.2 to 6.2 mBGS, with the exception of BH18-08 where the clay and silt was observed to a depth of only 4.8 mBGS.

The clay and silt unit was generally stiff to very stiff in the upper 3.5 to 4.5 m, with SPT N-values ranging from 5 to 13. The lower clay and silt was generally firm to stiff, with N-values ranging from 1 to 4. Water content increased with depth from 30-40% to greater than 40% in the lower grey clay and silt unit.

Field shear vane tests were attempted in the upper firm to stiff clay, but unsuccessful and reported as greater than 120 kPa. Successful vane tests were completed in the lower clay and silt (greater than 4.5 mBGS), where undrained shear strength ranges from 29 to 62 kPa. The ratio of intact to remoulded shear strength (determined through field shear vane testing) is 4.0 to 5.3, indicating a sensitive clay.

3.1.3 Till

A relatively thin layer of till was encountered in BH18-03, BH18-05, BH18-07 and BH18-08. The till can be classified as clayey silt to clayey sandy silt with trace gravel. SPT N-values in the till were reported less than 4; however, this is based on a limited thickness of till prior to encountering bedrock. The till was observed in the field to be soft/loose and wet to saturated.

3.2 Bedrock

A total of 3.7 m of rock core was collected from BH18-01 and 0.91 m from BH18-05 to characterize the upper bedrock surface. Bedrock at the site is described as sandy dolostone, likely belonging to the March Formation. At 8 mBGS (67 mASL), a transition to cream coloured quartz sandstone, possibly of the Nepean Formation, was noted.

Rock quality was observed to be good to excellent, with few natural fractures identified and RQD values greater than 75%. It should be noted that a conservative RQD of 59% was recorded in BH18-01 Core Run 1 (5.64 to 6.25 m). This was a short run with adequate core recovery (TCR) and actual rock quality is likely better. No vertical or sub-vertical fractures were identified.

Unconfined compressive strength (UCS) tests were completed on one representative sample from each cored borehole. Peak UCS values range from 80.60 MPa (sample BH18-05-5.94) to 183.60 MPa (sample BH18-01-5.74).

The depth and elevation to bedrock, or refusal and inferred bedrock, is presented on Figure A.4.

3.3 Groundwater

Groundwater elevations were measured in two wells, BH/MW18-01 and BH/MW18-06, on three occasions to establish static water level conditions. A summary of groundwater elevations are presented in Table 3.

Table 3 Summary of Water Elevations

Location	Water Elevation (mASL) 7-Jun-18	Water Elevation (mASL) 12-Jun-18	Water Elevation (mASL) 15-Jun-18
BH/MW18-01	73.42	73.25	73.20
BH/MW18-06	73.72	73.56	73.53
Shirley's Brook	--	--	71.83

Water levels are considered static and representative of site conditions at the time; however, it should be noted that fluctuations in groundwater elevation will occur seasonally, for example, due to spring freshet or periods of high/low precipitation.

3.3.1 Water Elevation and Interpreted Flow Direction

Groundwater and surface water elevations were measured on June 15, 2018 from the two piezometers and Shirley's Brook to establish approximate flow directions. Water elevations are presented on Figure A.5, Appendix A.

Based on a review of the surface and groundwater elevation data, groundwater flow is interpreted to be northeast, toward the brook.

3.3.2 Hydraulic Conductivity

Falling head slug tests were completed on the two on-site piezometers for the purpose of providing a preliminary estimate of the hydraulic properties of the shallow bedrock at BH/MW18-01 and the lower overburden at BH/MW18-06. The hydraulic conductivity was estimated using the method developed by Hvorslev (1951) to analyze water recovery.

Based on the analysis of water level recovery, the estimated hydraulic conductivity of the upper bedrock (BH/MW18-01) is approximately 2×10^{-5} m/s, while the estimated hydraulic conductivity of the overburden is 4×10^{-7} m/s.

A copy of the slug test results and analysis are presented in Appendix G.

4 GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

The following sections provide an assessment of the site conditions and geotechnical recommendations for design and construction based on the interpreted subsurface conditions gained from the borehole drilling investigation and laboratory testing. It is understood that the proposed development will consist of a mid-rise residential building, with one to two underground parking levels.

The geotechnical assessment is intended to assist design engineers. Any contractors undertaking work at the site should carefully examine the factual information and make their own interpretation of the suitability of the data as it pertains to their specific scope of work and requirements.

4.1 Earthquake Design Parameters

4.1.1 Seismic Hazard Data

The design ground motions based on seismic loads associated for an event with a 2% probability of exceedance in 50 years was established using the on-line seismic hazard calculator found on the Natural Resources Canada website. The peak ground acceleration (PGA) for firm ground conditions (NBCC 2015 Site Class ‘C’) is 0.256 g, where g is the gravitational acceleration.

A copy of the calculator output is provided in Appendix H.

4.1.2 Liquefaction

The clay and silt deposit encountered at the site are generally considered non susceptible to liquefaction based on their water content, liquid limit and plasticity index. It is noted, however, that sample BH18-05-8 is considered moderately susceptible to liquefaction and cyclic mobility.

It is anticipated that the proposed construction will include excavation to bedrock where there are two underground parking levels and that the portion of the building constructed with one level of underground parking will be founded directly on bedrock using piles or caissons.

4.1.3 Seismic Site Class

The shear wave velocity profile for the upper 30 m (V_s30) was determined using the multi-channel analysis of surface waves (MASW) method by geophysics specialists from Geophysics GPR International Inc. A representative survey profile was completed across the site to characterize the seismic velocity of overburden and bedrock. The estimated shear wave velocity of the bedrock is estimated to be greater than 1500 m/s; however, the shear wave velocity of the overburden is considerably lower. As such, seismic site class, which is determined as the average shear wave velocity of 30 m of material underlying the footings is dependent on the final design founding elevations.

Based on the Geophysics GPR report, included in in Appendix F, the recommended site classification, in accordance with Table 4.1.8.4A of the Ontario Building Code, is Site Class B where the depth of excavation results in less than 3 m of soil between the bottom of the footing and bedrock surface. Where the depth of excavation results in more than 3 m of soil between the bottom of the footing and bedrock surface, a Site Class C should be applied. If the buildings are founded on conventional footings directly over sound bedrock, a Site Class A could be used.

If the buildings are founded on deep foundations, a Site Class B or C should be used in accordance with the thickness of soil between the basement slab and the surface of bedrock. A Site Class B could be used where the depth of excavation results in less than 3 m of soil between the bottom of the basement slab and bedrock surface. A Site Class C should be used where the thickness of soil is more than 3 m.

4.2 Lateral Earth Pressure for shoring/foundation wall design

Active earth pressure including static and dynamic (seismic effect) can be included to estimate the lateral earth pressure where the wall will allow lateral yielding. Where the wall does not allow lateral yielding, at rest earth pressure should be used to estimate the lateral earth pressure.

The following Table 4 provides the recommended soil parameters for the design of the retaining wall/shoring.

Table 4 Soil Parameters for Retaining Wall Design

Material Type	Total unit weight KN/m^3	Angle of internal friction φ’	“At rest” earth pressure coefficient (K_O)	Active earth pressure coefficient (K_A)	Combined static and seismic active earth pressure coefficient (K_{AE})
Granular A	23.0	35	0.43	0.27	0.37
Granular B Type II	23.5	32	0.47	0.31	0.41
Sand	20.0	30	0.50	0.33	0.43
Clay	17.0	30	1.00	0.50	0.60
Glacial till	23.0	35	0.43	0.27	0.37

The above values are given considering a horizontal surface at the back of the retaining wall/shoring, a straight wall and a wall friction angle of 0 degrees.

The active pressure at a depth (z) including static and dynamic (seismic effect) acting on the foundation wall for compacted sand or sand and gravel (OPSS Granular B Type I) can be estimated as suggested by the Canadian Foundation Engineering Manual (CFEM, 2006, Equation 24.9):

$$\sigma_z = (1 - K_v) \times K_{AE} \times \gamma \times (H) \qquad (1)$$

Where: σ_z is the lateral active earth pressure [kPa] including static, dynamic:

- H is the height of the wall [m];
- γ is the unit weight of backfill, assume 21 [kN/m^3];
- K_v is the vertical component of the earthquake acceleration [decimal of gravity acceleration g], K_v can be assumed to be 0g (as a conservative value);
- K_{AE} is the earth pressure coefficient for static and dynamic as shown in table 4 above,
- K_h is the horizontal component of the earthquake acceleration [decimal of gravity acceleration g], and can be taken as 50% of the PGA (k_h.= 0.128)

Additional external loads should be considered as well, such as surcharge at ground surface behind the wall accounting for traffic, equipment, or stockpiled soil.

4.3 Foundations

4.3.1 Conventional Strip and Pad Foundations

The boreholes completed at the site encountered silty clay overlying a relatively thin layer of till material directly above inferred bedrock at depths varying between 5.3 and 6.2 mBGS (elevation 69.7 to 71.6 mASL). As mentioned above, it is understood that the proposed footings will be founded around elevation 73 mASL. It is therefore anticipated that the structure will be founded on silty clay.

In order to minimize the potential for differential settlement, it is recommended that the structure be founded entirely over sound bedrock or entirely over a minimum of 0.8 m of overburden materials.

A preliminary analysis of the bearing capacity of the firm to stiff clay indicates that the serviceability limit state (SLS) bearing capacity bearing would be in the range of 90 to 150 kPa. It should be noted that maximum allowable footing widths will have to be restricted in order to limit the anticipated settlement of the buildings. Further geotechnical testing could be carried out at the site, including laboratory consolidation analyses, in order to evaluate the settlement potential of the firm silty clay. However, it should be noted that the bearing capacity and the maximum allowable footing width may not be sufficient to support the building’s loads without resulting in excessive settlements.

A coefficient of friction (μ) of 0.3 could be used in calculating the sliding friction forces between concrete and stiff silty clay.

4.3.2 Strip and pad Fondations on bedrock/Secant wall founded on bedrock

Strip and pad footings, designed with a minimum 1.0 metres in width, founded entirely over relatively sound bedrock may be designed using a maximum allowable bearing pressure of 2 MPa (Ultimate limit state (ULS) bearing resistance with a geotechnical resistance factor of 0.5). There is no corresponding serviceability limit state (SLS) bearing capacity for footings founded entirely over bedrock. All footings should be founded over relatively clean and sound bedrock. Any loose soil/mud/weathered bedrock should be removed from the footprint of the footings. It is recommended that the surface of the bedrock be cleaned by a pressure washer or compressed air prior to pouring the footings. Considering a founding level of about 73 mASL, sub excavations of at least 1.5 to 3.3 meters are anticipated in order to found the footings on bedrock.

Trenches could be excavated below the proposed footing level and backfilled with lean concrete (minimum 12 MPa) up to the footing level. However, this method may be difficult to implement as the surface of the bedrock should be cleaned of any loose/soil/mud/weathered bedrock. The sides of the excavations in overburden materials should therefore be sloped or shored in accordance with the requirements in Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 527/00 as mentioned below, in order to permit worker activity in the trenches. Furthermore, ground water infiltration from the till layer is anticipated and dewatering of the trenches may be required prior to pouring. Dewatering of the trenches may be difficult to carry out considering the low estimated hydraulic conductivity of the till. Trenches should extend at least 300 m on each side of the proposed footings. Dewatering methods may require the use of sump pits and pumps or well points.

A coefficient of friction (μ) of 0.46 could be used in calculating the sliding friction forces between concrete and clean bedrock.

4.3.3 Deep Foundations

As an alternative to lowering the footing level to the elevation of bedrock surface, the structure could be supported on end bearing deep foundations such as piles, caissons, piers, etc, on bedrock or socketed into bedrock. Attention should be given to the distance between the bottom of the proposed foundation and the surface of the bedrock in order to ensure that a suitable pile length is used.

4.3.3.1 Piles

Closed ended, concrete filled steel pipe piles or steel H-piles could be used. All of the piles should be driven to refusal. The refusal criteria will be highly dependent on the contractor's pile driving equipment.

The contractor should be required to submit to the geotechnical engineer a copy of the proposed pile type and driving criteria for review prior to construction. An allowance should be made in the specifications for re-striking all of the piles at least once to confirm the design set and/or the permanence of the refusal and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first re-strike should receive additional re-striking until the design set criteria is met. All re-striking should be performed after 48 hours of the previous set. Furthermore, the specifications should make provision for dynamic testing of selected piles by the engineer to verify the transferred energy and pile capacities.

It should be noted that a stepped, near horizontal bedrock surface should not pose any significant problems for piling. However, steeply sloping bedrock surfaces could pose some difficulties during construction due to the potential for sliding of the tip of the pile along the bedrock surface during pile driving and could require the use of rock injector tips for the piles.

The post construction settlement of elements of the structure, which derive their support from properly terminated piles, should be negligible.

4.3.3.2 Caissons/Piers

Deep foundations founded on bedrock may be designed using a maximum allowable bearing pressure of 2 MPa (Ultimate limit state (ULS) bearing resistance with a geotechnical resistance factor of 0.5). There is no corresponding serviceability limit state (SLS) bearing capacity for footings founded entirely over bedrock. In order to facilitate inspections and cleaning of the pier subgrade, the size of caissons should be at least 900 mm in diameter. Construction difficulties associates with cobbles/boulders, groundwater seepage and sloughing conditions should be expected during installation of caissons.

Temporary casings should be used to prevent soil sloughing into the caisson and pumps should be available to dewater the caissons prior to concrete placement. If excessive groundwater flows are encountered, concrete for the caissons should be placed using tremie procedures.

4.3.4 Resistance to Lateral Loading

Lateral loading could be resisted by the soil resistance in front of the piles, using battered piles or by the use of rock-socketed caissons. The SLS geotechnical response of the soil in front of the piles under lateral loading can be calculated using linear behavior (e.g., theory of subgrade reaction) where maximum pile deflections are small (less than 1 % of the pile diameter), where the loading is static (no cycling) and where the pile material is linear (e.g., steel). If one or more of these conditions are not met, methods that can model the pile and soil non-linearity should be used such as non-linear resistance displacement relationships (p-y curves). The nonlinear lateral displacement of the piles could be estimated using section 18.5.1 of the Canadian Manual of Foundation Engineering 4th edition (2006) or commercially available software programs such as LPILE or FLPIER. The geotechnical parameters for the non-linear resistance displacement method are provided in Table 5, below.

4.3.4.1 Horizontal Coefficient of subgrade reaction

The modulus of subgrade reaction is a difficult parameter to evaluate properly because it is not a unique fundamental property that is readily measured. Its value depends on several factors including the size and shape of the foundations, the type of soil, the relative stiffness of the foundation and soil, etc. The technical literature cites typical values for the vertical modulus of subgrade reaction, k_{v1} (for a one-foot square plate). The coefficient of horizontal subgrade reaction, k_h, can be estimated based on the vertical modulus of subgrade reaction (k_{v1}), as described by Terzaghi (1955). Typical ranges in k_{v1} are summarized in Table 7.1 of the Canadian Manual of Foundation Engineering 4th edition (2006). The recommended horizontal modulus of reaction for 1 m diameter pile or a wall of 1m unit width(k_{h1}) are provided below in Table 5.

For a pile/pier diameter of B (in meters) or a wall width of B, the horizontal subgrade reaction, k_{hb}, for actual pile/pier diameter and wall width is

$$k_{hb}= \frac{1}{(3.28\,B)}k_{h1}$$

where: k_{h1}= coefficient of horizontal subgrade reaction for a 1 m diameter pile/pier or a wall of 1 m unit width, provided below in Table 5.

B = the pile diameter / wall width (in meters)

The recommended geotechnical parameters for the design of resistance to lateral loading are provided in Table 5 below.

Table 5 Geotechnical parameters for resistance to lateral loading

Elevation (m)	Soil type	Horizontal modulus of reaction for 1 m diameter pile or wall of 1m unit width (k_{h1}) MPa/m	Undrained Shear Strength (Cu) kPa	Unit weight (γ) kN/m^3
Pile cap to elevation 72.00 m	Very stiff to stiff clay	20 - 53	100	17
Elevation 72.00 m to bedrock	Firm to stiff clay	6 - 20	40	16

It is noted that the bottom of very stiff to stiff clay elevation varies from about 70,1 to 72,0 meters. The more critical elevation should be considered.

The ULS geotechnical resistance to lateral loading may be calculated using Brom's method as outlined in section 18.4.1 of the Canadian Manual of Foundation Engineering 4th edition (2006). The recommended undrained shear strength values are provided in the above Table 5.

4.4 Underground Parking Slab on Grade

As stated above, it is expected that the proposed building will be founded in undisturbed native silty clay. For predictable performance of the proposed concrete floor slab all soft/loose and any deleterious material should be removed within the proposed building area. The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any soft areas evident should be subexcavated and replaced with suitable engineered fill.

Engineered fill materials provided to support the concrete floor slab should consist of sand, or sand and gravel meeting the OPSS grading requirements for Granular B Type I or crushed stone meeting OPSS grading requirements for Granular B Type II. A minimum 150 millimetre thickness of crushed stone meeting OPSS Granular A should be provided immediately beneath the concrete floor slab. The engineered fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. Depending on the thickness of engineered fill required, suitable lightweight fill material may have to be used for the engineered fill.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage and expansion of the concrete. The saw cut depth should be about one quarter of the thickness of the slab. The crack control cuts should be placed at a grid spacing not exceeding about 5 meters.

If the underground parking will be unheated, the floor slab will require protection from frost effects. Details for suitable frost protection using rigid insulation can be provided, if required.

4.4.1 Vertical modulus of subgrade reaction

The modulus of subgrade reaction is a difficult parameter to evaluate properly because it is not a unique fundamental property that is readily measured. Its value depends on several factors including the size and shape of the footings (raft), the type of soil, relative stiffness of the foundation and soil, etc.

The value of the vertical soil reaction modulus can also vary from one point to another (center, edge or corner). Because the modulus value can change with size of footing, a one foot (300 mm) square footing has been adopted as the standard basis for comparison purposes, and frequently serves as the starting point for design.

The technical literature cites typical values for the vertical modulus of subgrade reaction, k_{v1} (for a one-foot square plate). Typical ranges in k_{v1} are summarized in Table 7.1 of the Canadian Manual of Foundation Engineering 4th edition (2006). Based on the typical values of the vertical modulus of subgrade reaction provided in the CMFE, a recommended value between k_{v1} = 10 and 30 MPa/m could be used for the design of the slab one grade founded at an elevation between 73 and 75 mASL.

The soil vertical modulus of subgrade reaction can also be estimated from the bearing capacity according to Bowles (1996):

$$k_{v1} = 40\ (\mathrm{SF})qa\ (in\ kPa/m)$$

where SF = safety factor

qa = the bearing capacity (in kPa)

The estimated vertical modulus of subgrade reaction as per Bowles ranges between 12 and 27 MPa/m.

If the loaded area on cohesive soil is of width B (in meters) and length mB, the vertical modulus of subgrade reaction for actual footing dimension B (k_{vb}) can be estimated with:

$$k_{vb} = \frac{k_{v1}}{3.28\ B}\left[\frac{m + 0.5}{1.5m}\right]$$

Where k_{v1} = vertical modulus reaction for a one-foot square plate

B = Foundation width (in meters)

4.5 Frost Considerations

The design frost depth at the Site is 1.8 m (OPSD 3090.101), therefore all exterior footings should be protected by a minimum of 1.8 m of soil cover or an equivalent combination of soil thickness and insulation. If construction is completed during winter conditions, temporary frost protection should be provided.

4.6 Excavation

The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 527/00. The soil encountered at the site, can be classified as Type 3. That is, open cut excavations deeper than 1.2 metres within overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical starting at the base of the excavations, or flatter.

The listed slopes are for fully drained excavations. Much gentler slopes could be required under undrained conditions, where local water infiltrations occur and where the excavations are exposed for prolonged periods of time. Any excavated material stockpiled near an excavation or trench should be stored at a distance equal to or greater than the depth of the excavation/trench and construction equipment traffic should be limited near open excavation.

If the aforementioned slopes are not possible or practical to achieve due to space restrictions or obstacles, the excavation should be shored according to OHSA Reg. 213/91. A professional engineer should design, approve and supervise the shoring and establish the shoring depth under the excavation profile. The excavation for the underground services could be carried out within tightly fitting, braced steel trench boxes, approved by a professional engineer.

Groundwater inflow from the native soils into the excavations during construction, if any, should be handled by pumping from sumps within the excavations. Surface water runoff into the excavation should be minimized and diverted away from the excavation.

4.6.1 Excavation in Bedrock

It is assumed that the weathered portion of the bedrock may be excavated using a large excavator and that the sound bedrock may require the use of line drilling and blasting techniques or a high-energy hoe-ram.

The slopes of the rock excavation may be vertical with a 1m wide bench at the soil-rock interface on all sides of the excavation. Any loose pieces of rock from the sidewalls of the excavation should be removed and the bottom of the excavation should be sufficiently flattened and exempt of rock ledges.

A condition survey of any nearby structures and services should be undertaken prior to any blasting or hoe ramming operations. The blasting should be carried out under the supervision of a vibration specialist engineer to ensure that the limiting vibration criteria, established by the vibration specialist engineer, are not exceeded.

4.7 Groundwater Control

4.7.1 Inflow during Construction

For design purposes, groundwater elevation in the overburden can assumed to be at 74 mASL, although water levels will fluctuate seasonally. It is anticipated that the proposed below grade level(s) will be below the groundwater level and some degree of inflow should be expected during construction below the anticipated static groundwater level.

Significant inflow is possible when excavating elevator or sump pits in the upper bedrock. Evidence of groundwater flow through fractures was identified during drilling (iron stained joints) and water recovery of BH/MW18-01 installed in the upper bedrock was rapid.

Groundwater seepage and infiltration into the excavation should be managed by pumping from sumps in the excavation. Surface water runoff should be diverted from the excavations where possible to minimize additional dewatering requirements.

4.7.2 Foundation Drainage

Considering the proposed building will have an underground parking level below the interpreted groundwater level, permanent perimeter drainage and under slab drainage is recommended. Perimeter Perimeter drainage pipe shall be embedded in a 300 mm layer of clear crushed stone, wrapped in a geotextile and located adjacent to the perimeter footings and in a parallel row, spaced 5 meters apart below the slab.

The drainage pipes should be positively connected to a water drainage system such as a dry well, a drainage ditch or a storm drain. The drainage system under the slab and the peripheral drainage system should be connected separately in case a system fails.

Drainage from building roofs should be controlled and exterior grades sloped away from the buildings to prevent ponding of water adjacent to the foundation walls. Drainage should not be directed over the slope as this may increase erosion and lead to future stability issues.

4.8 Site Services

4.8.1 Pipe Bedding and Cover Material

It is suggested that the service pipe bedding material consist of at least 150 mm of granular material meeting OPSS requirements for Granular A. Granular material meeting OPSS specifications for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as bedding or sub-bedding material is not recommended.

Cover material, from pipe spring line to at least 300 mm above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The sub-bedding, bedding and cover materials should be compacted in maximum 200 mm thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

4.8.2 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Some of the native material from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or re-compaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future roadway areas. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, sidewalks, etc., the trench backfill should be compacted in maximum 300 mm thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced where the trench backfill is not located within or in close proximity to existing or future roadways, driveways, sidewalks, or any other type of permanent structure.

4.8.3 Reuse of On-site Soils

The existing overburden materials at the site consist of mainly silty clay. The silty clay is considered to be frost susceptible and should not be used as backfill material directly against foundation walls. Excavated bedrock should not be used as backfill material unless it meets the physical properties and gradation requirements of OPSS Granular B – Type I, or equivalent.

It should be noted that the adequacy of a material for reuse as backfill will mainly depend on the water content of the material at the time of use and on the weather conditions at that time. Any excavated materials proposed for reuse should be stockpiled in a manner to promote drying and should be inspected and approved for reuse by a geotechnical engineer.

4.9 Corrosion Potential and Cement Type

Two representative soil samples and one sample of groundwater from the upper bedrock were collected and analyzed for a suite of parameters to assess corrosion potential of buried concrete and steel. Testing results are summarized in Table 5.

Table 6 Analytical Results – Corrosion Potential

Sample ID	Media	Depth (Elevation)	pH	Resistivity (ohm.cm)	Chloride (µg/g or mg/L)	Sulphate (µg/g or mg/L)
BH18-04-7	Soil	4.7-5.3 (71.9-71.3)	7.75	816	783	175
BH18-07-5	Soil	3.0-3.6 (74.0-73.4)	7.64	3560	143	41
BH/MW18-01	Groundwater	5.6-9.3 (69.3-65.6)	7.7	1420	115	41

The values of pH, resistivity, chloride and sulphate are indicators of the potential corrosiveness of the subsurface to unprotected steel.

The pH is in the low basic range and does not particularly increase soil corrosivity. The sulphate concentrations in the soils are below critical levels. The resistivity of samples BH18-04-7 and BH/MW18-01 are indicative of a severely corrosive to corrosive soil/groundwater and the resistivity of sample BH18-07-5 is indicative of a corrosive to moderately corrosive soil. The chloride content of BH18-04-7 is also indicative of a corrosive soil environment.

It is therefore recommended that corrosion mitigation be considered for exposed structural elements.

It should be noted that the corrosion potential of native soil/groundwater could be influenced by the application of de-icing salt (sodium chloride).

The concentration of sulphate provides an indication of the potential for sulphate attack on concrete that is in contact with groundwater or soil. The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete. Up to 0.10 percent in soil and up to 150 ppm in groundwater, the potential is negligible. From 0.10 to 0.20 percent in soil and from 150 to 1000 ppm in groundwater, the potential is mild but positive. From 0.20 to 0.50 percent in soil and from 1000 to 2000 ppm in groundwater the potential is considerable and over 0.50 percent in soil and over 2000 ppm in groundwater the potential is severe.

Based on NRC guidelines, the above mentioned samples are considered to have a negligible potential for sulphate attack of buried concrete. Therefore general use (GU) cement is appropriate for concrete in contact with native soil or groundwater.

4.10 Pavement Design Recommendations

All existing topsoil, vegetation, and/or organic soils must be removed from the proposed pavement area (including parking, driveway, light and heavy traffic zones) to the elevation of the design subgrade line elevation. The slope of the excavation should be no steeper than 5H:1V within 1.2 m of finished grade to minimize the effect of differential frost heave. The exposed design subgrade line elevation should be inspected by experienced geotechnical personnel, and any soft soil or organic soil should be sub-excavated below the design subgrade line elevation and replaced with compacted subgrade fill consistent with the requirements of OPSS Select Subgrade Material (SSM). Geofirma Engineering Ltd should be contacted if the soft soil/organic soils extend beyond 500 mm below the subgrade line elevation. Fill material should be tested and approved by experienced geotechnical personnel prior delivery to the site. Subgrade fill should be placed in lifts no thicker than 300 mm then compacted to 95% SPMDD. The exposed subgrades should be surface compacted with a large vibratory roller and inspected by experienced geotechnical personnel.

The design of the pavement structure depends on the anticipated traffic volume and types of vehicles. The suggested minimum pavement designs are shown in Table 6.

Table 7 Suggested Minimum Pavement Designs

Emergency Routes	Parking Pavement
40 mm HL-3	50 mm HL-3
60 mm HL-8 (HS)	-
150 mm Granular A	150 mm Granular A
450 mm Granular B Type II	300 mm Granular B Type II

The performance of the pavement structure is highly dependent on the subsurface groundwater conditions. To intercept excess subsurface water within the pavement structure granular materials, sub-drains with suitable outlets could be installed below the pavement area's subgrade. The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features.

Infrastructure preparation works should carried out in such a way that a granular cover is put in place on the exposed subgrade line as quickly as possible in order to avoid the movement of heavy equipment on the subgrade material. In addition, exposed surfaces should be protected against frost if work is done in winter.

4.10.1 Pavement structure over concrete roof

It is understood that a portion of the pavement structure will be placed over a layer of rigid insulation over a concrete roof. It is recommended that the distance between the top of the pavement structure and the top of the rigid insulation be at least 450 mm, if the rigid insulation consists of an extruded polystyrene XPS (CAN/ULC AS701) of a compressive strength of 690 kPa (100 PSI). An XPS with a compressive strength of 415 kPa (60 PSI) could be used if that distance is increased to 600 mm and an XPS with a compressive strength of 275 kPa (40 PSI) could be used if that distance is increased to 750 mm.

Any areas where the pavement subgrade transitions from overburden materials to the concrete roof should be provided with a suitable granular frost taper consisting of excavating to a depth of at least 1.8 metres at the concrete roof face and tapering the excavation upwards at 5 horizontal to 1 vertical towards the pavement subgrade level. The excavation should then be filled with non-frost susceptible material such OPSS Granular B Type I or Type II compacted to 95 percent of the standard Proctor maximum dry density for the material used. The transition of the granular A layer could be carried out at a 3 horizontal to 1 vertical profile.

4.11 Grade Raise Restrictions

The site is underlain by a deposit of Silty Clay overlying till and bedrock. The silty clay layer is very stiff to stiff down to a depth of about 4.5 m where the clay becomes firm to stiff. Considering that the proposed structure will be founded on bedrock, the grade raise will have no impact on the structure. However, it should be noted that an excessive grade raise could result in the settlement of the ground surrounding the proposed structure and its paved areas. This settlement could result in cracking and/or unevenness of the asphaltic concrete and require maintenance in the form of, but not limited to, overlays and/or padding.

To limit settlements to 25 mm, a maximum grade raise limit of 1.8 metres above existing site grades could be used for this site. This grade raise restriction assumes that the fill material will have a maximum unit weight of 22 kN/m3 (e.g. OPSS Granular B Type II).

If a greater grade raise is required, additional geotechnical testing including consolidation analyses could be carried out in order to better evaluate the potential for settlement.

4.12 Slope Stability

The current slope conditions are described in the Slope Inspection Report, completed on May 18, 2018 and attached as Appendix B. Based on the inspection and rating of stability components following the guidelines prescribed by the Ministry of Natural Resources (1998), the slope rating is 22, which indicates low potential for slope instability. The following section describes development issues at the site as they relate to influencing slope stability.

4.12.1 Soil Stockpiling

The stockpiling of excess soil during excavation should be minimized and should not be placed at or near the slope crest to prevent loading of the slope.

4.12.2 Foundation Loading

It is anticipated that the proposed building will be founded directly on bedrock, where there are two underground parking levels (northern portion of the building), or indirectly on bedrock through piles. No additional foundation loading will occur as a result of site development.

4.12.3 Grade Raise Loading

The site grade will likely be altered as a result of the development and there will likely be a grade raise surrounding the building and the parking lot area. Any grade raise steeper than 5 horizontal to 1 vertical near the crest of the slope should be reviewed by a geotechnical engineer. The maximum grade raise limit of 1.8 m above the existing grade (Section 4.10) in the allowable development area (i.e. beyond the 30 m setback) will not result in steepening of the slope beyond the 5H:1V threshold.

4.12.4 Drainage

Surface runoff during and after construction should be directed to swales. Any drainage works (drainage pipes, etc.) should be directed away from the slope or should extended sufficiently to outlet below the toe of the slope. The drainage outlets should be protected using suitable riprap and underlain by a suitable geotextile.

A geotechnical engineer should review and approve any proposed works carried out within the slope or near the crest of the slope.

4.13 Other Considerations

4.13.1 Excess Soil Management

Excess soil material generated during excavation activities at the site should be managed in accordance with Ontario's Best Management Practices Guide (2014) and Excess Soil Management Policy Framework (MOECC, 2016). At this time the MOECC has released proposed regulations, which are anticipated to be released as a final document sometime in 2018.

Soil samples were not tested for contamination (either natural or human induced) and this report does not constitute a Soil Management Plan.

4.13.2 Abandonment of Piezometers

The two piezometers installed during the field investigations (BH/MW18-01 and BH/MW18-06) should be decommissioned by a MOECC-licensed well technician. Well abandonment can be completed before or during construction activities.

4.13.3 Silt Fencing

Consideration should be given to installation of silt fencing along the 30 m setback line during construction to prevent sediment laden runoff from entering Shirley's Brook.

5 LIMITATIONS AND CLOSURE

This report has been prepared for the exclusive use of 10731845 Canada Inc. for specific application to the proposed project at 788 March Road, in Kanata (Ottawa), Ontario. Data obtained from sampling investigations represent the conditions at the time of sampling and are subject to variability.

Geofirma Engineering Ltd. (Geofirma) has completed the study in accordance with generally accepted geotechnical engineering practice. Geofirma has exercised professional judgment in collecting and analyzing the information and in formulating recommendations based on the results for the guidance of the designers and is intended for this project only. The mandate at Geofirma is to perform the given tasks within guidelines prescribed by the client and with the quality and due diligence expected within the profession. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. No other warranty or representation expressed or implied, as to the accuracy of the information or recommendations is included or intended in this report.

Geofirma Engineering Ltd. hereby disclaims any liability or responsibility to any person or party, other than the party to whom this report is addressed, for any loss, damage, expense, fines or penalties which may arise or result from the use of any information or recommendations contained in this report by any other party. Any use of this report constitutes acceptance of the limits of Geofirma's liability. Geofirma's liability extends only to its client and only for the total amount of fees received from the client for this specific project and not to other parties who may obtain this report.

Respectfully submitted,

Geofirma Engineering Ltd.

LICENSED PROFESSIONAL ENGINEER
21 Dec 2018
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APPENDIX B

Slope Inspection Report

Geofirma Engineering Ltd. - SLOPE INSPECTION REPORT

Site Location: 788 March Road, Kanata (Ottawa), Ontario Ref. No.: 18-206-2
Property Owner: Omniplex Real Estate Inc. (c/o Ralph Esposito Jr.)
Inspection Date: May 18, 2018
Weather: Sunny, 12ºC
Inspected by: Steve Gaines, P.Eng.

The following sections outline the results of the site inspection completed at 788 March Road, Kanata (Ottawa) and represent an assessment of the current (i.e. pre-development) slope conditions. The inspection report has been prepared in general accordance with Ontario Ministry of Natural Resources Geotechnical Principles for Stable Slopes (1998).

1 GENERAL SITE OBSERVATIONS

The property is a vacant lot, located at the southeast corner of the intersection of March Road and Klondike Road and bounded by Shirley's Brook to the northwest. The proposed development at the site includes a six-story residential complex with two levels of underground parking to be constructed fronting March and Klondike Roads.

The site is gently sloping northeast toward Shirley's Brook. Historical filling was observed along the bank of the watercourse, creating a secondary, minor slope, and swale running parallel to the brook. Vegetation is well established at the site, consisting of grasses, shrubs and small to medium sized tress. The northern portion of the site more heavily vegetated with mature trees.

A site survey completed by JD Barnes on May 15, 2018 was provided to Geofirma. Based on the survey, the width of Shirley's Brook is approximately 5 m and ranges from 3 to 14 m. The width of the watercourse will vary seasonally, with spring freshet expected to represent the highest flow conditions. Shirley's Brook flows toward the northwest. At the time of the survey and site inspection flow in the brook was minimal.

2 SLOPE STABILITY RATING

2.1 Slope Inclination (Rating = 0)

- Gentle slope from table land to Shirley's Brook. Minor swale mid-slope, likely a result of fill piling near river bank at slope toe. Site slope from top of upper slope to toe is approximately 9h:1v, with the slope near the toe (including fill piles) closer to 3h:1v.
- Maximum inclination is less than 18 degrees. Immediately adjacent to the bank the localized slope may be higher, however the height of the fill piles is not significant (generally less than 2 to 3 m).

2.2 Soil Stratigraphy (Rating = 12; clay, silt)

- Site is mapped by the Ontario Geological Survey as fine-textured glaciomarine deposits, which is supported by historical site investigations at the site.

2.3 Seepage from Slope Face (Rating = 0)

- No seepage was observed from the slope face.

2.4 Slope Height (Rating = 2)

- The elevation of Shirley's Brook is approximately 72 mASL and the elevation of the upper slope crest is 76.5 mASL. Conservatively, the slope height is 2.1 to 5 m.

2.5 Vegetation Cover on Slope Face (Rating = 2)

- The slope face was observed to be primarily vegetated with well-established grass and shrubs, with some trees on the southern portion, and heavily vegetated with heavy shrubs and mature trees along the northern portion of the slope.

2.6 Table Land Drainage (Rating = 0)

- Surface water will drain northeast into Shirley's Brook. Given the gentle slope and well established vegetation it is expected that there will be reasonably high infiltration, minimizing surface runoff. Furthermore, piles of fill near the bank have created a swale, which will likely minimize erosion and direct runoff to the watercourse.
- There was no evidence of erosion due to overland drainage.

2.7 Proximity of Watercourse to Slope Toe (Rating = 6)

- Shirley's Brook is located along the slope toe. At the time of the site inspection the Brook was approximately 5 m wide and 0.3 m deep with minor flow toward the northwest.

2.8 Previous Landslide Activity (Rating = 0)

- There is no evidence of historical landslide activity.

3 RECOMMENDATIONS

The slope rating (sum of all slope stability components in Section 2) is 22. MNR guidance specifies that a slope rating of less than 24 indicates there is low potential for slope instability. The general site observations indicated there was no significant erosion at the slope toe, vegetation is well developed, and that there is no evidence of past instability.

No further investigations are recommended to assess slope stability; however, an assessment and discussion of the impact due to structural loading and grade raise near the slope crest will be addressed in the geotechnical investigation report.

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Revision Number:	0	Date: May 18, 2018
Prepared By:	Steve Gaines, P.Eng.	
Reviewed By:	Benoit Charlebois, P.Eng. (Charlebois Engineering Ltd.)	

APPENDIX C

Site Photographs

Geotechnical Investigation Report
788 March Road, Kanata (Ottawa), Ontario

Revision: 1 (Final)

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Project Number: 18-206-2

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Client:	10731854 Canada Inc.	
Project Number:	18-206-2	
Document ID:	18-206-2_Geotech Report_R1	
Revision Number:	1	Date: July 13, 2018
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Reviewed by:	Sean Sterling, P.Eng.	
Approved by:	Sean Sterling Sean Sterling, P.Eng. Principal	

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1 INTRODUCTION

Geofirma Engineering Ltd. (Geofirma) was retained by Omnipex Real Estate Inc. to complete a geotechnical investigation of the property located at 788 March Road, Kanata, herein referred to as `the site`. The geotechnical assessment has been completed to support development of the site.

The report presents the factual results of the geotechnical investigation and provides geotechnical recommendations for the proposed development of the site. Work has been completed in accordance with the *Geotechnical Investigation and Reporting Guidelines for Development Applications in the City of Ottawa* and the Canadian Foundation Engineering Manual, 4th Ed. (2006).

1.1 Project and Site Description

The site is currently a vacant parcel of land located at the southeast corner of March Road and Klondike Road, and bordered by Shirley's Brook to the northeast. The total area of the parcel is approximately 1.2 hectares (~3 acres). Land use surrounding the site is primarily low-rise commercial, with residential subdivisions located south of the site.

Site topography is moderately sloping downward to the northeast (toward Shirley's Brook). Elevation of the site adjacent to March Road is approximately 78 mASL, with the elevation at Shirley's Brook of approximately 72 mASL. The site is well vegetated with established grass and shrubs on the southern portion the site and mature trees on the north portion of the site.

It is understood that the proposed development of the site will include a multi-level residential complex, with two levels of underground parking.

The location of the site is shown on Figure A.1, the proposed building footprint is illustrated on Figure A.2, and site topography is presented on Figure A.3, Appendix A.

1.2 Regional Geological Setting

The site is located within the Clay Plains physiographic region (Chapman and Putnam, 1984) and consists of fine-texture glaciomarine deposits, including silt and clay with minor sand and gravel (OGS, 2011). Bedrock at the site is mapped as dolostone and sandstone of the Beekmantown Group (OGS, 2011).

1.3 Background

There are no available geotechnical reports for the site; however, a review of the Ontario Geotechnical Borehole Database was completed prior commencing field activities to provide some indication on site specific stratigraphic conditions. Three boreholes are mapped within 200 m of the site: ID 609813, 609814 and 609816 with bedrock depths of 6.4 m, 5.5 m and 9.1 m, respectively.

1.4 Purpose and Scope

The purpose of the geotechnical investigation at the site was to characterize the engineering properties of the subsurface materials, and provide recommendations for site development based on the soil, rock and groundwater conditions.

The scope of work for the geotechnical investigation included the following:

- Review of existing background geological information, primarily data from Ontario Geotechnical Borehole Database.
- Completion of a borehole drilling program to provide spatial coverage and to characterize material properties of the key stratigraphic units.
- Completion of a geotechnical laboratory testing program to characterize the engineering properties of site soil and bedrock.
- Collection of representative soil and groundwater samples to characterize corrosion potential and recommend cement type; and
- Preparation of a geotechnical report providing a summary of field observations and subsurface conditions, laboratory results and geotechnical recommendations.

1.5 Report Organization

The Geotechnical Report is organized into six Sections and eight Appendices. Section 1 provides an overview of the site, and the purpose and scope of the project. The investigation methodology is outlined in Section 2. Section 3 presents a summary of the factual site data collected during the geotechnical investigation. Section 4 provides an assessment of the geotechnical properties of the subsurface materials and geotechnical recommendations. Limitations of this report are outlined in Section 5 and references are included in Section 6.

Appendices include report figures and borehole logs, as well as supporting laboratory results, hydraulic testing results and a copy of the seismic information for the site.

2 METHODOLOGY

Project activities were conducted by Geofirma between May 18 and June 15, 2018. This included the following:

- Slope stability assessment (site inspection)
- Underground utility locates
- Borehole drilling
- Piezometer installation and collection of groundwater elevations
- Surveying
- Geotechnical laboratory analysis; and
- Geochemical laboratory analysis.

2.1 Slope Stability Assessment

A site inspection was completed by Steve Gaines, M.A.Sc., P.Eng., on May 18, 2018 to visually inspect the slope along Shirley's Brook. Specifically, the site inspection was completed to observe current site conditions, slope characteristics, vegetative cover, as well as drainage and water course characteristics.

A preliminary assessment of the slope was completed in accordance with the guidelines provided in the Ontario Ministry of Natural Resources (MNR) Geotechnical Principles for Stable Slopes (MNR, 1998). Based on the observed site features, a slope rating of 22 was assigned and it was concluded that there was no significant toe erosion at the slope toe, the vegetation is well established, and there is no evidence of past instability. A copy of the preliminary inspection report is included in Appendix B. Site photographs collected during the inspection are included as Appendix C.

Based on MNR guidelines, a rating of less than 24 requires no further investigation; however, given the proposed development, an assessment of the post-development conditions (i.e. structural loading and grade-raises) is evaluated and discussed in Section 4.11.

2.2 Underground Utility Locates

Ontario OneCall was contacted to identify the location of all underground buried utilities at the site. Utilities including telephone, gas, hydro, cable/fiber and municipal services were cleared through these services.

2.3 Borehole Drilling

A total of eight (8) boreholes were drilled using a CME-75 track-mounted drilling rig, operated by Aardvark Drilling Inc. (ADI) of Carleton Place, Ontario. Drilling was completed between June 5 and 7, 2018. Boreholes were identified sequentially from BH18-01 to BH18-08 and presented on Figure A.2, Appendix A.

BH18-01 and BH18-02 were advanced using 203 mm hollow stem augers, while advancing a 51 mm diameter split spoon sampler. The remaining boreholes were advanced using a 152 mm diameter solid stem auger, also advancing a 51 mm diameter split spoon. Standard Penetration Tests (SPT) N-values were recorded by the supervising Geofirma technician. Field shear vane tests were collected, where appropriate, using a standard N-sized vane.

Table 1 summarizes the borehole drilling program, including ground surface elevation, depth to bedrock, and total drilled depth.

Table 1 Summary of Bedrock and Total Borehole Depth and Elevations

Borehole ID	Ground Surface Elevation (mASL)	Interpreted Bedrock Depth (Elevation)	Borehole Depth (Elevation)
BH18-01	74.86	5.6 (69.7)	9.3 (66.0)
BH18-02	77.07	6.2 (70.5)	6.2 (70.5)
BH18-03	77.26	6.2 (71.0)	6.2 (71.0)
BH18-04	76.57	5.9 (70.3)	5.9 (70.3)
BH18-05	77.06	5.6 (71.4)	6.6 (70.5)
BH18-06	76.46	5.6 (70.8)	5.6 (70.8)
BH18-07	77.03	5.5 (71.6)	5.5 (71.6)
BH18-08	76.35	5.3 (70.7)	5.3 (70.7)

Note: depth is referenced in metres below ground surface (mBGS) and elevation as metres above sea level (mASL)

Soil samples were collected from each borehole in 0.6 m intervals, and are identified as BH18-XX-Y, where XX is the borehole identifier and Y is the sequential sample interval. For example, BH18-02-3 indicates the third sample from borehole number 2. Samples were inspected in the field by an experienced field technician and collected in plastic freezer bags to preserve moisture conditions. Samples were brought back to the Geofirma office for detailed inspection by a geotechnical engineer. Selected samples were submitted for additional laboratory testing, as discussed in Section 2.7.

Complete borehole logs are included in Appendix D.

2.4 Groundwater Investigations

Piezometers were installed in BH18-01 and BH18-06 during the drilling investigation and subsequently renamed BH/MW18-01 and BH/MW18-06. The piezometer installed in BH/MW18-01 is screened in the upper bedrock surface, while the piezometer in BH/MW18-06 is screened above the bedrock surface. Both piezometers were constructed using 51 mm diameter PVC pipe with a 3.0 m slotted screen.

Water levels were measured using an electronic water level tape on June 8, 12 and 15, 2018 to determine static groundwater elevations. The elevation of Shirley's Brook was also collected on June 15, 2018 to establish groundwater flow direction and approximate horizontal groundwater flow gradient.

Monitoring well instrumentation details, including static water level elevations, are included on the borehole stratigraphic logs in Appendix D.

2.5 Soil Classification

Soils were classified in the field based on visual and tactile examination by Geofirma technical staff based on accepted methods of classification used in geotechnical engineering practice. Laboratory testing of subsurface units was completed and incorporated into the finalized borehole logs and soil classification. Boundaries between stratigraphic units are generally transitional in nature and subsurface conditions represent the conditions in the borehole only. Conditions between boreholes represent an interpretation of the subsurface geology and should be confirmed during construction activities.

2.6 Surveying

A site topographical survey, referenced to a geodetic datum, was completed by J.D. Barnes Ltd. and provided to Geofirma. A supplemental survey was completed to tie in top of riser elevations for each of the piezometers, as well as to correct the ground surface elevation for boreholes that were adjusted during the drilling program (i.e. after the J.D. Barnes survey).

2.7 Laboratory Testing

Soil samples identified for additional characterization were submitted to the materials testing laboratory at Cambium Inc. in Peterborough, Ontario. Rock core samples were submitted to GEMTEC in Kanata. Both Cambium and GEMTEC labs are certified by the Canadian Council of Independent Laboratories (CCIL) for the completion of standard geotechnical laboratory testing.

In total, the following number of samples were submitted for geotechnical laboratory analysis:

- 6 x sieve/hydrometer
- 5 x atterberg
- 2 x unconfined compressive strength (rock core)

Two soil samples and one groundwater sample were collected and submitted to Paracel Laboratories in Ottawa to measure pH, resistivity, chloride and sulphate, for the purpose of determining the corrosion potential and recommended cement type based on the geochemistry of soil and groundwater.

Results of geotechnical and geochemical laboratory testing are discussed in Section 3. Complete laboratory reports are included in Appendix E.

2.8 Geophysical Testing

A geophysical testing program was completed by Geophysics GPR International Inc., of Longueil, Quebec on June 8, 2018. The geophysical survey was completed to determine the time-averaged shear wave velocity (Vs) in the upper 30 metres in order to determine the appropriate seismic site class as per the National Building Code of Canada and Ontario Building Code requirements.

A copy of the geophysics report is included in Appendix F and outlines the testing methodology and results.

3 SUBSURFACE CONDITIONS

All boreholes were completed to a minimum of spoon refusal, interpreted to be the bedrock surface. In general, site stratigraphy consists of topsoil, underlain by a clay and silt to approximately 5-6 mBGS. A thin (less than 0.5 m), discontinuous layer of till (clayey to sandy silt) was observed in some boreholes across the site overlying bedrock.

3.1 Overburden

A description of the key overburden units are described in the following sub-sections. A summary of the soil geotechnical laboratory testing is presented in Table 2. This includes grain size analysis (sieve/hydrometer), atterberg limits, and associated USCS classification and soil description. Moisture content for each sample is included on the borehole logs.

Complete laboratory reports are included in Appendix E.

Table 2 Summary of Sieve/Hydrometer Laboratory Analysis

Sample ID	Depth (mBGS)	Gravel (%)	Sand (%)	Silt / Clay (%)	LL / PL	USCS Classification - Description
BH18-01-3	1.5 - 2.1	0	1	99	56.5 / 23.6	CH-MH – Clay and Silt
BH18-03-7	4.6 - 5.2	0	0	100	56.4 / 20.0	CH-MH – Clay and Silt
BH18-04-8	5.5 – 5.9	0	2	98	42.6 / 19.4	CL-ML – Clay and Silt
BH18-05-8	5.3 – 5.9	2	13	85	32.2 / 15.6	ML – Clayey Silt some Sand trace Gravel
BH18-08-5	3.0 – 3.7	0	1	99	48.9 / 24.9	CL-ML – Clay and Silt
BH18-08-7	4.7 – 5.3	8	24	68	--	ML – Clayey Sandy Silt trace Gravel

3.1.1 Topsoil

Brown sandy silt topsoil, measuring approximately 0.3 m thick, was encountered across the site.

3.1.2 Clay and Silt

A deposit of clay and silt of low to high plasticity (CL-ML to CH-MH) was encountered across the entire site underlying the topsoil unit. The clay and silt was generally observed to extend to a depth of approximately 5.2 to 6.2 mBGS, with the exception of BH18-08 where the clay and silt was observed to a depth of only 4.8 mBGS.

The clay and silt unit was generally stiff to very stiff in the upper 3.5 to 4.5 m, with SPT N-values ranging from 5 to 13. The lower clay and silt was generally firm to stiff, with N-values ranging from 1 to 4. Water content increased with depth from 30-40% to greater than 40% in the lower grey clay and silt unit.

Field shear vane tests were attempted in the upper firm to stiff clay, but unsuccessful and reported as greater than 120 kPa. Successful vane tests were completed in the lower clay and silt (greater than 4.5 mBGS), where undrained shear strength ranges from 29 to 62 kPa. The ratio of intact to remoulded shear strength (determined through field shear vane testing) is 4.0 to 5.3, indicating a sensitive clay.

3.1.3 Till

A relatively thin layer of till was encountered in BH18-03, BH18-05, BH18-07 and BH18-08. The till can be classified as clayey silt to clayey sandy silt with trace gravel. SPT N-values in the till were reported less than 4; however, this is based on a limited thickness of till prior to encountering bedrock. The till was observed in the field to be soft/loose and wet to saturated.

3.2 Bedrock

A total of 3.7 m of rock core was collected from BH18-01 and 0.91 m from BH18-05 to characterize the upper bedrock surface. Bedrock at the site is described as sandy dolostone, likely belonging to the March Formation. At 8 mBGS (67 mASL), a transition to cream coloured quartz sandstone, possibly of the Nepean Formation, was noted.

Rock quality was observed to be good to excellent, with few natural fractures identified and RQD values greater than 75%. It should be noted that a conservative RQD of 59% was recorded in BH18-01 Core Run 1 (5.64 to 6.25 m). This was a short run with adequate core recovery (TCR) and actual rock quality is likely better. No vertical or sub-vertical fractures were identified.

Unconfined compressive strength (UCS) tests were completed on one representative sample from each cored borehole. Peak UCS values range from 80.60 MPa (sample BH18-05-5.94) to 183.60 MPa (sample BH18-01-5.74).

The depth and elevation to bedrock, or refusal and inferred bedrock, is presented on Figure A.4.

3.3 Groundwater

Groundwater elevations were measured in two wells, BH/MW18-01 and BH/MW18-06, on three occasions to establish static water level conditions. A summary of groundwater elevations are presented in Table 3.

Table 3 Summary of Water Elevations

Location	Water Elevation (mASL) 7-Jun-18	Water Elevation (mASL) 12-Jun-18	Water Elevation (mASL) 15-Jun-18
BH/MW18-01	73.42	73.25	73.20
BH/MW18-06	73.72	73.56	73.53
Shirley’s Brook	--	--	71.83

Water levels are considered static and representative of site conditions at the time; however, it should be noted that fluctuations in groundwater elevation will occur seasonally, for example, due to spring freshet or periods of high/low precipitation.

3.3.1 Water Elevation and Interpreted Flow Direction

Groundwater and surface water elevations were measured on June 15, 2018 from the two piezometers and Shirley’s Brook to establish approximate flow directions. Water elevations are presented on Figure A.5, Appendix A.

Based on a review of the surface and groundwater elevation data, groundwater flow is interpreted to be northeast, toward the brook.

3.3.2 Hydraulic Conductivity

Falling head slug tests were completed on the two on-site piezometers for the purpose of providing a preliminary estimate of the hydraulic properties of the shallow bedrock at BH/MW18-01 and the lower overburden at BH/MW18-06. The hydraulic conductivity was estimated using the method developed by Hvorslev (1951) to analyze water recovery.

Based on the analysis of water level recovery, the estimated hydraulic conductivity of the upper bedrock (BH/MW18-01) is approximately 2 x 10^{-5} m/s, while the estimated hydraulic conductivity of the overburden is 4 x 10^{-7} m/s.

A copy of the slug test results and analysis are presented in Appendix G.

4 GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

The following sections provide an assessment of the site conditions and geotechnical recommendations for design and construction based on the interpreted subsurface conditions gained from the borehole drilling investigation and laboratory testing. It is understood that the proposed development will consist of a mid-rise residential building, with one to two underground parking levels.

The geotechnical assessment is intended to assist design engineers. Any contractors undertaking work at the site should carefully examine the factual information and make their own interpretation of the suitability of the data as it pertains to their specific scope of work and requirements.

4.1 Earthquake Design Parameters

4.1.1 Seismic Hazard Data

The design ground motions based on seismic loads associated for an event with a 2% probability of exceedance in 50 years was established using the on-line seismic hazard calculator found on the Natural Resources Canada website. The peak ground acceleration (PGA) for firm ground conditions (NBCC 2015 Site Class ‘C’) is 0.256 g, where g is the gravitational acceleration.

A copy of the calculator output is provided in Appendix H.

4.1.2 Liquefaction

The clay and silt deposit encountered at the site are generally considered non susceptible to liquefaction based on their water content, liquid limit and plasticity index. It is noted, however, that sample BH18-05-8 is considered moderately susceptible to liquefaction and cyclic mobility.

It is anticipated that the proposed construction will include excavation to bedrock where there are two underground parking levels and that the portion of the building constructed with one level of underground parking will be founded directly on bedrock using piles or caissons.

4.1.3 Seismic Site Class

The shear wave velocity profile for the upper 30 m (V_s30) was determined using the multi-channel analysis of surface waves (MASW) method by geophysics specialists from Geophysics GPR International Inc. A representative survey profile was completed across the site to characterize the seismic velocity of overburden and bedrock. The estimated shear wave velocity of the bedrock is estimated to be greater than 1500 m/s; however, the shear wave velocity of the overburden is considerably lower. As such, seismic site class, which is determined as the average shear wave velocity of 30 m of material underlying the footings is dependent on the final design founding elevations.

Based on the Geophysics GPR report, included in in Appendix F, the recommended site classification, in accordance with Table 4.1.8.4A of the Ontario Building Code and **assuming that buildings are founded directly over sound bedrock, or on lean concrete bearing directly on the bedrock surface, is Site Class A**. This would apply to the northern section of the proposed development, where two levels of underground parking are proposed.

Where the depth of excavation results in **less than 3 m of soil between the bottom of the footing and bedrock surface, Site Class B can be applied**.

4.2 Lateral Earth Pressure for shoring/foundation wall design

Active earth pressure including static and dynamic (seismic effect) can be included to estimate the lateral earth pressure where the wall will allow lateral yielding. Where the wall does not allow lateral yielding, at rest earth pressure should be used to estimate the lateral earth pressure.

The following Table 4 provides the recommended soil parameters for the design of the retaining wall/shoring.

Table 4 Soil Parameters for Retaining Wall Design

Material Type	Total unit weight KN/m^3	Angle of internal friction φ'	"At rest" earth pressure coefficient (K_O)	Active earth pressure coefficient (K_A)	Combined static and seismic active earth pressure coefficient (K_{AE})
Granular A	23.0	35	0.43	0.27	0.37
Granular B Type II	23.5	32	0.47	0.31	0.41
Sand	20.0	30	0.50	0.33	0.43
Clay	17.0	30	1.00	0.50	0.60
Glacial till	23.0	35	0.43	0.27	0.37

The above values are given considering a horizontal surface at the back of the retaining wall/shoring, a straight wall and a wall friction angle of 0 degrees.

The active pressure at a depth (z) including static and dynamic (seismic effect) acting on the foundation wall for compacted sand or sand and gravel (OPSS Granular B Type I) can be estimated as suggested by the Canadian Foundation Engineering Manual (CFEM, 2006, Equation 24.9):

$$\sigma_z = (1 - K_v) \times K_{AE} \times \gamma \times (H) \qquad (1)$$

Where: σ_z is the lateral active earth pressure [kPa] including static, dynamic:

- H is the height of the wall [m];
- γ is the unit weight of backfill, assume 21 [kN/m^3];
- K_v is the vertical component of the earthquake acceleration [decimal of gravity acceleration g], K_v can be assumed to be 0g (as a conservative value);
- K_{AE} is the earth pressure coefficient for static and dynamic as shown in table 4 above,
- K_h is the horizontal component of the earthquake acceleration [decimal of gravity acceleration g], and can be taken as 50% of the PGA (k_h.= 0.128)

Additional external loads should be considered as well, such as surcharge at ground surface behind the wall accounting for traffic, equipment, or stockpiled soil.

4.3 Foundations

4.3.1 Conventional Strip and Pad Foundations

The boreholes completed at the site encountered inferred bedrock at depths varying between 5.3 and 6.2 mBGS (elevation 69.7 to 71.6 mASL). As mentioned above, two levels of underground parking are proposed and it is therefore anticipated that the structure will be founded on bedrock.

In order to minimize the potential for differential settlement, it is recommended that the structure be founded entirely over sound bedrock or entirely over a minimum of 0.8 m of overburden materials. .

Strip and pad footings a minimum 1.0 metres in width founded entirely over relatively sound bedrock may be designed using a maximum allowable bearing pressure of 2 MPa (Ultimate limit state (ULS) bearing resistance with a geotechnical resistance factor of 0.5). There is no corresponding serviceability limit state (SLS) bearing capacity for footings founded entirely over bedrock. All footings should be founded over relatively clean and sound bedrock. Any loose soil/mud/weathered bedrock should be removed from the footprint of the footings. It is recommended that the surface of the bedrock be cleaned by a pressure washer or compressed air prior to pouring the footings.

The anticipated total and differential settlement of footings founded on sound bedrock are low (less than 10 mm).

In the event that excavation below the underside of the footing is required, the footings should rest over lean concrete at minimum (min. 12 MPa).

4.3.2 Deep Foundations (Piles)

As an alternative to lowering the footing level to the surface of the bedrock, the structure could be supported on end bearing piles driven to refusal on bedrock. Attention should be given to the distance between the bottom of the proposed foundation and the surface of the bedrock in order to ensure that a suitable pile length is used.

Closed ended, concrete filled steel pipe piles or steel H-piles could be used. All of the piles should be driven to refusal. The refusal criteria will be highly dependent on the contractor's pile driving equipment.

The contractor should be required to submit to the geotechnical engineer a copy of the proposed pile type and driving criteria for review prior to construction. An allowance should be made in the specifications for re-striking all of the piles at least once to confirm the design set and/or the permanence of the refusal and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first re-strike should receive additional re-striking until the design set criteria is met. All re-striking should be performed after 48 hours of the previous set. Furthermore, the specifications should make provision for dynamic testing of selected piles by the engineer to verify the transferred energy and pile capacities.

It should be noted that a stepped, near horizontal bedrock surface should not pose any significant problems for piling. However, steeply sloping bedrock surfaces could pose some difficulties during construction due to the potential for sliding of the tip of the pile along the bedrock surface during pile driving and could require the use of rock injector tips for the piles.

The post construction settlement of elements of the structure, which derive their support from properly terminated piles, should be negligible.

4.4 Frost Considerations

The design frost depth at the Site is 1.8 m (OPSD 3090.101), therefore all exterior footings should be protected by a minimum of 1.8 m of soil cover or an equivalent combination of soil thickness and insulation. If construction is completed during winter conditions, temporary frost protection should be provided.

4.5 Excavation

The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 527/00. The soil encountered at the site, can be classified as Type 3. That is, open cut excavations deeper than 1.2 metres within overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical starting at the base of the excavations, or flatter.

The listed slopes are for fully drained excavations. Much gentler slopes could be required under undrained conditions, where local water infiltrations occur and where the excavations are exposed for prolonged periods of time. Any excavated material stockpiled near an excavation or trench should be stored at a distance equal to or greater than the depth of the excavation/trench and construction equipment traffic should be limited near open excavation.

If the aforementioned slopes are not possible or practical to achieve due to space restrictions or obstacles, the excavation should be shored according to OHSA Reg. 213/91. A professional engineer should design, approve and supervise the shoring and establish the shoring depth under the excavation profile. The excavation for the underground services could be carried out within tightly fitting, braced steel trench boxes, approved by a professional engineer.

Groundwater inflow from the native soils into the excavations during construction, if any, should be handled by pumping from sumps within the excavations. Surface water runoff into the excavation should be minimized and diverted away from the excavation.

4.5.1 Excavation in Bedrock

It is assumed that the weathered portion of the bedrock may be excavated using a large excavator and that the sound bedrock may require the use of line drilling and blasting techniques or a high-energy hoe-ram.

The slopes of the rock excavation may be vertical with a 1m wide bench at the soil-rock interface on all sides of the excavation. Any loose pieces of rock from the sidewalls of the excavation should be removed and the bottom of the excavation should be sufficiently flattened and exempt of rock ledges.

A condition survey of any nearby structures and services should be undertaken prior to any blasting or hoe ramming operations. The blasting should be carried out under the supervision of a vibration specialist engineer to ensure that the limiting vibration criteria, established by the vibration specialist engineer, are not exceeded.

4.6 Groundwater Control

4.6.1 Inflow during Construction

For design purposes, groundwater elevation in the overburden can assumed to be at 74 mASL, although water levels will fluctuate seasonally. It is anticipated that the proposed below grade level(s) will be below the groundwater level and some degree of inflow should be expected during construction below the anticipated static groundwater level.

Significant inflow is possible when excavating elevator or sump pits in the upper bedrock. Evidence of groundwater flow through fractures was identified during drilling (iron stained joints) and water recovery of BH/MW18-01 installed in the upper bedrock was rapid.

Groundwater seepage and infiltration into the excavation should be managed by pumping from sumps in the excavation. Surface water runoff should be diverted from the excavations where possible to minimize additional dewatering requirements.

4.6.2 Foundation Drainage

Considering the proposed building will have underground parking level(s) below the interpreted groundwater level, permanent perimeter drainage will be required. Perimeter drainage pipe shall be embedded in a 300 mm layer of clear crushed stone, wrapped in a geotextile and located adjacent to the perimeter footings.

Drainage from building roofs should be controlled and exterior grades sloped away from the buildings to prevent ponding of water adjacent to the foundation walls. Drainage should not be directed over the slope as this may increase erosion and lead to future stability issues.

4.7 Site Services

4.7.1 Pipe Bedding and Cover Material

It is suggested that the service pipe bedding material consist of at least 150 mm of granular material meeting OPSS requirements for Granular A. Granular material meeting OPSS specifications for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as bedding or sub-bedding material is not recommended.

Cover material, from pipe spring line to at least 300 mm above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The sub-bedding, bedding and cover materials should be compacted in maximum 200 mm thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

4.7.2 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Some of the native material from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or re-compaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future roadway areas. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, sidewalks, etc., the trench backfill should be compacted in maximum 300 mm thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced where the trench backfill is not located within or in close proximity to existing or future roadways, driveways, sidewalks, or any other type of permanent structure.

4.7.3 Reuse of On-site Soils

The existing overburden materials at the site consist of mainly silty clay. The silty clay is considered to be frost susceptible and should not be used as backfill material directly against foundation walls. Excavated bedrock should not be used as backfill material unless it meets the physical properties and gradation requirements of OPSS Granular B – Type I, or equivalent.

It should be noted that the adequacy of a material for reuse as backfill will mainly depend on the water content of the material at the time of use and on the weather conditions at that time. Any excavated materials proposed for reuse should be stockpiled in a manner to promote drying and should be inspected and approved for reuse by a geotechnical engineer.

4.8 Corrosion Potential and Cement Type

Two representative soil samples and one sample of groundwater from the upper bedrock were collected and analyzed for a suite of parameters to assess corrosion potential of buried concrete and steel. Testing results are summarized in Table 5.

Table 5 Analytical Results – Corrosion Potential

Sample ID	Media	Depth (Elevation)	pH	Resistivity (ohm.cm)	Chloride (µg/g or mg/L)	Sulphate (µg/g or mg/L)
BH18-04-7	Soil	4.7-5.3 (71.9-71.3)	7.75	816	783	175
BH18-07-5	Soil	3.0-3.6 (74.0-73.4)	7.64	3560	143	41
BH/MW18-01	Groundwater	5.6-9.3 (69.3-65.6)	7.7	1420	115	41

The values of pH, resistivity, chloride and sulphate are indicators of the potential corrosiveness of the subsurface to unprotected steel.

The pH is in the low basic range and does not particularly increase soil corrosivity. The sulphate concentrations in the soils are below critical levels. The resistivity of samples BH18-04-7 and BH/MW18-01 are indicative of a severely corrosive to corrosive soil/groundwater and the resistivity of sample BH18-07-5 is indicative of a corrosive to moderately corrosive soil. The chloride content of BH18-04-7 is also indicative of a corrosive soil environment.

It is therefore recommended that corrosion mitigation be considered for exposed structural elements.

It should be noted that the corrosion potential of native soil/groundwater could be influenced by the application of de-icing salt (sodium chloride).

The concentration of sulphate provides an indication of the potential for sulphate attack on concrete that is in contact with groundwater or soil. The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete. Up to 0.10 percent in soil and up to 150 ppm in groundwater, the potential is negligible. From 0.10 to 0.20 percent in soil and from 150 to 1000 ppm in groundwater, the potential is mild but positive. From 0.20 to 0.50 percent in soil and from 1000 to 2000 ppm in groundwater the potential is considerable and over 0.50 percent in soil and over 2000 ppm in groundwater the potential is severe.

Based on NRC guidelines, the above mentioned samples are considered to have a negligible potential for sulphate attack of buried concrete. Therefore general use (GU) cement is appropriate for concrete in contact with native soil or groundwater.

4.9 Pavement Design Recommendations

All existing topsoil, vegetation, and/or organic soils must be removed from the proposed pavement area (including parking, driveway, light and heavy traffic zones) to the elevation of the design subgrade line elevation. The slope of the excavation should be no steeper than 5H:1V within 1.2 m of finished grade to minimize the effect of differential frost heave. The exposed design subgrade line elevation should be inspected by experienced geotechnical personnel, and any soft soil or organic soil should be sub-excavated below the design subgrade line elevation and replaced with compacted subgrade fill consistent with the requirements of OPSS Select Subgrade Material (SSM). Geofirma Engineering Ltd should be contacted if the soft soil/organic soils extend beyond 500 mm below the subgrade line elevation. Fill material should be tested and approved by experienced geotechnical personnel prior delivery to the site. Subgrade fill should be placed in lifts no thicker than 300 mm then compacted to 95% SPMDD. The exposed subgrades should be surface compacted with a large vibratory roller and inspected by experienced geotechnical personnel.

The design of the pavement structure depends on the anticipated traffic volume and types of vehicles. The suggested minimum pavement designs are shown in Table 6.

Table 6 Suggested Minimum Pavement Designs

Emergency Routes	Parking Pavement
40 mm HL-3	50 mm HL-3
60 mm HL-8 (HS)	-
150 mm Granular A	150 mm Granular A
450 mm Granular B Type II	300 mm Granular B Type II

The performance of the pavement structure is highly dependent on the subsurface groundwater conditions. To intercept excess subsurface water within the pavement structure granular materials, sub-drains with suitable outlets could be installed below the pavement area’s subgrade. The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features.

Infrastructure preparation works should carried out in such a way that a granular cover is put in place on the exposed subgrade line as quickly as possible in order to avoid the movement of heavy equipment on the subgrade material. In addition, exposed surfaces should be protected against frost if work is done in winter.

4.9.1 Pavement structure over concrete roof

It is understood that a portion of the pavement structure will be placed over a layer of rigid insulation over a concrete roof. It is recommended that the distance between the top of the pavement structure and the top of the rigid insulation be at least 450 mm, if the rigid insulation consists of an extruded polystyrene XPS (CAN/ULC AS701) of a compressive strength of 690 kPa (100 PSI). An XPS with a compressive strength of 415 kPa (60 PSI) could be used if that distance is increased to 600 mm and an XPS with a compressive strength of 275 kPa (40 PSI) could be used if that distance is increased to 750 mm.

Any areas where the pavement subgrade transitions from overburden materials to the concrete roof should be provided with a suitable granular frost taper consisting of excavating to a depth of at least 1.8 metres at the concrete roof face and tapering the excavation upwards at 5 horizontal to 1 vertical towards the pavement subgrade level. The excavation should then be filled with non-frost susceptible material such OPSS Granular B Type I or Type II compacted to 95 percent of the standard Proctor maximum dry density for the material used. The transition of the granular A layer could be carried out at a 3 horizontal to 1 vertical profile.

4.10 Grade Raise Restrictions

The site is underlain by a deposit of Silty Clay overlying till and bedrock. The silty clay layer is very stiff to stiff down to a depth of about 4.5 m where the clay becomes firm to stiff. Considering that the proposed structure will be founded on bedrock, the grade raise will have no impact on the structure. However, it should be noted that an excessive grade raise could result in the settlement of the ground surrounding the proposed structure and its paved areas. This settlement could result in cracking and/or unevenness of the asphaltic concrete and require maintenance in the form of, but not limited to, overlays and/or padding.

To limit settlements to 25 mm, a maximum grade raise limit of 1.8 metres above existing site grades could be used for this site. This grade raise restriction assumes that the fill material will have a maximum unit weight of 22 kN/m3 (e.g. OPSS Granular B Type II).

If a greater grade raise is required, additional geotechnical testing including consolidation analyses could be carried out in order to better evaluate the potential for settlement.

4.11 Slope Stability

The current slope conditions are described in the Slope Inspection Report, completed on May 18, 2018 and attached as Appendix B. Based on the inspection and rating of stability components following the guidelines prescribed by the Ministry of Natural Resources (1998), the slope rating is 22, which indicates low potential for slope instability. The following section describes development issues at the site as they relate to influencing slope stability.

4.11.1 Soil Stockpiling

The stockpiling of excess soil during excavation should be minimized and should not be placed at or near the slope crest to prevent loading of the slope.

4.11.2 Foundation Loading

It is anticipated that the proposed building will be founded directly on bedrock, where there are two underground parking levels (northern portion of the building), or indirectly on bedrock through piles. No additional foundation loading will occur as a result of site development.

4.11.3 Grade Raise Loading

The site grade will likely be altered as a result of the development and there will likely be a grade raise surrounding the building and the parking lot area. Any grade raise steeper than 5 horizontal to 1 vertical near the crest of the slope should be reviewed by a geotechnical engineer. The maximum grade raise limit of 1.8 m above the existing grade (Section 4.10) in the allowable development area (i.e. beyond the 30 m setback) will not result in steepening of the slope beyond the 5H:1V threshold.

4.11.4 Drainage

Surface runoff during and after construction should be directed to swales. Any drainage works (drainage pipes, etc.) should be directed away from the slope or should extended sufficiently to outlet below the toe of the slope. The drainage outlets should be protected using suitable riprap and underlain by a suitable geotextile.

A geotechnical engineer should review and approve any proposed works carried out within the slope or near the crest of the slope.

4.12 Other Considerations

4.12.1 Excess Soil Management

Excess soil material generated during excavation activities at the site should be managed in accordance with Ontario’s Best Management Practices Guide (2014) and Excess Soil Management Policy Framework (MOECC, 2016). At this time the MOECC has released proposed regulations, which are anticipated to be released as a final document sometime in 2018.

Soil samples were not tested for contamination (either natural or human induced) and this report does not constitute a Soil Management Plan.

4.12.2 Abandonment of Piezometers

The two piezometers installed during the field investigations (BH/MW18-01 and BH/MW18-06) should be decommissioned by a MOECC-licensed well technician. Well abandonment can be completed before or during construction activities.

4.12.3 Silt Fencing

Consideration should be given to installation of silt fencing along the 30 m setback line during construction to prevent sediment laden runoff from entering Shirley’s Brook.

5 LIMITATIONS AND CLOSURE

This report has been prepared for the exclusive use of 10731845 Canada Inc. for specific application to the proposed project at 788 March Road, in Kanata (Ottawa), Ontario. Data obtained from sampling investigations represent the conditions at the time of sampling and are subject to variability.

Geofirma Engineering Ltd. (Geofirma) has completed the study in accordance with generally accepted geotechnical engineering practice. Geofirma has exercised professional judgment in collecting and analyzing the information and in formulating recommendations based on the results for the guidance of the designers and is intended for this project only. The mandate at Geofirma is to perform the given tasks within guidelines prescribed by the client and with the quality and due diligence expected within the profession. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. No other warranty or representation expressed or implied, as to the accuracy of the information or recommendations is included or intended in this report.

Geofirma Engineering Ltd. hereby disclaims any liability or responsibility to any person or party, other than the party to whom this report is addressed, for any loss, damage, expense, fines or penalties which may arise or result from the use of any information or recommendations contained in this report by any other party. Any use of this report constitutes acceptance of the limits of Geofirma's liability. Geofirma's liability extends only to its client and only for the total amount of fees received from the client for this specific project and not to other parties who may obtain this report.

Respectfully submitted,

Geofirma Engineering Ltd.

21 Dec 2018
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Geotechnical Investigation Report
788 March Road, Kanata (Ottawa), Ontario

Revision: 2

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1 INTRODUCTION

Geofirma Engineering Ltd. (Geofirma) was retained by Omnipex Real Estate Inc. to complete a geotechnical investigation of the property located at 788 March Road, Kanata, herein referred to as `the site`. The geotechnical assessment has been completed to support development of the site.

The report presents the factual results of the geotechnical investigation and provides geotechnical recommendations for the proposed development of the site. Work has been completed in accordance with the *Geotechnical Investigation and Reporting Guidelines for Development Applications in the City of Ottawa* and the Canadian Foundation Engineering Manual, 4th Ed. (2006).

1.1 Project and Site Description

The site is currently a vacant parcel of land located at the southeast corner of March Road and Klondike Road, and bordered by Shirley's Brook to the northeast. The total area of the parcel is approximately 1.2 hectares (~3 acres). Land use surrounding the site is primarily low-rise commercial, with residential subdivisions located south of the site.

Site topography is moderately sloping downward to the northeast (toward Shirley's Brook). Elevation of the site adjacent to March Road is approximately 78 mASL, with the elevation at Shirley's Brook of approximately 72 mASL. The site is well vegetated with established grass and shrubs on the southern portion the site and mature trees on the north portion of the site.

It is understood that the proposed development of the site will include a multi-level residential complex, with one level of underground parking. The exact depth of footing for the proposed structure was not known at the time of submitting this report but it is understood that the structure will be founded at an elevation of about 73 mASL.

The location of the site is shown on Figure A.1, the proposed building footprint is illustrated on Figure A.2, and site topography is presented on Figure A.3, Appendix A.

1.2 Regional Geological Setting

The site is located within the Clay Plains physiographic region (Chapman and Putnam, 1984) and consists of fine-texture glaciomarine deposits, including silt and clay with minor sand and gravel (OGS, 2011). Bedrock at the site is mapped as dolostone and sandstone of the Beekmantown Group (OGS, 2011).

1.3 Background

There are no available geotechnical reports for the site; however, a review of the Ontario Geotechnical Borehole Database was completed prior commencing field activities to provide some indication on site specific stratigraphic conditions. Three boreholes are mapped within 200 m of the site: ID 609813, 609814 and 609816 with bedrock depths of 6.4 m, 5.5 m and 9.1 m, respectively.

1.4 Purpose and Scope

The purpose of the geotechnical investigation at the site was to characterize the engineering properties of the subsurface materials, and provide recommendations for site development based on the soil, rock and groundwater conditions.

The scope of work for the geotechnical investigation included the following:

- Review of existing background geological information, primarily data from Ontario Geotechnical Borehole Database.
- Completion of a borehole drilling program to provide spatial coverage and to characterize material properties of the key stratigraphic units.
- Completion of a geotechnical laboratory testing program to characterize the engineering properties of site soil and bedrock.
- Collection of representative soil and groundwater samples to characterize corrosion potential and recommend cement type; and
- Preparation of a geotechnical report providing a summary of field observations and subsurface conditions, laboratory results and geotechnical recommendations.

1.5 Report Organization

The Geotechnical Report is organized into six Sections and eight Appendices. Section 1 provides an overview of the site, and the purpose and scope of the project. The investigation methodology is outlined in Section 2. Section 3 presents a summary of the factual site data collected during the geotechnical investigation. Section 4 provides an assessment of the geotechnical properties of the subsurface materials and geotechnical recommendations. Limitations of this report are outlined in Section 5 and references are included in Section 6.

Appendices include report figures and borehole logs, as well as supporting laboratory results, hydraulic testing results and a copy of the seismic information for the site.

2 METHODOLOGY

Project activities were conducted by Geofirma between May 18 and June 15, 2018. This included the following:

- Slope stability assessment (site inspection)
- Underground utility locates
- Borehole drilling
- Piezometer installation and collection of groundwater elevations
- Surveying
- Geotechnical laboratory analysis; and
- Geochemical laboratory analysis.

2.1 Slope Stability Assessment

A site inspection was completed by Steve Gaines, M.A.Sc., P.Eng., on May 18, 2018 to visually inspect the slope along Shirley's Brook. Specifically, the site inspection was completed to observe current site conditions, slope characteristics, vegetative cover, as well as drainage and water course characteristics.

A preliminary assessment of the slope was completed in accordance with the guidelines provided in the Ontario Ministry of Natural Resources (MNR) Geotechnical Principles for Stable Slopes (MNR, 1998). Based on the observed site features, a slope rating of 22 was assigned and it was concluded that there was no significant toe erosion at the slope toe, the vegetation is well established, and there is no evidence of past instability. A copy of the preliminary inspection report is included in Appendix B. Site photographs collected during the inspection are included as Appendix C.

Based on MNR guidelines, a rating of less than 24 requires no further investigation; however, given the proposed development, an assessment of the post-development conditions (i.e. structural loading and grade-raises) is evaluated and discussed in Section 4.11.

2.2 Underground Utility Locates

Ontario OneCall was contacted to identify the location of all underground buried utilities at the site. Utilities including telephone, gas, hydro, cable/fiber and municipal services were cleared through these services.

2.3 Borehole Drilling

A total of eight (8) boreholes were drilled using a CME-75 track-mounted drilling rig, operated by Aardvark Drilling Inc. (ADI) of Carleton Place, Ontario. Drilling was completed between June 5 and 7, 2018. Boreholes were identified sequentially from BH18-01 to BH18-08 and presented on Figure A.2, Appendix A.

BH18-01 and BH18-02 were advanced using 203 mm hollow stem augers, while advancing a 51 mm diameter split spoon sampler. The remaining boreholes were advanced using a 152 mm diameter solid stem auger, also advancing a 51 mm diameter split spoon. Standard Penetration Tests (SPT) N-values were recorded by the supervising Geofirma technician. Field shear vane tests were collected, where appropriate, using a standard N-sized vane.

Table 1 summarizes the borehole drilling program, including ground surface elevation, depth to bedrock, and total drilled depth.

Table 1 Summary of Bedrock and Total Borehole Depth and Elevations

Borehole ID	Ground Surface Elevation (mASL)	Interpreted Bedrock Depth (Elevation)	Borehole Depth (Elevation)
BH18-01	74.86	5.6 (69.7)	9.3 (66.0)
BH18-02	77.07	6.2 (70.5)	6.2 (70.5)
BH18-03	77.26	6.2 (71.0)	6.2 (71.0)
BH18-04	76.57	5.9 (70.3)	5.9 (70.3)
BH18-05	77.06	5.6 (71.4)	6.6 (70.5)
BH18-06	76.46	5.6 (70.8)	5.6 (70.8)
BH18-07	77.03	5.5 (71.6)	5.5 (71.6)
BH18-08	76.35	5.3 (70.7)	5.3 (70.7)

Note: depth is referenced in metres below ground surface (mBGS) and elevation as metres above sea level (mASL)

Soil samples were collected from each borehole in 0.6 m intervals, and are identified as BH18-XX-Y, where XX is the borehole identifier and Y is the sequential sample interval. For example, BH18-02-3 indicates the third sample from borehole number 2. Samples were inspected in the field by an experienced field technician and collected in plastic freezer bags to preserve moisture conditions. Samples were brought back to the Geofirma office for detailed inspection by a geotechnical engineer. Selected samples were submitted for additional laboratory testing, as discussed in Section 2.7.

Complete borehole logs are included in Appendix D.

2.4 Groundwater Investigations

Piezometers were installed in BH18-01 and BH18-06 during the drilling investigation and subsequently renamed BH/MW18-01 and BH/MW18-06. The piezometer installed in BH/MW18-01 is screened in the upper bedrock surface, while the piezometer in BH/MW18-06 is screened above the bedrock surface. Both piezometers were constructed using 51 mm diameter PVC pipe with a 3.0 m slotted screen.

Water levels were measured using an electronic water level tape on June 8, 12 and 15, 2018 to determine static groundwater elevations. The elevation of Shirley’s Brook was also collected on June 15, 2018 to establish groundwater flow direction and approximate horizontal groundwater flow gradient.

Monitoring well instrumentation details, including static water level elevations, are included on the borehole stratigraphic logs in Appendix D.

2.5 Soil Classification

Soils were classified in the field based on visual and tactile examination by Geofirma technical staff based on accepted methods of classification used in geotechnical engineering practice. Laboratory testing of subsurface units was completed and incorporated into the finalized borehole logs and soil classification. Boundaries between stratigraphic units are generally transitional in nature and subsurface conditions represent the conditions in the borehole only. Conditions between boreholes represent an interpretation of the subsurface geology and should be confirmed during construction activities.

2.6 Surveying

A site topographical survey, referenced to a geodetic datum, was completed by J.D. Barnes Ltd. and provided to Geofirma. A supplemental survey was completed to tie in top of riser elevations for each of the piezometers, as well as to correct the ground surface elevation for boreholes that were adjusted during the drilling program (i.e. after the J.D. Barnes survey).

2.7 Laboratory Testing

Soil samples identified for additional characterization were submitted to the materials testing laboratory at Cambium Inc. in Peterborough, Ontario. Rock core samples were submitted to GEMTEC in Kanata. Both Cambium and GEMTEC labs are certified by the Canadian Council of Independent Laboratories (CCIL) for the completion of standard geotechnical laboratory testing.

In total, the following number of samples were submitted for geotechnical laboratory analysis:

- 6 x sieve/hydrometer
- 5 x atterberg
- 2 x unconfined compressive strength (rock core)

Two soil samples and one groundwater sample were collected and submitted to Paracel Laboratories in Ottawa to measure pH, resistivity, chloride and sulphate, for the purpose of determining the corrosion potential and recommended cement type based on the geochemistry of soil and groundwater.

Results of geotechnical and geochemical laboratory testing are discussed in Section 3. Complete laboratory reports are included in Appendix E.

2.8 Geophysical Testing

A geophysical testing program was completed by Geophysics GPR International Inc., of Longueil, Quebec on June 8, 2018. The geophysical survey was completed to determine the time-averaged shear wave velocity (Vs) in the upper 30 metres in order to determine the appropriate seismic site class as per the National Building Code of Canada and Ontario Building Code requirements.

A copy of the geophysics report is included in Appendix F and outlines the testing methodology and results.

3 SUBSURFACE CONDITIONS

All boreholes were completed to a minimum of spoon refusal, interpreted to be the bedrock surface. In general, site stratigraphy consists of topsoil, underlain by a clay and silt to approximately 5-6 mBGS. A thin (less than 0.5 m), discontinuous layer of till (clayey to sandy silt) was observed in some boreholes across the site overlying bedrock.

3.1 Overburden

A description of the key overburden units are described in the following sub-sections. A summary of the soil geotechnical laboratory testing is presented in Table 2. This includes grain size analysis (sieve/hydrometer), atterberg limits, and associated USCS classification and soil description. Moisture content for each sample is included on the borehole logs.

Complete laboratory reports are included in Appendix E.

Table 2 Summary of Sieve/Hydrometer Laboratory Analysis

Sample ID	Depth (mBGS)	Gravel (%)	Sand (%)	Silt / Clay (%)	LL / PL	USCS Classification - Description
BH18-01-3	1.5 - 2.1	0	1	99	56.5 / 23.6	CH-MH – Clay and Silt
BH18-03-7	4.6 - 5.2	0	0	100	56.4 / 20.0	CH-MH – Clay and Silt
BH18-04-8	5.5 – 5.9	0	2	98	42.6 / 19.4	CL-ML – Clay and Silt
BH18-05-8	5.3 – 5.9	2	13	85	32.2 / 15.6	ML – Clayey Silt some Sand trace Gravel
BH18-08-5	3.0 – 3.7	0	1	99	48.9 / 24.9	CL-ML – Clay and Silt
BH18-08-7	4.7 – 5.3	8	24	68	--	ML – Clayey Sandy Silt trace Gravel

3.1.1 Topsoil

Brown sandy silt topsoil, measuring approximately 0.3 m thick, was encountered across the site.

3.1.2 Clay and Silt

A deposit of clay and silt of low to high plasticity (CL-ML to CH-MH) was encountered across the entire site underlying the topsoil unit. The clay and silt was generally observed to extend to a depth of approximately 5.2 to 6.2 mBGS, with the exception of BH18-08 where the clay and silt was observed to a depth of only 4.8 mBGS.

The clay and silt unit was generally stiff to very stiff in the upper 3.5 to 4.5 m, with SPT N-values ranging from 5 to 13. The lower clay and silt was generally firm to stiff, with N-values ranging from 1 to 4. Water content increased with depth from 30-40% to greater than 40% in the lower grey clay and silt unit.

Field shear vane tests were attempted in the upper firm to stiff clay, but unsuccessful and reported as greater than 120 kPa. Successful vane tests were completed in the lower clay and silt (greater than 4.5 mBGS), where undrained shear strength ranges from 29 to 62 kPa. The ratio of intact to remoulded shear strength (determined through field shear vane testing) is 4.0 to 5.3, indicating a sensitive clay.

3.1.3 Till

A relatively thin layer of till was encountered in BH18-03, BH18-05, BH18-07 and BH18-08. The till can be classified as clayey silt to clayey sandy silt with trace gravel. SPT N-values in the till were reported less than 4; however, this is based on a limited thickness of till prior to encountering bedrock. The till was observed in the field to be soft/loose and wet to saturated.

3.2 Bedrock

A total of 3.7 m of rock core was collected from BH18-01 and 0.91 m from BH18-05 to characterize the upper bedrock surface. Bedrock at the site is described as sandy dolostone, likely belonging to the March Formation. At 8 mBGS (67 mASL), a transition to cream coloured quartz sandstone, possibly of the Nepean Formation, was noted.

Rock quality was observed to be good to excellent, with few natural fractures identified and RQD values greater than 75%. It should be noted that a conservative RQD of 59% was recorded in BH18-01 Core Run 1 (5.64 to 6.25 m). This was a short run with adequate core recovery (TCR) and actual rock quality is likely better. No vertical or sub-vertical fractures were identified.

Unconfined compressive strength (UCS) tests were completed on one representative sample from each cored borehole. Peak UCS values range from 80.60 MPa (sample BH18-05-5.94) to 183.60 MPa (sample BH18-01-5.74).

The depth and elevation to bedrock, or refusal and inferred bedrock, is presented on Figure A.4.

3.3 Groundwater

Groundwater elevations were measured in two wells, BH/MW18-01 and BH/MW18-06, on three occasions to establish static water level conditions. A summary of groundwater elevations are presented in Table 3.

Table 3 Summary of Water Elevations

Location	Water Elevation (mASL) 7-Jun-18	Water Elevation (mASL) 12-Jun-18	Water Elevation (mASL) 15-Jun-18
BH/MW18-01	73.42	73.25	73.20
BH/MW18-06	73.72	73.56	73.53
Shirley's Brook	--	--	71.83

Water levels are considered static and representative of site conditions at the time; however, it should be noted that fluctuations in groundwater elevation will occur seasonally, for example, due to spring freshet or periods of high/low precipitation.

3.3.1 Water Elevation and Interpreted Flow Direction

Groundwater and surface water elevations were measured on June 15, 2018 from the two piezometers and Shirley's Brook to establish approximate flow directions. Water elevations are presented on Figure A.5, Appendix A.

Based on a review of the surface and groundwater elevation data, groundwater flow is interpreted to be northeast, toward the brook.

3.3.2 Hydraulic Conductivity

Falling head slug tests were completed on the two on-site piezometers for the purpose of providing a preliminary estimate of the hydraulic properties of the shallow bedrock at BH/MW18-01 and the lower overburden at BH/MW18-06. The hydraulic conductivity was estimated using the method developed by Hvorslev (1951) to analyze water recovery.

Based on the analysis of water level recovery, the estimated hydraulic conductivity of the upper bedrock (BH/MW18-01) is approximately 2 x 10^{-5} m/s, while the estimated hydraulic conductivity of the overburden is 4 x 10^{-7} m/s.

A copy of the slug test results and analysis are presented in Appendix G.

4 GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

The following sections provide an assessment of the site conditions and geotechnical recommendations for design and construction based on the interpreted subsurface conditions gained from the borehole drilling investigation and laboratory testing. It is understood that the proposed development will consist of a mid-rise residential building, with one to two underground parking levels.

The geotechnical assessment is intended to assist design engineers. Any contractors undertaking work at the site should carefully examine the factual information and make their own interpretation of the suitability of the data as it pertains to their specific scope of work and requirements.

4.1 Earthquake Design Parameters

4.1.1 Seismic Hazard Data

The design ground motions based on seismic loads associated for an event with a 2% probability of exceedance in 50 years was established using the on-line seismic hazard calculator found on the Natural Resources Canada website. The peak ground acceleration (PGA) for firm ground conditions (NBCC 2015 Site Class ‘C’) is 0.256 g, where g is the gravitational acceleration.

A copy of the calculator output is provided in Appendix H.

4.1.2 Liquefaction

The clay and silt deposit encountered at the site are generally considered non susceptible to liquefaction based on their water content, liquid limit and plasticity index. It is noted, however, that sample BH18-05-8 is considered moderately susceptible to liquefaction and cyclic mobility.

It is anticipated that the proposed construction will include excavation to bedrock where there are two underground parking levels and that the portion of the building constructed with one level of underground parking will be founded directly on bedrock using piles or caissons.

4.1.3 Seismic Site Class

The shear wave velocity profile for the upper 30 m (V_s30) was determined using the multi-channel analysis of surface waves (MASW) method by geophysics specialists from Geophysics GPR International Inc. A representative survey profile was completed across the site to characterize the seismic velocity of overburden and bedrock. The estimated shear wave velocity of the bedrock is estimated to be greater than 1500 m/s; however, the shear wave velocity of the overburden is considerably lower. As such, seismic site class, which is determined as the average shear wave velocity of 30 m of material underlying the footings is dependent on the final design founding elevations.

Based on the Geophysics GPR report, included in in Appendix F, the recommended site classification, in accordance with Table 4.1.8.4A of the Ontario Building Code, is Site Class B where the depth of excavation results in less than 3 m of soil between the bottom of the footing and bedrock surface. Where the depth of excavation results in more than 3 m of soil between the bottom of the footing and bedrock surface, a Site Class C should be applied. If the buildings are founded on conventional footings directly over sound bedrock, a Site Class A could be used.

If the buildings are founded on deep foundations, a Site Class B or C should be used in accordance with the thickness of soil between the basement slab and the surface of bedrock. A Site Class B could be used where the depth of excavation results in less than 3 m of soil between the bottom of the basement slab and bedrock surface. A Site Class C should be used where the thickness of soil is more than 3 m.

4.2 Lateral Earth Pressure for shoring/foundation wall design

Active earth pressure including static and dynamic (seismic effect) can be included to estimate the lateral earth pressure where the wall will allow lateral yielding. Where the wall does not allow lateral yielding, at rest earth pressure should be used to estimate the lateral earth pressure.

The following Table 4 provides the recommended soil parameters for the design of the retaining wall/shoring.

Table 4 Soil Parameters for Retaining Wall Design

Material Type	Total unit weight KN/m³	Angle of internal friction φ'	"At rest" earth pressure coefficient (K_O)	Active earth pressure coefficient (K_A)	Combined static and seismic active earth pressure coefficient (K_{AE})
Granular A	23.0	35	0.43	0.27	0.37
Granular B Type II	23.5	32	0.47	0.31	0.41
Sand	20.0	30	0.50	0.33	0.43
Clay	17.0	30	1.00	0.50	0.60
Glacial till	23.0	35	0.43	0.27	0.37

The above values are given considering a horizontal surface at the back of the retaining wall/shoring, a straight wall and a wall friction angle of 0 degrees.

The active pressure at a depth (z) including static and dynamic (seismic effect) acting on the foundation wall for compacted sand or sand and gravel (OPSS Granular B Type I) can be estimated as suggested by the Canadian Foundation Engineering Manual (CFEM, 2006, Equation 24.9):

$$\sigma_z = (1 - K_v) \times K_{AE} \times \gamma \times (H) \qquad (1)$$

Where: σ_z is the lateral active earth pressure [kPa] including static, dynamic:

- H is the height of the wall [m];
- γ is the unit weight of backfill, assume 21 [kN/m^3];
- K_v is the vertical component of the earthquake acceleration [decimal of gravity acceleration g], K_v can be assumed to be 0g (as a conservative value);
- K_{AE} is the earth pressure coefficient for static and dynamic as shown in table 4 above,
- K_h is the horizontal component of the earthquake acceleration [decimal of gravity acceleration g], and can be taken as 50% of the PGA (k_h.= 0.128)

Additional external loads should be considered as well, such as surcharge at ground surface behind the wall accounting for traffic, equipment, or stockpiled soil.

4.3 Foundations

4.3.1 Conventional Strip and Pad Foundations

The boreholes completed at the site encountered silty clay overlying a relatively thin layer of till material directly above inferred bedrock at depths varying between 5.3 and 6.2 mBGS (elevation 69.7 to 71.6 mASL). As mentioned above, it is understood that the proposed footings will be founded around elevation 73 mASL. It is therefore anticipated that the structure will be founded on silty clay.

In order to minimize the potential for differential settlement, it is recommended that the structure be founded entirely over sound bedrock or entirely over a minimum of 0.8 m of overburden materials.

A preliminary analysis of the bearing capacity of the firm to stiff clay indicates that the serviceability limit state (SLS) bearing capacity bearing would be in the range of 90 to 150 kPa. It should be noted that maximum allowable footing widths will have to be restricted in order to limit the anticipated settlement of the buildings. Further geotechnical testing could be carried out at the site, including laboratory consolidation analyses, in order to evaluate the settlement potential of the firm silty clay. However, it should be noted that the bearing capacity and the maximum allowable footing width may not be sufficient to support the building’s loads without resulting in excessive settlements.

A coefficient of friction (μ) of 0.3 could be used in calculating the sliding friction forces between concrete and stiff silty clay.

4.3.2 Strip and pad Fondations on bedrock/Secant wall founded on bedrock

Strip and pad footings, designed with a minimum 1.0 metres in width, founded entirely over relatively sound bedrock may be designed using a maximum allowable bearing pressure of 2 MPa (Ultimate limit state (ULS) bearing resistance with a geotechnical resistance factor of 0.5). There is no corresponding serviceability limit state (SLS) bearing capacity for footings founded entirely over bedrock. All footings should be founded over relatively clean and sound bedrock. Any loose soil/mud/weathered bedrock should be removed from the footprint of the footings. It is recommended that the surface of the bedrock be cleaned by a pressure washer or compressed air prior to pouring the footings. Considering a founding level of about 73 mASL, sub excavations of at least 1.5 to 3.3 meters are anticipated in order to found the footings on bedrock.

Trenches could be excavated below the proposed footing level and backfilled with lean concrete (minimum 12 MPa) up to the footing level. However, this method may be difficult to implement as the surface of the bedrock should be cleaned of any loose/soil/mud/weathered bedrock. The sides of the excavations in overburden materials should therefore be sloped or shored in accordance with the requirements in Ontario’s Occupational Health and Safety Act (OHSA), O. Reg. 527/00 as mentioned below, in order to permit worker activity in the trenches. Furthermore, ground water infiltration from the till layer is anticipated and dewatering of the trenches may be required prior to pouring. Dewatering of the trenches may be difficult to carry out considering the low estimated hydraulic conductivity of the till. Trenches should extend at least 300 m on each side of the proposed footings. Dewatering methods may require the use of sump pits and pumps or well points.

A coefficient of friction (μ) of 0.46 could be used in calculating the sliding friction forces between concrete and clean bedrock.

4.3.3 Deep Foundations

As an alternative to lowering the footing level to the elevation of bedrock surface, the structure could be supported on end bearing deep foundations such as piles, caissons, piers, etc, on bedrock or socketed into bedrock. Attention should be given to the distance between the bottom of the proposed foundation and the surface of the bedrock in order to ensure that a suitable pile length is used.

4.3.3.1 Piles

Closed ended, concrete filled steel pipe piles or steel H-piles could be used. All of the piles should be driven to refusal. The refusal criteria will be highly dependent on the contractor’s pile driving equipment.

The contractor should be required to submit to the geotechnical engineer a copy of the proposed pile type and driving criteria for review prior to construction. An allowance should be made in the specifications for re-striking all of the piles at least once to confirm the design set and/or the permanence of the refusal and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first re-strike should receive additional re-striking until the design set criteria is met. All re-striking should be performed after 48 hours of the previous set. Furthermore, the specifications should make provision for dynamic testing of selected piles by the engineer to verify the transferred energy and pile capacities.

It should be noted that a stepped, near horizontal bedrock surface should not pose any significant problems for piling. However, steeply sloping bedrock surfaces could pose some difficulties during construction due to the potential for sliding of the tip of the pile along the bedrock surface during pile driving and could require the use of rock injector tips for the piles.

The post construction settlement of elements of the structure, which derive their support from properly terminated piles, should be negligible.

4.3.3.2 Caissons/Piers

Deep foundations founded on bedrock may be designed using a maximum allowable bearing pressure of 2 MPa (Ultimate limit state (ULS) bearing resistance with a geotechnical resistance factor of 0.5). There is no corresponding serviceability limit state (SLS) bearing capacity for footings founded entirely over bedrock. In order to facilitate inspections and cleaning of the pier subgrade, the size of caissons should be at least 900 mm in diameter. Construction difficulties associates with cobbles/boulders, groundwater seepage and sloughing conditions should be expected during installation of caissons.

Temporary casings should be used to prevent soil sloughing into the caisson and pumps should be available to dewater the caissons prior to concrete placement. If excessive groundwater flows are encountered, concrete for the caissons should be placed using tremie procedures.

4.3.4 Resistance to Lateral Loading

Lateral loading could be resisted by the soil resistance in front of the piles, using battered piles or by the use of rock-socketed caissons. The SLS geotechnical response of the soil in front of the piles under lateral loading can be calculated using linear behavior (e.g., theory of subgrade reaction) where maximum pile deflections are small (less than 1 % of the pile diameter), where the loading is static (no cycling) and where the pile material is linear (e.g., steel). If one or more of these conditions are not met, methods that can model the pile and soil non-linearity should be used such as non-linear resistance displacement relationships (p-y curves). The nonlinear lateral displacement of the piles could be estimated using section 18.5.1 of the Canadian Manual of Foundation Engineering 4th edition (2006) or commercially available software programs such as LPILE or FLPIER. The geotechnical parameters for the non-linear resistance displacement method are provided in Table 5, below.

4.3.4.1 Horizontal Coefficient of subgrade reaction

The modulus of subgrade reaction is a difficult parameter to evaluate properly because it is not a unique fundamental property that is readily measured. Its value depends on several factors including the size and shape of the foundations, the type of soil, the relative stiffness of the foundation and soil, etc. The technical literature cites typical values for the vertical modulus of subgrade reaction, k_{v1} (for a one-foot square plate). The coefficient of horizontal subgrade reaction, k_h, can be estimated based on the vertical modulus of subgrade reaction (k_{v1}), as described by Terzaghi (1955). Typical ranges in k_{v1} are summarized in Table 7.1 of the Canadian Manual of Foundation Engineering 4th edition (2006). The recommended horizontal modulus of reaction for 1 m diameter pile or a wall of 1m unit width(k_{h1}) are provided below in Table 5.

For a pile/pier diameter of B (in meters) or a wall width of B, the horizontal subgrade reaction, k_{hb}, for actual pile/pier diameter and wall width is

$$k_{hb}= \frac{1}{(3.28\,B)}k_{h1}$$

where: k_{h1}= coefficient of horizontal subgrade reaction for a 1 m diameter pile/pier or a wall of 1 m unit width, provided below in Table 5.

B = the pile diameter / wall width (in meters)

The recommended geotechnical parameters for the design of resistance to lateral loading are provided in Table 5 below.

Table 5 Geotechnical parameters for resistance to lateral loading

Elevation (m)	Soil type	Horizontal modulus of reaction for 1 m diameter pile or wall of 1m unit width (k_{h1}) MPa/m	Undrained Shear Strength (Cu) kPa	Unit weight (γ) kN/m^3
Pile cap to elevation 72.00 m	Very stiff to stiff clay	20 - 53	100	17
Elevation 72.00 m to bedrock	Firm to stiff clay	6 - 20	40	16

It is noted that the bottom of very stiff to stiff clay elevation varies from about 70,1 to 72,0 meters. The more critical elevation should be considered.

The ULS geotechnical resistance to lateral loading may be calculated using Brom's method as outlined in section 18.4.1 of the Canadian Manual of Foundation Engineering 4th edition (2006). The recommended undrained shear strength values are provided in the above Table 5.

4.4 Underground Parking Slab on Grade

As stated above, it is expected that the proposed building will be founded in undisturbed native silty clay. For predictable performance of the proposed concrete floor slab all soft/loose and any deleterious material should be removed within the proposed building area. The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any soft areas evident should be subexcavated and replaced with suitable engineered fill.

Engineered fill materials provided to support the concrete floor slab should consist of sand, or sand and gravel meeting the OPSS grading requirements for Granular B Type I or crushed stone meeting OPSS grading requirements for Granular B Type II. A minimum 150 millimetre thickness of crushed stone meeting OPSS Granular A should be provided immediately beneath the concrete floor slab. The engineered fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. Depending on the thickness of engineered fill required, suitable lightweight fill material may have to be used for the engineered fill.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage and expansion of the concrete. The saw cut depth should be about one quarter of the thickness of the slab. The crack control cuts should be placed at a grid spacing not exceeding about 5 meters.

If the underground parking will be unheated, the floor slab will require protection from frost effects. Details for suitable frost protection using rigid insulation can be provided, if required.

4.4.1 Vertical modulus of subgrade reaction

The modulus of subgrade reaction is a difficult parameter to evaluate properly because it is not a unique fundamental property that is readily measured. Its value depends on several factors including the size and shape of the footings (raft), the type of soil, relative stiffness of the foundation and soil, etc.

The value of the vertical soil reaction modulus can also vary from one point to another (center, edge or corner). Because the modulus value can change with size of footing, a one foot (300 mm) square footing has been adopted as the standard basis for comparison purposes, and frequently serves as the starting point for design.

The technical literature cites typical values for the vertical modulus of subgrade reaction, k_{v1} (for a one-foot square plate). Typical ranges in k_{v1} are summarized in Table 7.1 of the Canadian Manual of Foundation Engineering 4th edition (2006). Based on the typical values of the vertical modulus of subgrade reaction provided in the CMFE, a recommended value between k_{v1} = 10 and 30 MPa/m could be used for the design of the slab one grade founded at an elevation between 73 and 75 mASL.

The soil vertical modulus of subgrade reaction can also be estimated from the bearing capacity according to Bowles (1996):

$$k_{v1} = 40\ (\mathrm{SF})qa\ (in\ kPa/m)$$

where SF = safety factor

qa = the bearing capacity (in kPa)

The estimated vertical modulus of subgrade reaction as per Bowles ranges between 12 and 27 MPa/m.

If the loaded area on cohesive soil is of width B (in meters) and length mB, the vertical modulus of subgrade reaction for actual footing dimension B (k_{vb}) can be estimated with:

$$k_{vb} = \frac{k_{v1}}{3.28\ B}\left[\frac{m+0.5}{1.5m}\right]$$

Where k_{v1} = vertical modulus reaction for a one-foot square plate

B = Foundation width (in meters)

4.5 Frost Considerations

The design frost depth at the Site is 1.8 m (OPSD 3090.101), therefore all exterior footings should be protected by a minimum of 1.8 m of soil cover or an equivalent combination of soil thickness and insulation. If construction is completed during winter conditions, temporary frost protection should be provided.

4.6 Excavation

The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario’s Occupational Health and Safety Act (OHSA), O. Reg. 527/00. The soil encountered at the site, can be classified as Type 3. That is, open cut excavations deeper than 1.2 metres within overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical starting at the base of the excavations, or flatter.

The listed slopes are for fully drained excavations. Much gentler slopes could be required under undrained conditions, where local water infiltrations occur and where the excavations are exposed for prolonged periods of time. Any excavated material stockpiled near an excavation or trench should be stored at a distance equal to or greater than the depth of the excavation/trench and construction equipment traffic should be limited near open excavation.

If the aforementioned slopes are not possible or practical to achieve due to space restrictions or obstacles, the excavation should be shored according to OHSA Reg. 213/91. A professional engineer should design, approve and supervise the shoring and establish the shoring depth under the excavation profile. The excavation for the underground services could be carried out within tightly fitting, braced steel trench boxes, approved by a professional engineer.

Groundwater inflow from the native soils into the excavations during construction, if any, should be handled by pumping from sumps within the excavations. Surface water runoff into the excavation should be minimized and diverted away from the excavation.

4.6.1 Excavation in Bedrock

It is assumed that the weathered portion of the bedrock may be excavated using a large excavator and that the sound bedrock may require the use of line drilling and blasting techniques or a high-energy hoe-ram.

The slopes of the rock excavation may be vertical with a 1m wide bench at the soil-rock interface on all sides of the excavation. Any loose pieces of rock from the sidewalls of the excavation should be removed and the bottom of the excavation should be sufficiently flattened and exempt of rock ledges.

A condition survey of any nearby structures and services should be undertaken prior to any blasting or hoe ramming operations. The blasting should be carried out under the supervision of a vibration specialist engineer to ensure that the limiting vibration criteria, established by the vibration specialist engineer, are not exceeded.

4.7 Groundwater Control

4.7.1 Inflow during Construction

For design purposes, groundwater elevation in the overburden can assumed to be at 74 mASL, although water levels will fluctuate seasonally. It is anticipated that the proposed below grade level(s) will be below the groundwater level and some degree of inflow should be expected during construction below the anticipated static groundwater level.

Significant inflow is possible when excavating elevator or sump pits in the upper bedrock. Evidence of groundwater flow through fractures was identified during drilling (iron stained joints) and water recovery of BH/MW18-01 installed in the upper bedrock was rapid.

Groundwater seepage and infiltration into the excavation should be managed by pumping from sumps in the excavation. Surface water runoff should be diverted from the excavations where possible to minimize additional dewatering requirements.

4.7.2 Foundation Drainage

Considering the proposed building will have an underground parking level below the interpreted groundwater level, permanent perimeter drainage and under slab drainage is recommended. Perimeter Perimeter drainage pipe shall be embedded in a 300 mm layer of clear crushed stone, wrapped in a geotextile and located adjacent to the perimeter footings and in a parallel row, spaced 5 meters apart below the slab.

The drainage pipes should be positively connected to a water drainage system such as a dry well, a drainage ditch or a storm drain. The drainage system under the slab and the peripheral drainage system should be connected separately in case a system fails.

Drainage from building roofs should be controlled and exterior grades sloped away from the buildings to prevent ponding of water adjacent to the foundation walls. Drainage should not be directed over the slope as this may increase erosion and lead to future stability issues.

4.8 Site Services

4.8.1 Pipe Bedding and Cover Material

It is suggested that the service pipe bedding material consist of at least 150 mm of granular material meeting OPSS requirements for Granular A. Granular material meeting OPSS specifications for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as bedding or sub-bedding material is not recommended.

Cover material, from pipe spring line to at least 300 mm above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The sub-bedding, bedding and cover materials should be compacted in maximum 200 mm thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

4.8.2 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Some of the native material from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or re-compaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future roadway areas. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, sidewalks, etc., the trench backfill should be compacted in maximum 300 mm thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced where the trench backfill is not located within or in close proximity to existing or future roadways, driveways, sidewalks, or any other type of permanent structure.

4.8.3 Reuse of On-site Soils

The existing overburden materials at the site consist of mainly silty clay. The silty clay is considered to be frost susceptible and should not be used as backfill material directly against foundation walls. Excavated bedrock should not be used as backfill material unless it meets the physical properties and gradation requirements of OPSS Granular B – Type I, or equivalent.

It should be noted that the adequacy of a material for reuse as backfill will mainly depend on the water content of the material at the time of use and on the weather conditions at that time. Any excavated materials proposed for reuse should be stockpiled in a manner to promote drying and should be inspected and approved for reuse by a geotechnical engineer.

4.9 Corrosion Potential and Cement Type

Two representative soil samples and one sample of groundwater from the upper bedrock were collected and analyzed for a suite of parameters to assess corrosion potential of buried concrete and steel. Testing results are summarized in Table 5.

Table 6 Analytical Results – Corrosion Potential

Sample ID	Media	Depth (Elevation)	pH	Resistivity (ohm.cm)	Chloride (µg/g or mg/L)	Sulphate (µg/g or mg/L)
BH18-04-7	Soil	4.7-5.3 (71.9-71.3)	7.75	816	783	175
BH18-07-5	Soil	3.0-3.6 (74.0-73.4)	7.64	3560	143	41
BH/MW18-01	Groundwater	5.6-9.3 (69.3-65.6)	7.7	1420	115	41

The values of pH, resistivity, chloride and sulphate are indicators of the potential corrosiveness of the subsurface to unprotected steel.

The pH is in the low basic range and does not particularly increase soil corrosivity. The sulphate concentrations in the soils are below critical levels. The resistivity of samples BH18-04-7 and BH/MW18-01 are indicative of a severely corrosive to corrosive soil/groundwater and the resistivity of sample BH18-07-5 is indicative of a corrosive to moderately corrosive soil. The chloride content of BH18-04-7 is also indicative of a corrosive soil environment.

It is therefore recommended that corrosion mitigation be considered for exposed structural elements.

It should be noted that the corrosion potential of native soil/groundwater could be influenced by the application of de-icing salt (sodium chloride).

The concentration of sulphate provides an indication of the potential for sulphate attack on concrete that is in contact with groundwater or soil. The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete. Up to 0.10 percent in soil and up to 150 ppm in groundwater, the potential is negligible. From 0.10 to 0.20 percent in soil and from 150 to 1000 ppm in groundwater, the potential is mild but positive. From 0.20 to 0.50 percent in soil and from 1000 to 2000 ppm in groundwater the potential is considerable and over 0.50 percent in soil and over 2000 ppm in groundwater the potential is severe.

Based on NRC guidelines, the above mentioned samples are considered to have a negligible potential for sulphate attack of buried concrete. Therefore general use (GU) cement is appropriate for concrete in contact with native soil or groundwater.

4.10 Pavement Design Recommendations

All existing topsoil, vegetation, and/or organic soils must be removed from the proposed pavement area (including parking, driveway, light and heavy traffic zones) to the elevation of the design subgrade line elevation. The slope of the excavation should be no steeper than 5H:1V within 1.2 m of finished grade to minimize the effect of differential frost heave. The exposed design subgrade line elevation should be inspected by experienced geotechnical personnel, and any soft soil or organic soil should be sub-excavated below the design subgrade line elevation and replaced with compacted subgrade fill consistent with the requirements of OPSS Select Subgrade Material (SSM). Geofirma Engineering Ltd should be contacted if the soft soil/organic soils extend beyond 500 mm below the subgrade line elevation. Fill material should be tested and approved by experienced geotechnical personnel prior delivery to the site. Subgrade fill should be placed in lifts no thicker than 300 mm then compacted to 95% SPMDD. The exposed subgrades should be surface compacted with a large vibratory roller and inspected by experienced geotechnical personnel.

The design of the pavement structure depends on the anticipated traffic volume and types of vehicles. The suggested minimum pavement designs are shown in Table 6.

Table 7 Suggested Minimum Pavement Designs

Emergency Routes	Parking Pavement
40 mm HL-3	50 mm HL-3
60 mm HL-8 (HS)	-
150 mm Granular A	150 mm Granular A
450 mm Granular B Type II	300 mm Granular B Type II

The performance of the pavement structure is highly dependent on the subsurface groundwater conditions. To intercept excess subsurface water within the pavement structure granular materials, sub-drains with suitable outlets could be installed below the pavement area's subgrade. The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features.

Infrastructure preparation works should carried out in such a way that a granular cover is put in place on the exposed subgrade line as quickly as possible in order to avoid the movement of heavy equipment on the subgrade material. In addition, exposed surfaces should be protected against frost if work is done in winter.

4.10.1 Pavement structure over concrete roof

It is understood that a portion of the pavement structure will be placed over a layer of rigid insulation over a concrete roof. It is recommended that the distance between the top of the pavement structure and the top of the rigid insulation be at least 450 mm, if the rigid insulation consists of an extruded polystyrene XPS (CAN/ULC AS701) of a compressive strength of 690 kPa (100 PSI). An XPS with a compressive strength of 415 kPa (60 PSI) could be used if that distance is increased to 600 mm and an XPS with a compressive strength of 275 kPa (40 PSI) could be used if that distance is increased to 750 mm.

Any areas where the pavement subgrade transitions from overburden materials to the concrete roof should be provided with a suitable granular frost taper consisting of excavating to a depth of at least 1.8 metres at the concrete roof face and tapering the excavation upwards at 5 horizontal to 1 vertical towards the pavement subgrade level. The excavation should then be filled with non-frost susceptible material such OPSS Granular B Type I or Type II compacted to 95 percent of the standard Proctor maximum dry density for the material used. The transition of the granular A layer could be carried out at a 3 horizontal to 1 vertical profile.

4.11 Grade Raise Restrictions

The site is underlain by a deposit of Silty Clay overlying till and bedrock. The silty clay layer is very stiff to stiff down to a depth of about 4.5 m where the clay becomes firm to stiff. Considering that the proposed structure will be founded on bedrock, the grade raise will have no impact on the structure. However, it should be noted that an excessive grade raise could result in the settlement of the ground surrounding the proposed structure and its paved areas. This settlement could result in cracking and/or unevenness of the asphaltic concrete and require maintenance in the form of, but not limited to, overlays and/or padding.

To limit settlements to 25 mm, a maximum grade raise limit of 1.8 metres above existing site grades could be used for this site. This grade raise restriction assumes that the fill material will have a maximum unit weight of 22 kN/m3 (e.g. OPSS Granular B Type II).

If a greater grade raise is required, additional geotechnical testing including consolidation analyses could be carried out in order to better evaluate the potential for settlement.

4.12 Slope Stability

The current slope conditions are described in the Slope Inspection Report, completed on May 18, 2018 and attached as Appendix B. Based on the inspection and rating of stability components following the guidelines prescribed by the Ministry of Natural Resources (1998), the slope rating is 22, which indicates low potential for slope instability. The following section describes development issues at the site as they relate to influencing slope stability.

4.12.1 Soil Stockpiling

The stockpiling of excess soil during excavation should be minimized and should not be placed at or near the slope crest to prevent loading of the slope.

4.12.2 Foundation Loading

It is anticipated that the proposed building will be founded directly on bedrock, where there are two underground parking levels (northern portion of the building), or indirectly on bedrock through piles. No additional foundation loading will occur as a result of site development.

4.12.3 Grade Raise Loading

The site grade will likely be altered as a result of the development and there will likely be a grade raise surrounding the building and the parking lot area. Any grade raise steeper than 5 horizontal to 1 vertical near the crest of the slope should be reviewed by a geotechnical engineer. The maximum grade raise limit of 1.8 m above the existing grade (Section 4.10) in the allowable development area (i.e. beyond the 30 m setback) will not result in steepening of the slope beyond the 5H:1V threshold.

4.12.4 Drainage

Surface runoff during and after construction should be directed to swales. Any drainage works (drainage pipes, etc.) should be directed away from the slope or should extended sufficiently to outlet below the toe of the slope. The drainage outlets should be protected using suitable riprap and underlain by a suitable geotextile.

A geotechnical engineer should review and approve any proposed works carried out within the slope or near the crest of the slope.

4.13 Other Considerations

4.13.1 Excess Soil Management

Excess soil material generated during excavation activities at the site should be managed in accordance with Ontario's Best Management Practices Guide (2014) and Excess Soil Management Policy Framework (MOECC, 2016). At this time the MOECC has released proposed regulations, which are anticipated to be released as a final document sometime in 2018.

Soil samples were not tested for contamination (either natural or human induced) and this report does not constitute a Soil Management Plan.

4.13.2 Abandonment of Piezometers

The two piezometers installed during the field investigations (BH/MW18-01 and BH/MW18-06) should be decommissioned by a MOECC-licensed well technician. Well abandonment can be completed before or during construction activities.

4.13.3 Silt Fencing

Consideration should be given to installation of silt fencing along the 30 m setback line during construction to prevent sediment laden runoff from entering Shirley's Brook.

5 LIMITATIONS AND CLOSURE

This report has been prepared for the exclusive use of 10731845 Canada Inc. for specific application to the proposed project at 788 March Road, in Kanata (Ottawa), Ontario. Data obtained from sampling investigations represent the conditions at the time of sampling and are subject to variability.

Geofirma Engineering Ltd. (Geofirma) has completed the study in accordance with generally accepted geotechnical engineering practice. Geofirma has exercised professional judgment in collecting and analyzing the information and in formulating recommendations based on the results for the guidance of the designers and is intended for this project only. The mandate at Geofirma is to perform the given tasks within guidelines prescribed by the client and with the quality and due diligence expected within the profession. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. No other warranty or representation expressed or implied, as to the accuracy of the information or recommendations is included or intended in this report.

Geofirma Engineering Ltd. hereby disclaims any liability or responsibility to any person or party, other than the party to whom this report is addressed, for any loss, damage, expense, fines or penalties which may arise or result from the use of any information or recommendations contained in this report by any other party. Any use of this report constitutes acceptance of the limits of Geofirma's liability. Geofirma's liability extends only to its client and only for the total amount of fees received from the client for this specific project and not to other parties who may obtain this report.

Respectfully submitted,

Geofirma Engineering Ltd.

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APPENDIX A

Report Figures

LEGEND

🟥 788 March Road

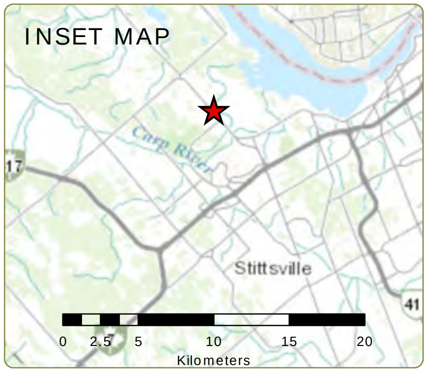

Figure A.1
Site Location

Scale 1:25,000

0 100 200 400 600 800
Meters

N

Coordinate System: NAD 1983 MTM 9
Service Layer Credits: © OpenStreetMap (and) contributors, CC-BY-SA
Sources: Esri, HERE, DeLorme, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community

PROJECT No. 18-206-2

PROJECT

Geotechnical Ivestigation
788 March Road, Ottawa, ON

DESIGN: ADG
CAD/GIS: ADG
CHECK: SNS
REV: 0

DATE: 04/07/2018

Geofirma
Engineering Ltd

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LEGEND

🟥	788 March Road
🟧	Proposed Building Outline
▥	Proposed Parking Area
▨	Proposed Amenity Areas
- - -	30m Setback from Shirley's Brook
--------	Required Setback from Property Boundary
⊕	Borehole Location
◕	Piezometer / Monitoring Well Location

Figure A.2
Site Layout

Scale 1:1,000

0 5 10 20 30 40
Meters

N

Coordinate System: NAD 1983 MTM 9
Source: NEUF Architect(e)s 18.06.22
Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

PROJECT No. 18-206-2

PROJECT

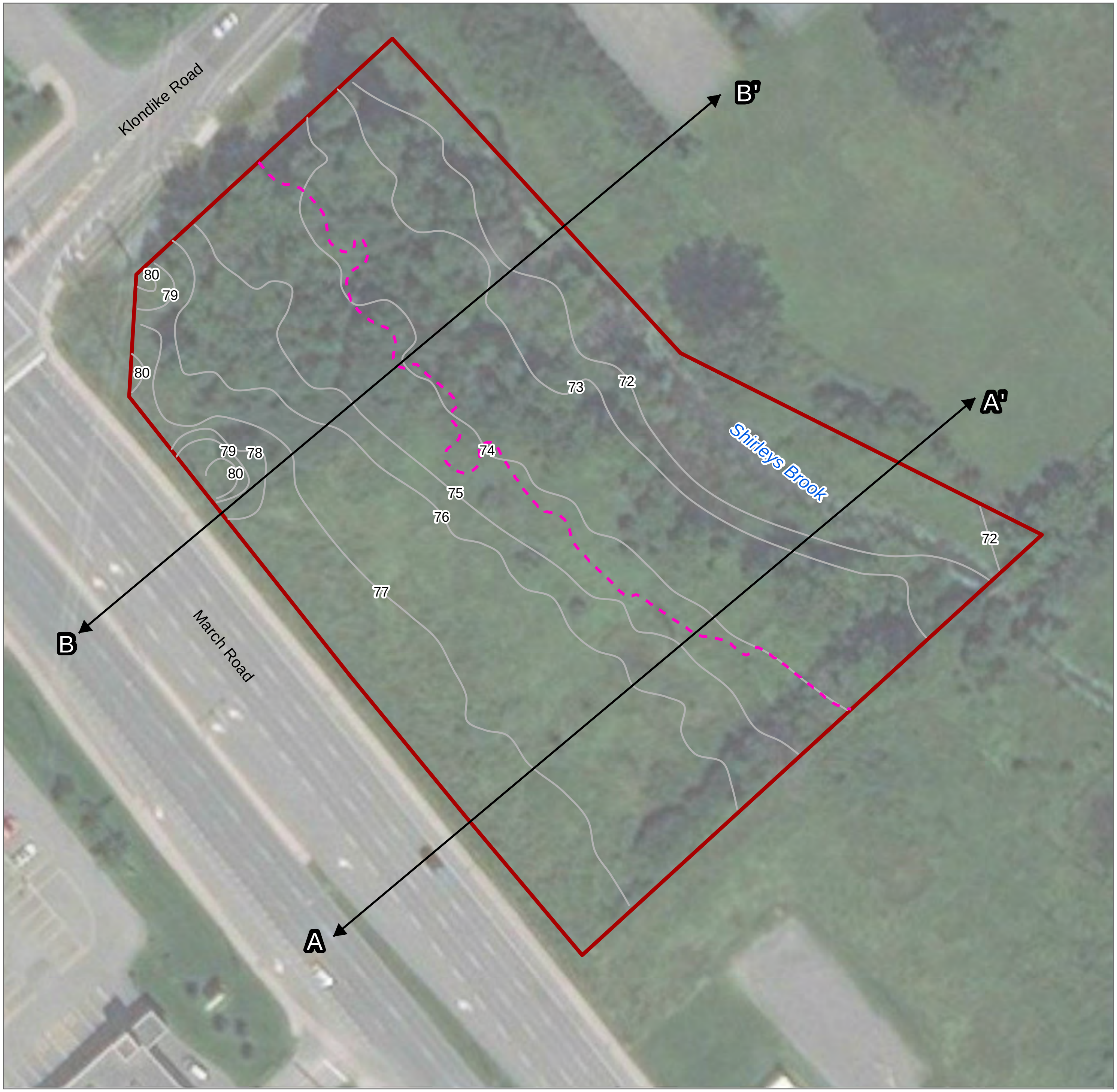
Geotechnical Ivestigation
788 March Road, Ottawa, ON

DESIGN: ADG
CAD/GIS: NMP/ADG
CHECK: SNG
REV: 0

DATE: 04/07/2018

Geofirma Engineering Ltd

G:\Data\Project\MarchRoad\Maps\18-206-2_788_MarchRd\18-206-2_FigA2_Layout.mxd



LEGEND

▭ 788 March Road

- - - 30m Setback from Shirley's Brook

— Contour Elevation Lines (mASL)

⟷ Cross-Section Lines

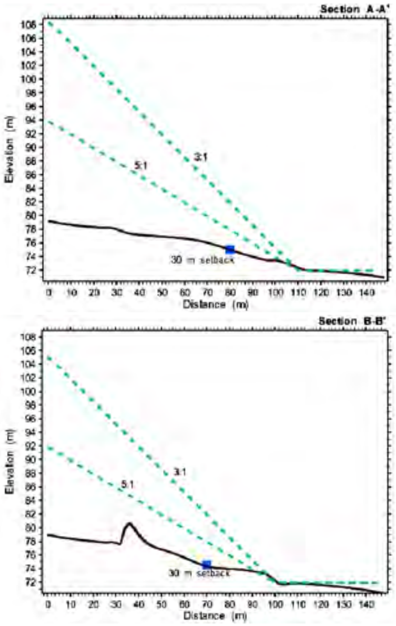


Figure A.3
Topography and Cross-Section Locations

Scale 1:1,000

0 5 10 20 30 40

Meters

N

Coordinate System: NAD 1983 MTM 9
Source: J.D. Barnes, 2018
Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

PROJECT No. 18-206-2

PROJECT
Geotechnical Ivestigation
788 March Road, Ottawa, ON

DESIGN: ADG
CAD/GIS: NMP/ADG
CHECK: SNG
REV: 0

DATE: 13/07/2018

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G:\Data\Project\MarchRoad\Maps\18-206-2_788_MarchRd\18-206-2_FigA3_Topographic.mxd

LEGEND

▭	788 March Road
- - -	30m Setback from Shirley's Brook
⊕	Borehole Location
◕	Piezometer / Monitoring Well Location
(5.45/70.3)	Inferred Bedrock Depth / Elevation (mBGS/mASL)

Figure A.4
Depth and Elevation to Bedrock Surface

Scale 1:1,000

0 5 10 20 30 40
Meters

N

Coordinate System: NAD 1983 MTM 9
Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

PROJECT No. 18-206-2

PROJECT

Geotechnical Ivestigation
788 March Road, Ottawa, ON

DESIGN: ADG
CAD/GIS: NMP/ADG
CHECK: SNG
REV: 0

DATE: 04/07/2018

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G:\Data\Project\MarchRoad\Maps\18-206-2_788_MarchRd\18-206-2_FigA4_BedrockSurface.mxd

LEGEND

▭	788 March Road
- - -	30m Setback from Shirley's Brook
⊕	Borehole Location
◑	Piezometer/Monitoring Well Location
(71.83)	Groundwater Elevation (mASL)
↗	Interpretted Groundwater Flow Direction

Figure A.5
Groundwater Elevation and Interpretted Flow Direction

Scale 1:1,000

0 5 10 20 30 40
Meters

N

Coordinate System: NAD 1983 MTM 9
Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

PROJECT No. 18-206-2

PROJECT

Geotechnical Ivestigation
788 March Road, Ottawa, ON

DESIGN: ADG
CAD/GIS: NMP/ADG
CHECK: SNG
REV: 0

Geofirma Engineering Ltd

DATE: 04/07/2018

G:\Data\Project\MarchRoad\Maps\18-206-2_788_MarchRd\18-206-2_FigA5_Groundwater.mxd

LEGEND

🟥 788 March Road

Ⓐ Photo Direction

Figure C.1
Photo Location and Directions

Scale 1:1,000

0 5 10 20 30 40

Meters

N

Coordinate System: NAD 1983 MTM 9
Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

PROJECT No. 18-206-2

PROJECT

Geotechnical Ivestigation
788 March Road, Ottawa, ON

DESIGN: ADG
CAD/GIS: NMP
CHECK: SNG
REV: 0

Geofirma Engineering Ltd

DATE: 04/07/2018

G:\Data\Project\MarchRoad\Maps\18-206-2_788_MarchRd\18-206-2_FigC1_Photos.mxd

A – Upper slope, looking north from the southern boundary of the property

B – Upper slope, looking northwest toward intersection of March Rd and Klondike Rd

C – Toe of slope at Shirley's Brook, looking northwest

D – Fill piles located at the base of the slope in the middle of the site, looking southeast

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Project: 18-206-2 Date: 18-May-18

E – Edge of Shirley's Brook, looking southeast from middle of site

F – Edge of Shirley's Brook, looking northwest from middle of site

G – Drainage swale with rip rap, northern boundary of site along Klondike Road, looking southwest

H – Box culvert across Klondike Road, looking northwest

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I – Shirley's Brook, looking southeast from northern boundary

J – View of site, from southern boundary looking north

APPENDIX D

Borehole Stratigraphic and Instrumentation Logs

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

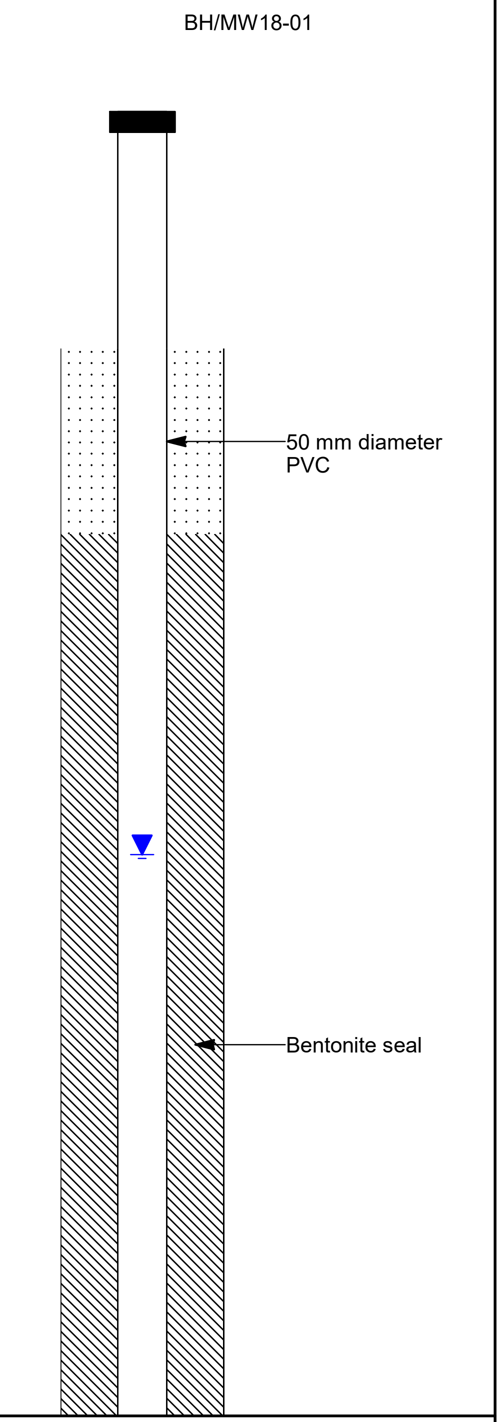
Borehole Number: BH/MW18-01

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427030.4E, 5022829.8N (UTM Zone 18)
Drilling Method: Split Spoon through Hollow Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID: A212889
Date Completed: 5-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 74.86 mASL
Date of Water Level Measurement: 15-Jun-18

DEPTH BGS (ft)	DEPTH BGS (m)	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION
0	0							GROUND SURFACE
0–1		☒						TOPSOIL sandy silt with organics, brown, dry
1		☒		3		20.4		CLAY and SILT brownish grey to grey, stiff to very stiff, dry
3.5		☒		8		30		
5.5								BH18-01-3: Gr=0%; Sa=1%; Si/Cl=99% {LL=56.5/PL=23.6}
6		☒	X	12		31.4		
8.5		☒		11		31.8		
11		☒		9		39.2		

INSTALLATION



Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

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BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

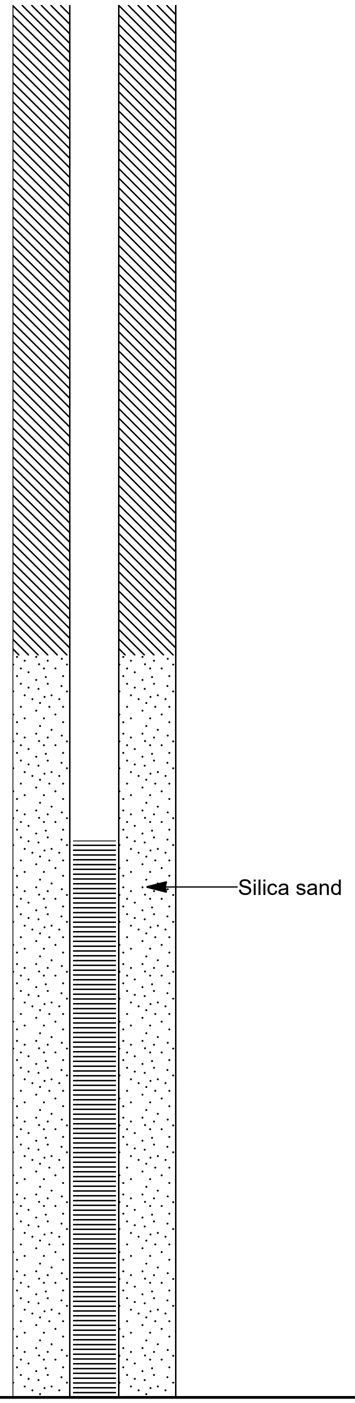
Borehole Number: BH/MW18-01

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427030.4E, 5022829.8N (UTM Zone 18)
Drilling Method: Split Spoon through Hollow Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID: A212889
Date Completed: 5-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 74.86 mASL
Date of Water Level Measurement: 15-Jun-18

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION
12	☒					▨	CLAY and SILT brownish grey to grey, stiff to firm, dry *(continued)*
13 4						▨	grey, firm to stiff, moist to saturated
14	☒		3		48.1	▨	
15				--	--	▨	Field Vane: Su = >120 kPa
16 5 17	☒		1		43.4	▨	
18				--	--	▨	Field Vane: Su = 48 kPa, Rem = no value
19							BEDROCK grey sandy dolostone with minor thin shale stringers
6 20				--	--		Core Run #1 (5.64 - 6.25 m): TCR = 79%, RQD = 56%
21 22 23 7				--	--		
24 25 26 8							Core Run #2 (6.25 - 7.77 m): TCR = 100%, RQD = 78%

INSTALLATION



Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

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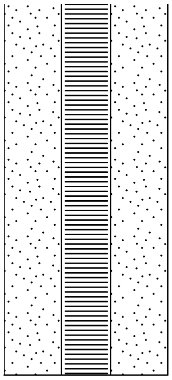
BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH/MW18-01

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427030.4E, 5022829.8N (UTM Zone 18)
Drilling Method: Split Spoon through Hollow Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID: A212889
Date Completed: 5-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 74.86 mASL
Date of Water Level Measurement: 15-Jun-18

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
27 28 29 9 30				--	--		sandstone, cream coloured, very hard *(continued)* Core Run #3 (7.77 - 9.30 m): TCR = 90%, RQD = 79% Borehole terminated at 66.0 mASL	
31 32 10 33 34 35 36 11 37 38 39 12 40 41							BOREHOLE TERMINATED Total Depth of BH/MW18-01 9.30 mBGS	Static Water Level = 2.86 mBGS / 73.20 mASL

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

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BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-02

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427023.3E, 5022812.2N (UTM Zone 18)
Drilling Method: Split Spoon through Hollow Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 5-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 77.07 mASL
Date of Water Level Measurement:

DEPTH BGS (ft / m)	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
							GROUND SURFACE	No Installation
1 ft	☒		4	--	46.6	≈	TOPSOIL sandy silt with organics, brown, dry	
	☒		11	--	28.9	▨	CLAY and SILT brownish grey to grey, stiff to very stiff, dry	
6 ft	☒		10	--	36.3	▨		
	☒		9	--	36.3	▨		
11 ft	☒		6	--	38.9	▨		
	☒		4	--	56.8	▨	grey, firm to stiff, moist to saturated	

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

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BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-02

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427023.3E, 5022812.2N (UTM Zone 18)
Drilling Method: Split Spoon through Hollow Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 5-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 77.07 mASL
Date of Water Level Measurement:

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
16 5 17	◫					▨	Field Vane: Su = >120 kPa CLAY and SILT brownish grey to grey, stiff to firm, dry *(continued)*	
18 19 6	☒		1	--	59	▨	brown sand lenses, soft to firm, moist to wet	
20	■			--	--	▨	Borehole terminated at 70.5 mASL - refusal, assumed bedrock	
21 22 23 7 24 25 26 8 27 28 29 9							BOREHOLE TERMINATED Total Depth of BH18-02 6.16 mBGS	

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

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BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-03

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427060.6E, 5022802.6N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 6-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 77.26 mASL
Date of Water Level Measurement:

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
							GROUND SURFACE	No Installation
							TOPSOIL sandy silt with organics, brown, dry	
			9	--	22.1		CLAY and SILT brownish grey to grey, stiff to very stiff, dry	
			11	--	31.7			
			10	--	32.2			
			8	--	39.3			
			7	--	38.5			
			6	--	47.0			

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

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BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-03

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427060.6E, 5022802.6N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 6-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 77.26 mASL
Date of Water Level Measurement:

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
	☒					▥	**CLAY and SILT** brownish grey to grey, stiff to firm, dry *(continued)*	
16		X	3		41.4	▥	grey, firm to stiff, moist to saturated	
5						▥	BH18-03-7: Gr=0%; Sa=0%; Si/Cl=100% {LL=56.4/PL=20.0}	
17						▥		
	▭			--	--	▥	Field Vane: Su = 62 kPa, Rem = 12 kPa	
18	☒					▨	**TILL** silty clay with trace gravel and sand, soft, wet	
19			3	--	40.8	▨		
6						▨	Borehole terminated at 71.0 mASL - refusal, assumed bedrock	
20							BOREHOLE TERMINATED Total Depth of BH18-03 6.16 mBGS	
21								
22								
23								
7								
24								
25								
26								
8								
27								
28								
29								
9								

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-04

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427044.3E, 5022788.1N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 6-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 76.57 mASL
Date of Water Level Measurement:

DEPTH BGS ft	DEPTH BGS m	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
								GROUND SURFACE	No Installation
		☒					≈	TOPSOIL sandy silt with organics, brown, dry	
1		☒		3	--	39.9	▨	CLAY and SILT brownish grey to grey, stiff to very stiff, dry	
2							▨		
3	1	☒		9	--	33.9	▨		
4							▨		
5							▨		
6	2	☒		10	--	35.0	▨		
7							▨		
8		☒		9	--	40.3	▨		
9							▨		
10	3						▨		
11		☒		5	--	40.9	▨		
12							▨		
13	4	☒		4	--	49.2	▨		
14							▨		

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

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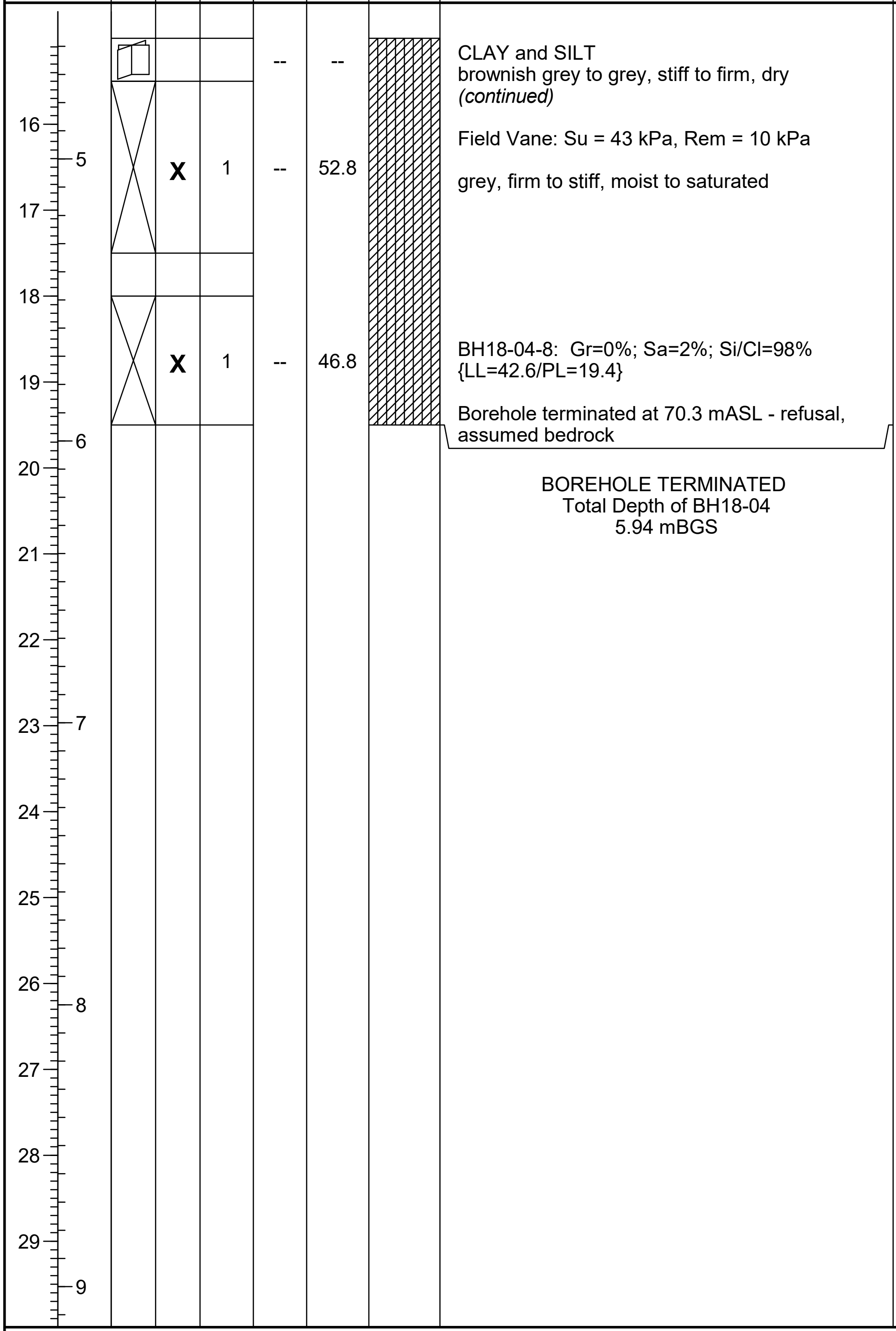
BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-04

Project Number: 17-224-2	MOE Well ID:
Client: 10731854 Canada Inc.	Date Completed: 6-Jun-18
Site Location: 788 March Road, Kanata, Ontario	Supervisor: SNG
Coordinates: 427044.3E, 5022788.1N (UTM Zone 18)	Logged By: TKG
Drilling Method: Split Spoon and Solid Stem Augers	Ground Surface Elevation: 76.57 mASL
Drilling Rig: CME 75 Track Mount	Date of Water Level Measurement:



DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
				--	--	▨	CLAY and SILT brownish grey to grey, stiff to firm, dry *(continued)* Field Vane: Su = 43 kPa, Rem = 10 kPa	
	☒	**X**	1	--	52.8	▨	grey, firm to stiff, moist to saturated	
	☒	**X**	1	--	46.8	▨	BH18-04-8: Gr=0%; Sa=2%; Si/Cl=98% {LL=42.6/PL=19.4} Borehole terminated at 70.3 mASL - refusal, assumed bedrock	

BOREHOLE TERMINATED
Total Depth of BH18-04
5.94 mBGS

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

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BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-05

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427064.3E, 5022762.3N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 7-Jun-18
Supervisor: SNG
Logged By: SNG
Ground Surface Elevation: 77.06 mASL
Date of Water Level Measurement:

DEPTH BGS (ft | m)	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
							GROUND SURFACE	No Installation
0–1 ft	☒					≈	**TOPSOIL** sandy silt with organics, brown, dry	
1 ft			6		19.0	▨	**CLAY and SILT** brownish grey to grey, stiff to very stiff, dry	
2 ft						▨		
3 ft | 1 m	☒		12		32.3	▨		
4 ft						▨		
5 ft	☒					▨		
6 ft | 2 m			10		34.4	▨		
7 ft						▨		
8 ft	☒		9		35.4	▨		
9 ft						▨		
10 ft | 3 m	☒					▨		
11 ft			5		38.8	▨		
12 ft						▨		
13 ft | 4 m	☒		6		41.5	▨		
14 ft						▨		

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-05

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427064.3E, 5022762.3N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 7-Jun-18
Supervisor: SNG
Logged By: SNG
Ground Surface Elevation: 77.06 mASL
Date of Water Level Measurement:

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
16 ft 5 m 17 ft	☒		3	--	47.0	▨	CLAY and SILT brownish grey to grey, stiff to firm, dry *(continued)* grey, trace sand, firm, moist to saturated	
18 ft	☒	X	50		30.6	░	TILL clayey silt some sand trace gravel, grey, wet BH18-05-8: Gr=2%; Sa=13%; Si/Cl=85% {LL=32.2/PL=15.6}	
19 ft 6 m 20 ft	▮			--	--	▤	BEDROCK light grey sandy dolostone, minor shale stringers, hard Core Run #1 (5.64 - 6.25 m): TCR = 100%, RQD = 87%	
21 ft	▮			--	--	▤	Core Run #2 (6.25 - 6.55 m): TCR = 100%, RQD = 78% Borehole terminated at 70.5 mASL	
22 ft 23 ft 7 m 24 ft 25 ft 26 ft 8 m 27 ft 28 ft 29 ft 9 m							BOREHOLE TERMINATED Total Depth of BH18-05 6.55 mBGS	

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

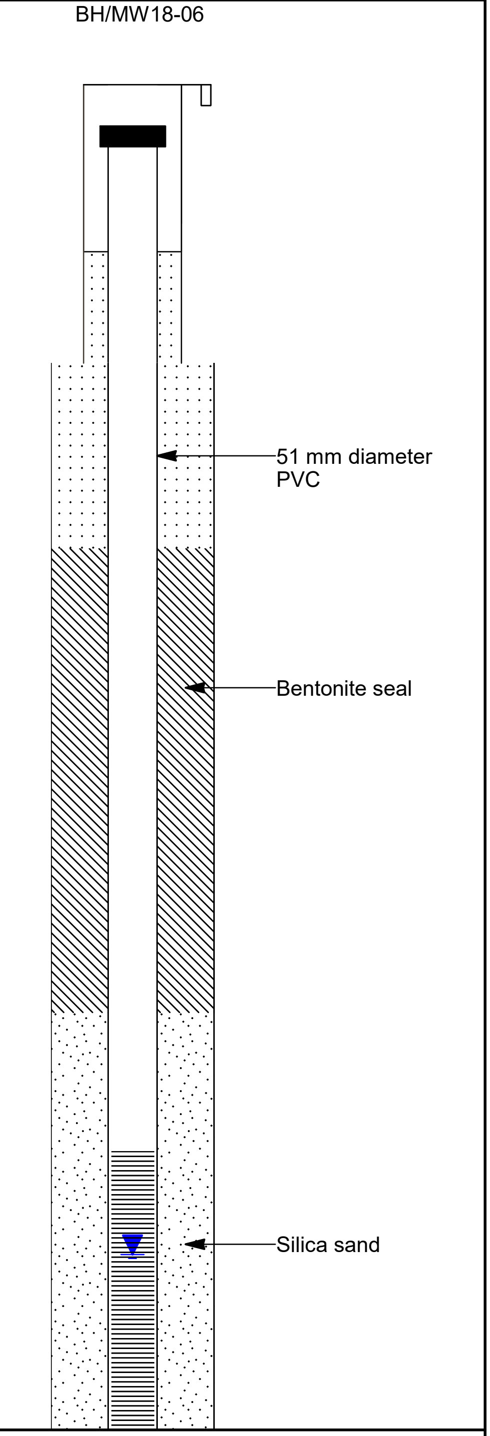
Borehole Number: BH/MW18-06

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427080.3E, 5022776.9N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID: A212888
Date Completed: 6-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 76.46 mASL
Date of Water Level Measurement: 15-Jun-18

DEPTH BGS (ft)	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION
0							GROUND SURFACE
0	☒					≈	TOPSOIL sandy silt with organics, brown, dry
1	☒		7	--	28.4	▨	CLAY and SILT brownish grey to grey, stiff to very stiff, dry
3.5	☒		10	--	33.6	▨	
6	☒		10	--	38.6	▨	
8.5	☒		10	--	39.4	▨	
11	☒		5	--	42.2	▨	

INSTALLATION



Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH/MW18-06

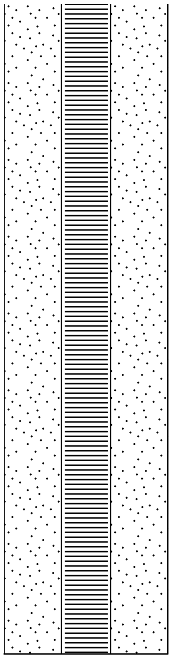
Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427080.3E, 5022776.9N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID: A212888
Date Completed: 6-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 76.46 mASL
Date of Water Level Measurement: 15-Jun-18

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION
	☒					▨	CLAY and SILT brownish grey to grey, stiff to very stiff, dry *(continued)*
	☒		3	--	44.9	▨	grey, firm to stiff, moist to saturated
				--	--	▨	Field Vane: Su = >120 kPa
	☒		3	--	45.7	▨	
				--	--	▨	Field Vane: Su = >120 kPa
							Borehole terminated at 70.8 mASL - refusal, assumed bedrock

BOREHOLE TERMINATED
Total Depth of BH/MW18-06
5.64 mBGS

INSTALLATION



Static Water Level =
3.98 mBGS / 73.53 mASL

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-07

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427086.9E, 5022735.5N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 7-Jun-18
Supervisor: SNG
Logged By: SNG
Ground Surface Elevation: 77.03 mASL
Date of Water Level Measurement:

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
ft | m							GROUND SURFACE	No Installation
1	☒		9	--	32.9	≈	TOPSOIL sandy silt with organics, brown, dry	
2						▨	CLAY and SILT brownish grey to grey, stiff to very stiff, dry	
3 | 1 4	☒		11	--	37.5	▨		
5 6 7 | 2	☒		13	--	43.4	▨		
8 9 10 | 3	☒		6	--	41.6	▨		
11 12	☒	X	4	--	44.7	▨		
13 | 4 14	☒		2	--	41.2	▨	grey, firm, moist to saturated	

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-07

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427086.9E, 5022735.5N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 7-Jun-18
Supervisor: SNG
Logged By: SNG
Ground Surface Elevation: 77.03 mASL
Date of Water Level Measurement:

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
	▯			--	--		Field Vane: Su = 38 kPa, Rem = 7 kPa	
16 5 17	☒		3	--	22.2			
18	☒		50	--	--	▨	TILL clayey sandy silt trace sand, grey, wet, soft Borehole terminated at 71.6 mASL - refusal, assumed bedrock	
19 6 20 21 22 23 7 24 25 26 8 27 28 29 9							BOREHOLE TERMINATED Total Depth of BH18-07 5.49 mBGS	

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma
Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-08

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427105E, 5022759.6N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 6-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 76.35 mASL
Date of Water Level Measurement:

DEPTH BGS ft	DEPTH BGS m	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
								GROUND SURFACE	No Installation
								TOPSOIL sandy silt with organics, brown, dry	
1				5	--	32.9		CLAY and SILT brownish grey to grey, stiff to very stiff, dry	
2									
3	1								
				8	--	37.5			
4									
5									
6				6	--	43.4			
	2								
7									
8									
				4	--	41.6			
9									
10	3								
11			X	4	--	44.7		BH18-08-5: Gr=0%; Sa=1%; Si/Cl=99% {LL=48.9/PL=24.9}	
12									
13	4							grey, firm, moist to saturated	
				3	--	41.2			
14									

Prepared By: SNG
Reviewed By: BC
Doc: 18-206-2_BH LOG_R0.GPJ
Template: GEOFIRMA_TEMPLATE.GDT

Geofirma Engineering Ltd

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Borehole Number: BH18-08

Project Number: 17-224-2
Client: 10731854 Canada Inc.
Site Location: 788 March Road, Kanata, Ontario
Coordinates: 427105E, 5022759.6N (UTM Zone 18)
Drilling Method: Split Spoon and Solid Stem Augers
Drilling Rig: CME 75 Track Mount

MOE Well ID:
Date Completed: 6-Jun-18
Supervisor: SNG
Logged By: TKG
Ground Surface Elevation: 76.35 mASL
Date of Water Level Measurement:

DEPTH BGS	SAMPLES	LAB SAMPLE	BLOW COUNT	CGI (ppm)	Water Content (%)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
							Field Vane: Su = 29 kPa, Rem = 7 kPa	
16 5 17		X	4	--	22.2	▨	TILL clayey sandy silt trace sand, grey, wet, soft BH18-08-7: Gr=8%; Sa=24%; Si/Cl=68% Borehole terminated at 70.7 mASL - refusal, assumed bedrock	
18 19 6 20 21 22 23 7 24 25 26 8 27 28 29 9							BOREHOLE TERMINATED Total Depth of BH18-08 5.27 mBGS	

APPENDIX E

Geotechnical and Geochemical Laboratory Results

Moisture Content Determination - LS-701

Lab Number: S-18-559

Project Name: Geofirma Lab Testing

Project No.: 7019-001

Client: Geo Firma Engineering Limited

Date Taken: June 5-7, 2018

Tested by:

Date Tested: June 11, 2018

BOREHOLE NUMBER	01	01	01	01	01	01	01
SAMPLE NUMBER	1	2	3	4	5	6	7
DEPTH OF SAMPLE	0 - 2'	2.5-4.5'	5-7'	7.5-9.5'	10-12'	12.5-14.5'	15-17'
TARE NUMBER	A312	A1	PAN	A301	A306	A112	A21
TARE	5.4	4.8	16.2	4.7	6.0	5.3	5.0
WT. WET SOIL + TARE	121.1	125.2	925.8	111.7	125.3	94.9	143.5
WT. DRY SOIL + TARE	101.5	97.4	708.2	85.9	91.7	65.8	101.6
WEIGHT OF WATER	19.6	27.8	217.6	25.8	33.6	29.1	41.9
WT. OF DRY SOIL	96.1	92.6	692.0	81.2	85.7	60.5	96.6
WATER CONTENT	**20.4**	**30.0**	**31.4**	**31.8**	**39.2**	**48.1**	**43.4**
ADDITIONAL OBSERVATIONS							

BOREHOLE NUMBER	02	02	02	02	02	02	02
SAMPLE NUMBER	1	2	3	4	5	6	7
DEPTH OF SAMPLE	0 - 2'	2.5-4.5'	5-7'	7.5-9.5'	10-12'	12.5-14.5'	18-20'
TARE NUMBER	A435	PIE	A52	A322	A55	A15	A311
TARE	4.2	4.6	4.6	5.0	5.4	4.4	4.8
WT. WET SOIL + TARE	127.6	78.1	66.6	76.7	67.5	83.1	61.1
WT. DRY SOIL + TARE	88.4	61.6	50.1	57.6	50.1	54.6	40.2
WEIGHT OF WATER	39.2	16.5	16.5	19.1	17.4	28.5	20.9
WT. OF DRY SOIL	84.2	57.0	45.5	52.6	44.7	50.2	35.4
WATER CONTENT	**46.6**	**28.9**	**36.3**	**36.3**	**38.9**	**56.8**	**59.0**
ADDITIONAL OBSERVATIONS		No remaining	9 No remaining	9 No remaining	9 No remaining		

BOREHOLE NUMBER	03	03	03	03	03	03	03
SAMPLE NUMBER	8	1	2	3	4	5	6
DEPTH OF SAMPLE	18-20'	0-2'	2.5-4.5'	5-7'	7.5-9.5'	10-12'	12.5-14.5'
TARE NUMBER	A330	A339	A19	A5	A120	A328	A30
TARE	5.7	5.3	4.5	4.6	4.6	4.4	5.2
WT. WET SOIL + TARE	146.9	73.8	117.0	177.0	143.5	105.9	124.6
WT. DRY SOIL + TARE	106.0	61.4	89.9	135.0	104.3	77.7	86.4
WEIGHT OF WATER	40.9	12.4	27.1	42.0	39.2	28.2	38.2
WT. OF DRY SOIL	100.3	56.1	85.4	130.4	99.7	73.3	81.2
WATER CONTENT	**40.8**	**22.1**	**31.7**	**32.2**	**39.3**	**38.5**	**47.0**
ADDITIONAL OBSERVATIONS		1	1				

1 - Contains organics
2 - Contains rubble
3 - Hydrocarbon odour
4 - Unknown chemical odour
5 - Saturated - free water visible
6 -Very moist - near optimum moisture content
7 - Moist - below optimum moisture
8 - Dry - dry texture - powdery
8 - Very small - caution may not be representative
10 - Hold sample for gradation analysis

Moisture Content Determination - LS-701

Lab Number: S-18-559

Project Name: Geofirma Lab Testing

Project No.: 7019-001

Client: Geo Firma Engineering Limited

Date Taken: June 5-7, 2018

Tested by:

Date Tested: June 11, 2018

CAMBIUM

BOREHOLE NUMBER	03	04	04	04	04	04	04
SAMPLE NUMBER	7	1	2	3	4	5	6
DEPTH OF SAMPLE	15-17'	0 - 2'	2.5-4.5'	5-7'	7.5-9.5'	10-12'	12.5-14.5'
TARE NUMBER	PAN	A213	A22	A109	A300	A336	A323
TARE	15.9	4.7	4.8	4.5	4.9	6.1	5.0
WT. WET SOIL + TARE	934.8	117.7	129.1	154.7	152.9	187.2	171.5
WT. DRY SOIL + TARE	665.7	85.5	97.6	115.8	110.4	134.6	116.6
WEIGHT OF WATER	269.1	32.2	31.5	38.9	42.5	52.6	54.9
WT. OF DRY SOIL	649.8	80.8	92.8	111.3	105.5	128.5	111.6
WATER CONTENT	**41.4**	**39.9**	**33.9**	**35.0**	**40.3**	**40.9**	**49.2**
ADDITIONAL OBSERVATIONS		1					

BOREHOLE NUMBER	04	04	05	05	05	05	05
SAMPLE NUMBER	7	8	1	2	3	4	5
DEPTH OF SAMPLE	15.5-17.5'	18-19.5	0 - 2'	2.5-4.5'	5-7'	7.5-9.5'	10-12'
TARE NUMBER	A304	PAN	A1014	A361	A200	A353	A320
TARE	4.8	15.6	4.7	4.8	4.4	4.8	4.7
WT. WET SOIL + TARE	115.3	1132.0	82.9	194.4	102.9	114.9	129.5
WT. DRY SOIL + TARE	77.1	775.9	70.4	148.1	77.7	86.1	94.6
WEIGHT OF WATER	38.2	356.1	12.5	46.3	25.2	28.8	34.9
WT. OF DRY SOIL	72.3	760.3	65.7	143.3	73.3	81.3	89.9
WATER CONTENT	**52.8**	**46.8**	**19.0**	**32.3**	**34.4**	**35.4**	**38.8**
ADDITIONAL OBSERVATIONS			1	1			

BOREHOLE NUMBER	05	05	05	06	06	06	06
SAMPLE NUMBER	6	7	8	1	2	3	4
DEPTH OF SAMPLE	12.5-14.5'	15-17'	17.5-19.5'	0 - 2'	2.5-4.5'	5-7'	7.5-9.5'
TARE NUMBER	A104	P3	PAN	PX	A313	A36	A334
TARE	5.0	4.7	15.8	4.4	6.2	4.1	4.8
WT. WET SOIL + TARE	144.7	134.5	703.6	125.7	122.8	159.8	156.3
WT. DRY SOIL + TARE	103.7	93.0	542.3	98.9	93.5	116.4	113.5
WEIGHT OF WATER	41.0	41.5	161.3	26.8	29.3	43.4	42.8
WT. OF DRY SOIL	98.7	88.3	526.5	94.5	87.3	112.3	108.7
WATER CONTENT	**41.5**	**47.0**	**30.6**	**28.4**	**33.6**	**38.6**	**39.4**
ADDITIONAL OBSERVATIONS				1			

CAMBIUM

1 - Contains organics
2 - Contains rubble
3 - Hydrocarbon odour
4 - Unknown chemical odour
5 - Saturated - free water visible
6 -Very moist - near optimum moisture content
7 - Moist - below optimum moisture
8 - Dry - dry texture - powdery
8 - Very small - caution may not be representative
10 - Hold sample for gradation analysis

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Moisture Content Determination - LS-701

Lab Number: S-18-559

Project Name: Geofirma Lab Testing

Project No.: 7019-001

Client: Geo Firma Engineering Limited

Date Taken: June 5-7, 2018

Tested by:

Date Tested: June 11, 2018

CAMBIUM

BOREHOLE NUMBER	06	06	06	07	07	07	07
SAMPLE NUMBER	5	6	7	1	2	3	4
DEPTH OF SAMPLE	10-12'	15 - 16.5'	20 - 21.5'	0 - 2'	2.5 - 4'	5 - 6.5'	7.5 - 9'
TARE NUMBER	A345	A113	A324	A329	A371	A366	A43
TARE	4.9	4.6	5.2	5.6	5.2	5.5	4.3
WT. WET SOIL + TARE	151.9	164.3	144.6	140.5	119.8	202.0	127.0
WT. DRY SOIL + TARE	108.3	114.8	100.9	108.9	90.1	149.0	91.8
WEIGHT OF WATER	43.6	49.5	43.7	31.6	29.7	53.0	35.2
WT. OF DRY SOIL	103.4	110.2	95.7	103.3	84.9	143.5	87.5
WATER CONTENT	**42.2**	**44.9**	**45.7**	**30.6**	**35.0**	**36.9**	**40.2**
ADDITIONAL OBSERVATIONS							

BOREHOLE NUMBER	07	07	07	07	08	08	08
SAMPLE NUMBER	5	6	7	8	1	2	3
DEPTH OF SAMPLE	10 - 11.5'	12-14.5'	15.5-17.5'	17.5-18'	0 - 2'	2.5-4.5'	5-7'
TARE NUMBER	A203	A351	A41	A12	A2	A33	A317
TARE	4.1	6.0	5.2	4.0	5.1	4.0	4.8
WT. WET SOIL + TARE	160.2	128.9	178.5	94.9	134.7	146.9	164.7
WT. DRY SOIL + TARE	112.5	88.4	131.6	72.9	102.6	107.9	116.3
WEIGHT OF WATER	47.7	40.5	46.9	22.0	32.1	39.0	48.4
WT. OF DRY SOIL	108.4	82.4	126.4	68.9	97.5	103.9	111.5
WATER CONTENT	**44.0**	**49.2**	**37.1**	**31.9**	**32.9**	**37.5**	**43.4**
ADDITIONAL OBSERVATIONS					1		

BOREHOLE NUMBER	08	08	08	08	BH18-05-7		
SAMPLE NUMBER	4	5	6	7	05-7		
DEPTH OF SAMPLE	7.5-9.5'	10-12'	12.5-14.5'	15.5-17.4'	15-17'		
TARE NUMBER	A357	PAN	A7	BOWL	A318		
TARE	5.6	15.9	5.2	276.5	5.3		
WT. WET SOIL + TARE	150.7	1109.3	182.4	1731.5	180.7		
WT. DRY SOIL + TARE	108.1	771.6	130.7	1467.2	129.9		
WEIGHT OF WATER	42.6	337.7	51.7	264.3	50.8		
WT. OF DRY SOIL	102.5	755.7	125.5	1190.7	124.6		
WATER CONTENT	**41.6**	**44.7**	**41.2**	**22.2**	**40.8**		
ADDITIONAL OBSERVATIONS							

CAMBIUM

1 - Contains organics
2 - Contains rubble
3 - Hydrocarbon odour
4 - Unknown chemical odour
5 - Saturated - free water visible
6 -Very moist - near optimum moisture content
7 - Moist - below optimum moisture
8 - Dry - dry texture - powdery
8 - Very small - caution may not be representative
10 - Hold sample for gradation analysis

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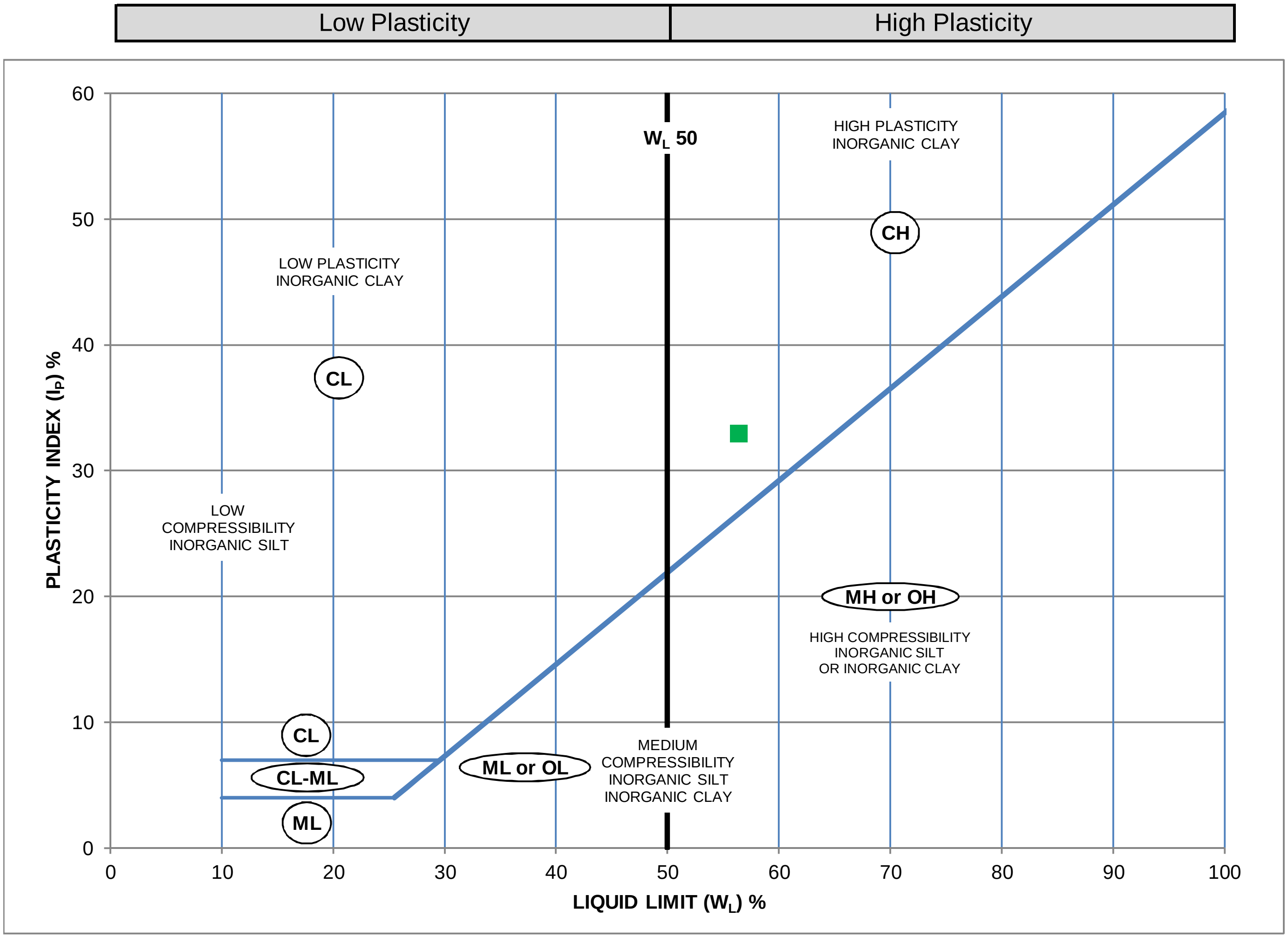
Plasticity Chart

Project Number: *7019-001*
Client: *Geo Firma Engineering Limited*
Project Name: *Geofirma Lab Testing*
Sampled By: *Client*
Sample Date: *June 5, 2018*

Hole No.: *B18-01-3*
Depth: *1.5 m to 2.1 m*
Lab Sample No: *S-18-0560*



Symbol	Borehole	Sample	Depth	Description
🟩	B18-01-3	-	1.5 m to 2.1 m	Clay and Silt

Liquid Limit (%)	Plastic Limit	Plasticity Index (%)
56.5	***23.6***	***32.9***

Issued By: ______________________

(Senior Project Manager)

Date Issued: *June 26, 2018*

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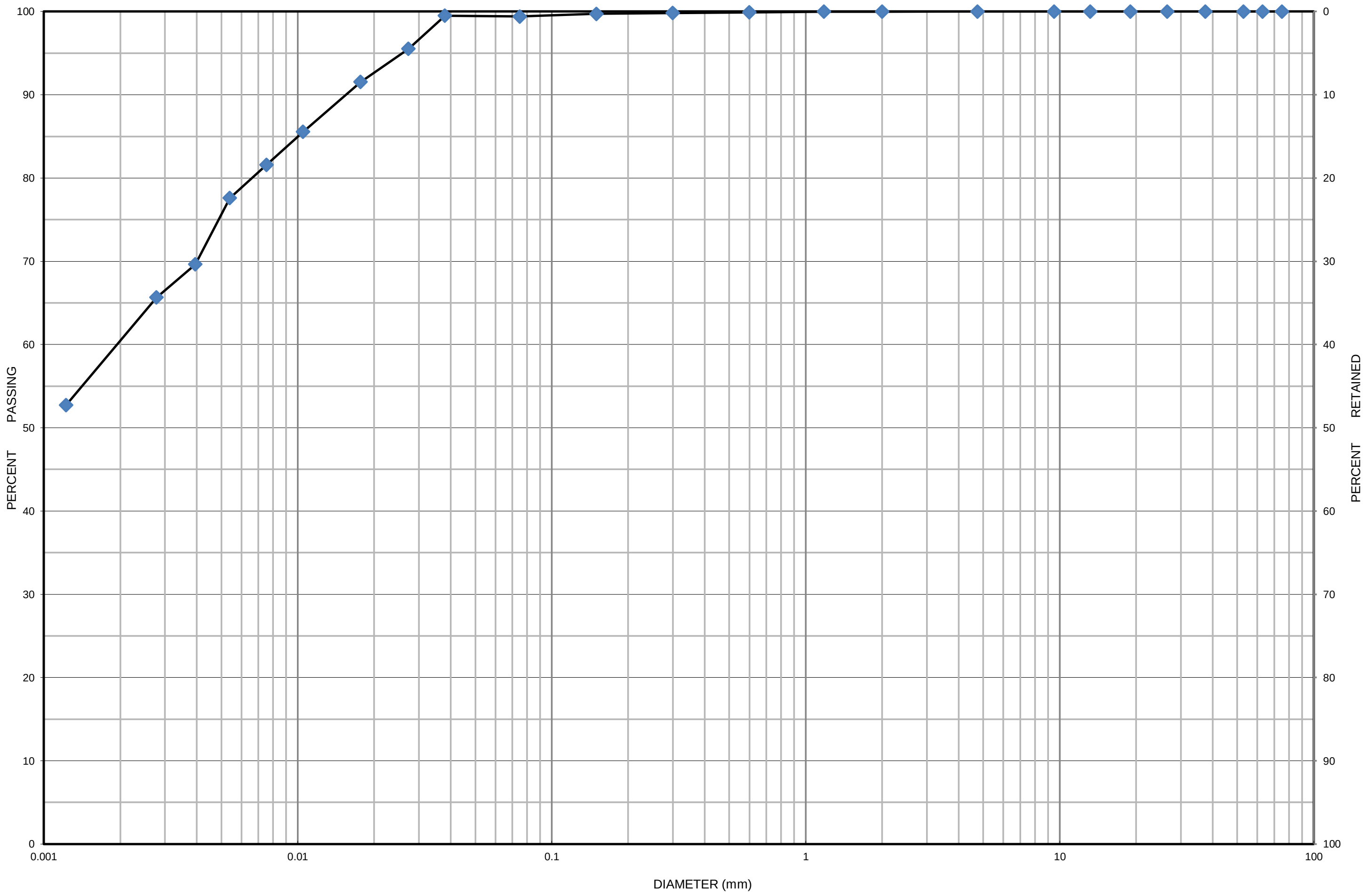
Grain Size Distribution Chart

Project Number: *7019-001* **Client:** *Geo Firma Engineering Limited*
Project Name: *Geofirma Lab Testing*
Sample Date: *June 5, 2018* **Sampled By:** *Client*
Location: *BH18-01-3* **Depth:** *1.5 m to 2.1 m* **Lab Sample No:** *S-18-0560*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Location	Depth	Gravel	Sand	Silt	Clay	Moisture
BH18-01-3	1.5 m to 2.1 m	0	1	99		31.4

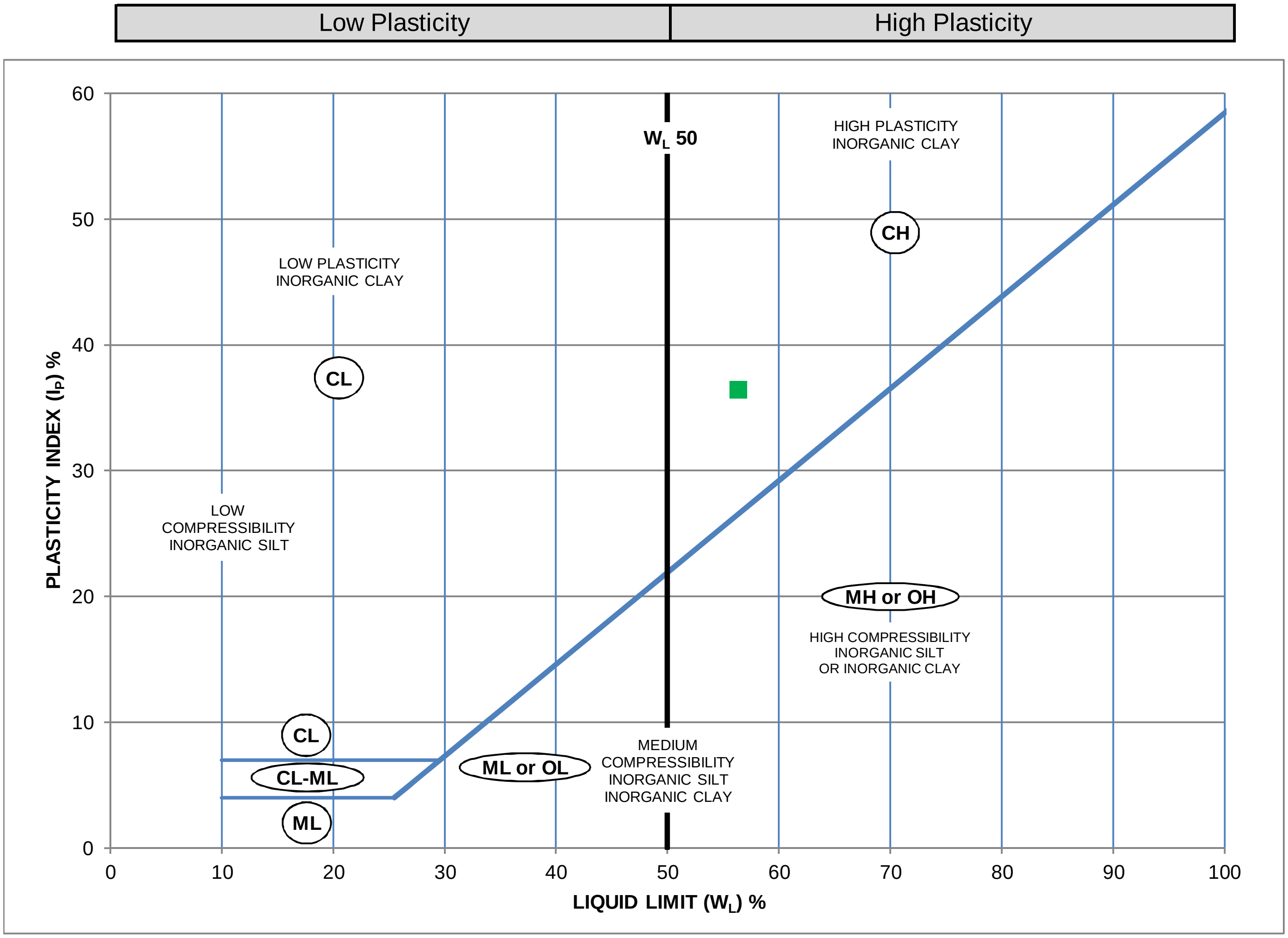
Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Clay and Silt	CH-MH	0.0019	-	-	-	-

Issued By: ____________________ *Date Issued:* *July 3, 2018*
(Senior Project Manager)

Plasticity Chart

Project Number: *7019-001*
Client: *Geo Firma Engineering Limited*
Project Name: *Geofirma Lab Testing*
Sampled By: *Client*
Sample Date: *June 5, 2018*

Hole No.: *B18-03-7*
Depth: *4.6 m to 5.2 m*
Lab Sample No: *S-18-0561*



Symbol	Borehole	Sample	Depth	Description
🟩	B18-03-7	-	4.6 m to 5.2 m	Clay and Silt

Liquid Limit (%)	Plastic Limit	Plasticity Index (%)
56.4	***20.0***	***36.4***

Issued By: ______________________

(Senior Project Manager)

Date Issued: *June 26, 2018*

Grain Size Distribution Chart

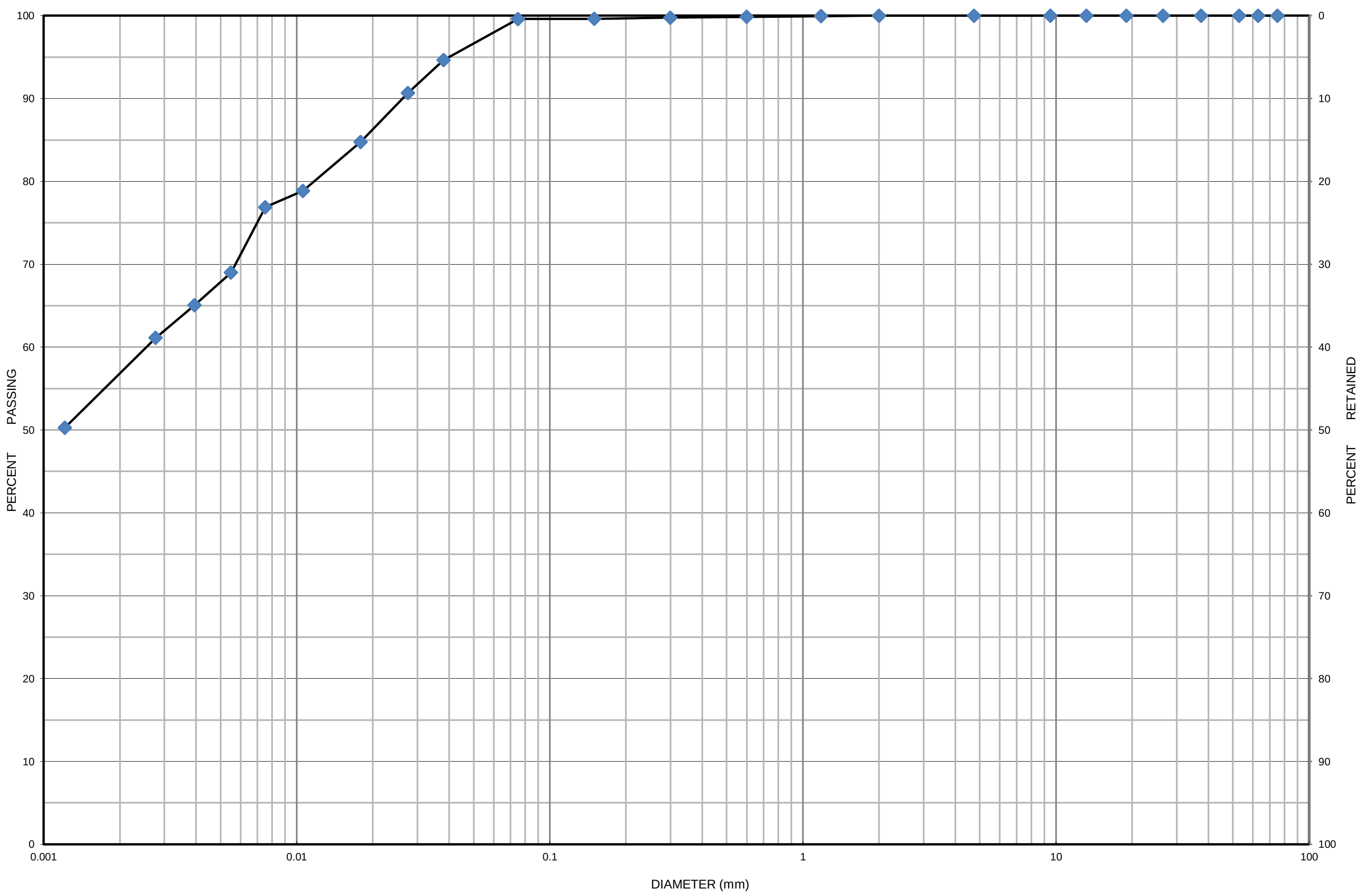
Project Number: *7019-001* **Client:** *Geo Firma Engineering Limited*

Project Name: *Geofirma Lab Testing*

Sample Date: *June 5, 2018* **Sampled By:** *Client*

Location: *BH18-03-7* **Depth:** *4.6 m to 5.2 m* **Lab Sample No:** *S-18-0561*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Location	Depth	Gravel	Sand	Silt	Clay	Moisture
BH18-03-7	4.6 m to 5.2 m	0	0	100		47.6

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Clay and Silt	CH-MH	0.0027	-	-	-	-

Issued By: ______________________ *Date Issued:* *July 3, 2018*

(Senior Project Manager)

Plasticity Chart

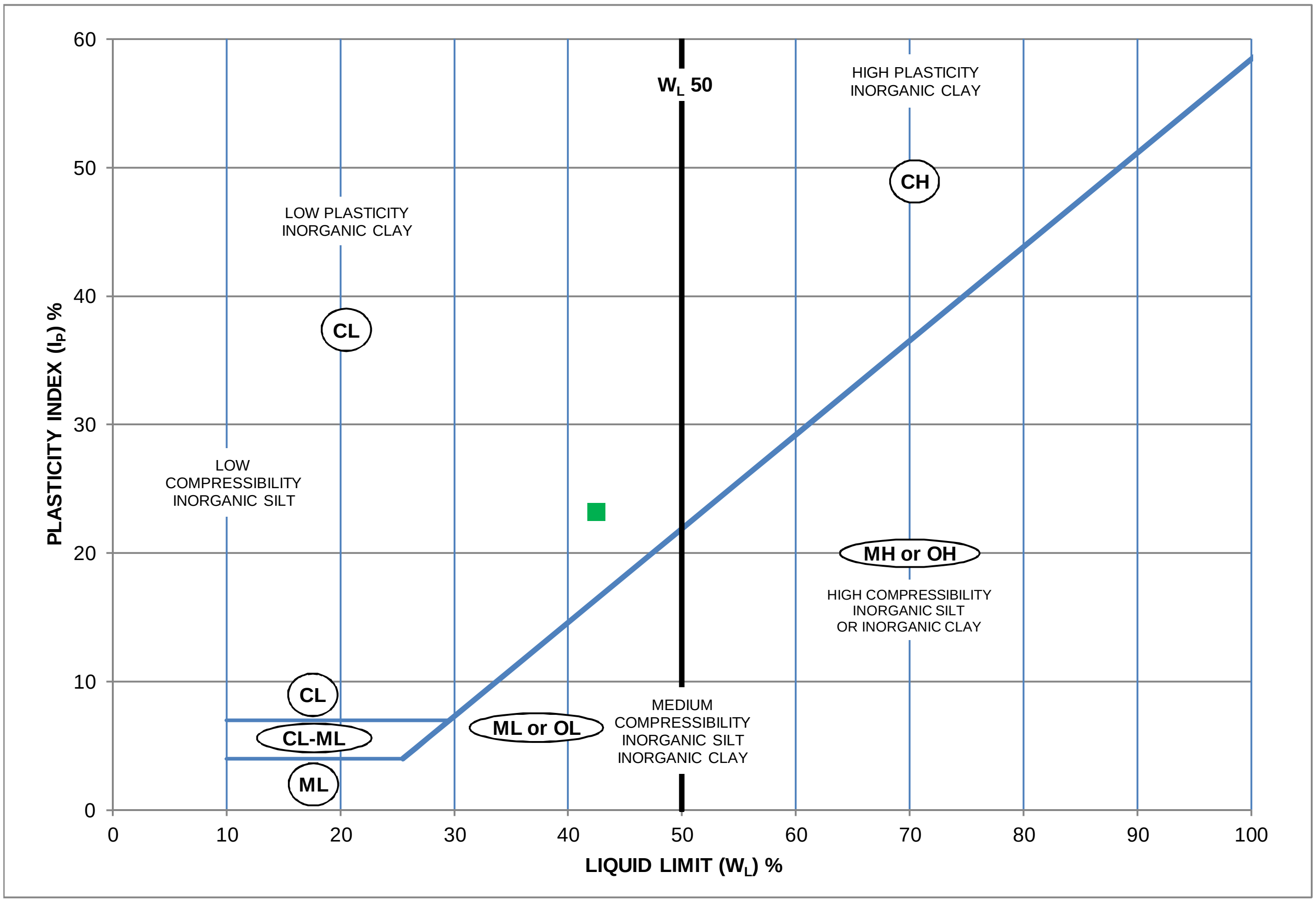
Project Number: *7019-001* **Client:** *Geo Firma Engineering Limited*

Project Name: *Geofirma Lab Testing*

Sampled By: *Client* **Sample Date:** *June 5, 2018*

Hole No.: *B18-04-8* **Depth:** *5.5 m to 5.9 m* **Lab Sample No:** *S-18-0562*

Low Plasticity	High Plasticity



Symbol	Borehole	Sample	Depth	Description
🟩	B18-04-8	-	5.5 m to 5.9 m	Clay and Silt

Liquid Limit (%)	Plastic Limit	Plasticity Index (%)
42.6	***19.4***	***23.1***

Issued By: _______________ *Date Issued:* *June 26, 2018*

(Senior Project Manager)

CAMBIUM

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Grain Size Distribution Chart

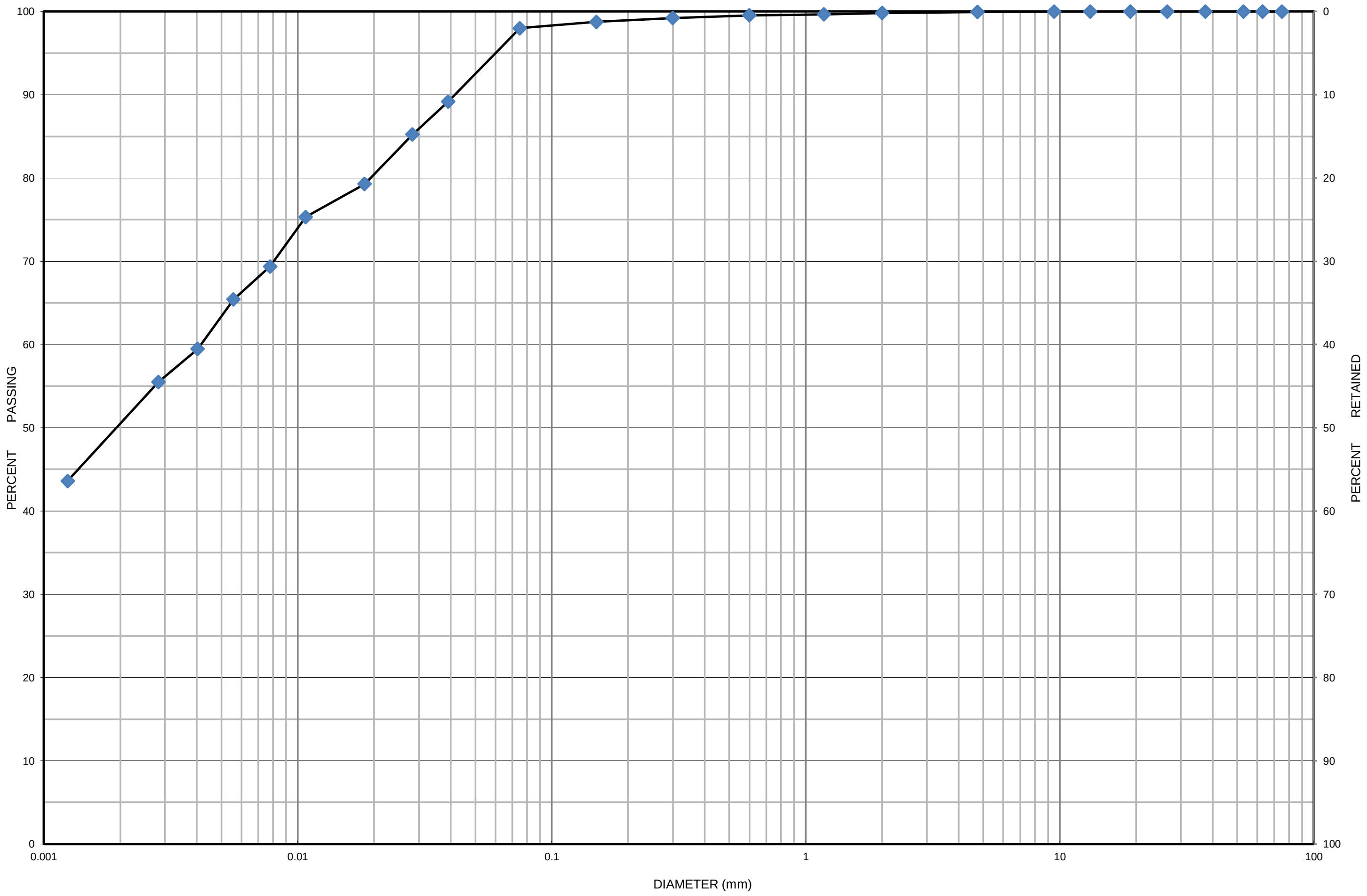
Project Number: *7019-001* **Client:** *Geo Firma Engineering Limited*

Project Name: *Geofirma Lab Testing*

Sample Date: *June 5, 2018* **Sampled By:** *Client*

Location: *BH18-04-8* **Depth:** *5.5 m to 5.9 m* **Lab Sample No:** *S-18-0562*

<table>
<tr><th colspan="6">UNIFIED SOIL CLASSIFICATION SYSTEM</th></tr>
<tr><td rowspan="2">CLAY & SILT (<0.075 mm)</td><td colspan="3">SAND (<4.75 mm to 0.075 mm)</td><td colspan="2">GRAVEL (>4.75 mm)</td></tr>
<tr><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>COARSE</td></tr>
</table>



<table>
<tr><th colspan="9">MIT SOIL CLASSIFICATION SYSTEM</th></tr>
<tr><td rowspan="2">CLAY</td><td rowspan="2">SILT</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td rowspan="2">BOULDERS</td></tr>
<tr><td colspan="3">SAND</td><td colspan="3">GRAVEL</td></tr>
</table>

Location	Depth	Gravel	Sand	Silt	Clay	Moisture
BH18-04-8	5.5 m to 5.9 m	0	2	98		46.8

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Clay and Silt	CL-ML	0.0042	-	-	-	-

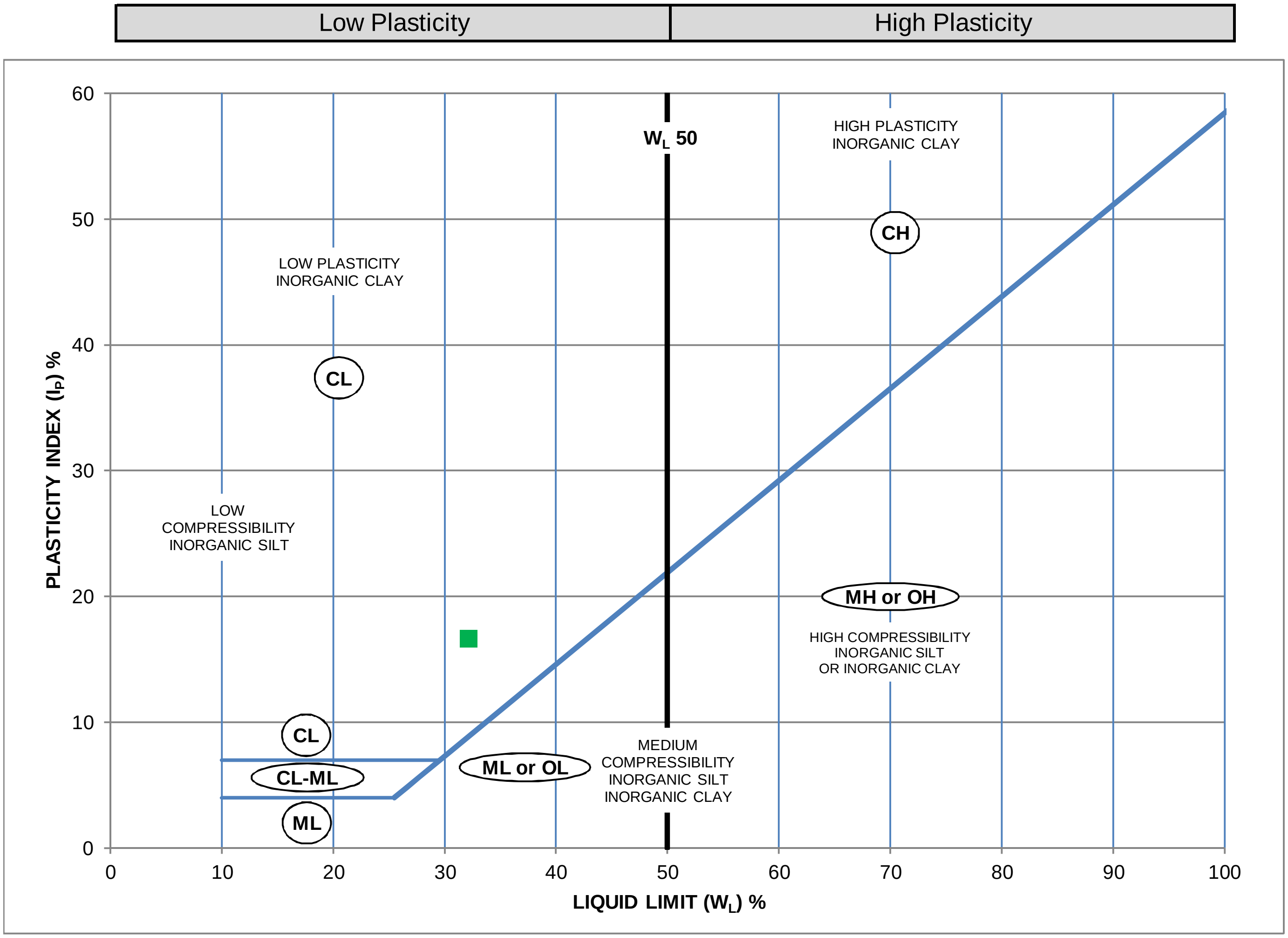
Issued By: ______________________ *Date Issued:* *July 2, 2018*

(Senior Project Manager)

Plasticity Chart

Project Number: *7019-001* **Client:** *Geo Firma Engineering Limited*
Project Name: *Geofirma Lab Testing*
Sampled By: *Client* **Sample Date:** *June 5, 2018*

Hole No.: *BH18-05-8* **Depth:** **Lab Sample No:** *S-18-0563*



Symbol	Borehole	Sample	Depth	Description
🟩	BH18-05-8	-	-	Clayey Silt

Liquid Limit (%)	Plastic Limit	Plasticity Index (%)
32.2	***15.6***	***16.6***

Issued By: ______________________ *Date Issued:* *June 26, 2018*

(Senior Project Manager)

CAMBIUM

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CCIL
Canadian Council of Independent Laboratories
For specific tests as listed on www.ccil.com

Grain Size Distribution Chart

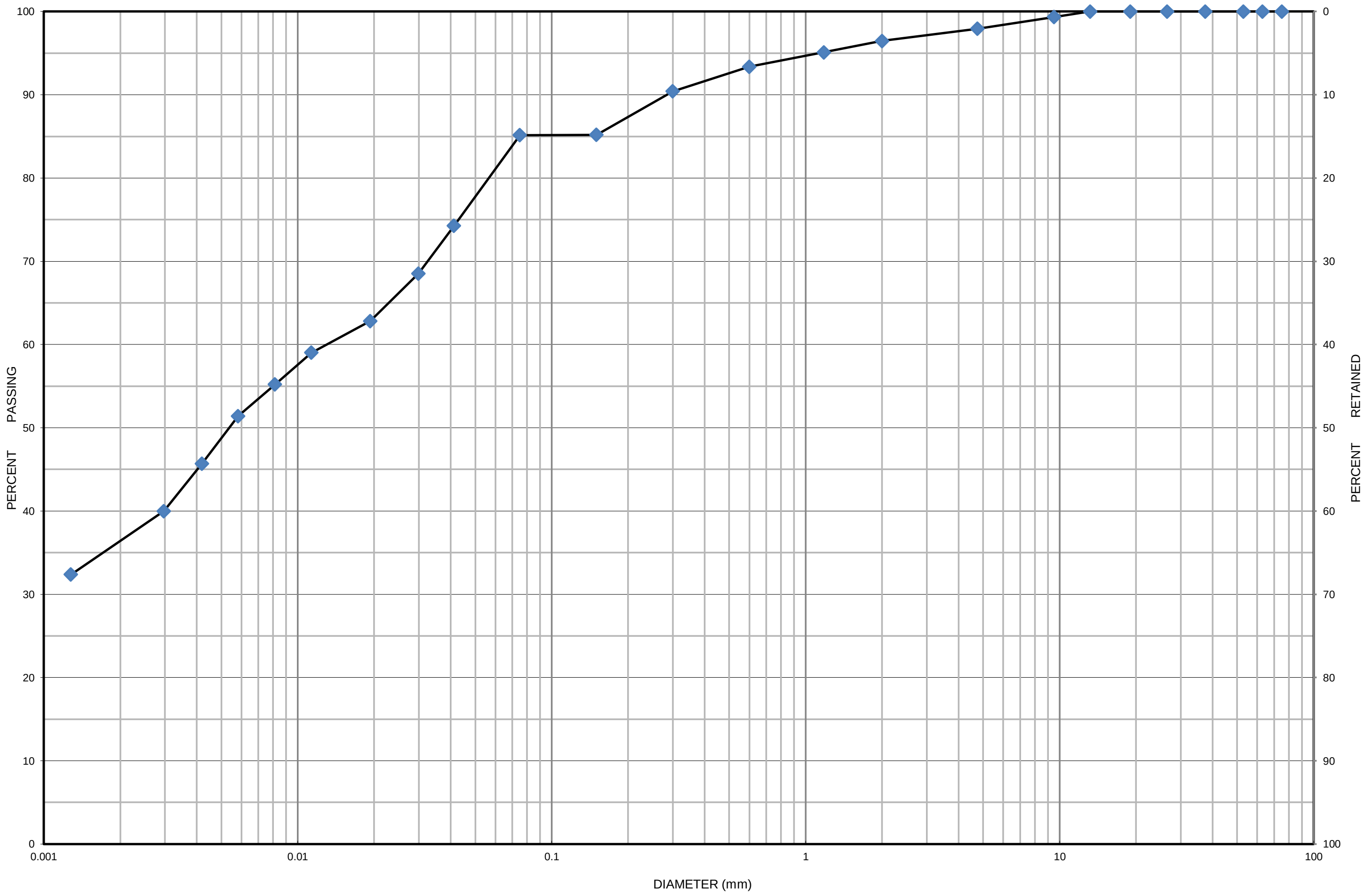
Project Number: *7019-001* **Client:** *Geo Firma Engineering Limited*

Project Name: *Geofirma Lab Testing*

Sample Date: *June 5, 2018* **Sampled By:** *Client*

Location: *BH18-05-8* **Depth:** *5.3 m to 5.9 m* **Lab Sample No:** *S-18-0563*

<table>
<tr><th colspan="6">UNIFIED SOIL CLASSIFICATION SYSTEM</th></tr>
<tr><td rowspan="2">CLAY & SILT (<0.075 mm)</td><td colspan="3">SAND (<4.75 mm to 0.075 mm)</td><td colspan="2">GRAVEL (>4.75 mm)</td></tr>
<tr><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>COARSE</td></tr>
</table>



<table>
<tr><th colspan="9">MIT SOIL CLASSIFICATION SYSTEM</th></tr>
<tr><td rowspan="2">CLAY</td><td rowspan="2">SILT</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td rowspan="2">BOULDERS</td></tr>
<tr><td colspan="3">SAND</td><td colspan="3">GRAVEL</td></tr>
</table>

Location	Depth	Gravel	Sand	Silt	Clay	Moisture
BH18-05-8	5.3 m to 5.9 m	2	13	85		30.6

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Clayey Silt some Sand trace Gravel	ML	0.014	-	-	-	-

Issued By: ______________________ *Date Issued:* *July 2, 2018*

(Senior Project Manager)

Cambium Inc. (Laboratory)
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701 The Queensway | Units 5-6 | Peterborough | ON | K9J 7J6

Form: L6V.2 - Grad.Hydo

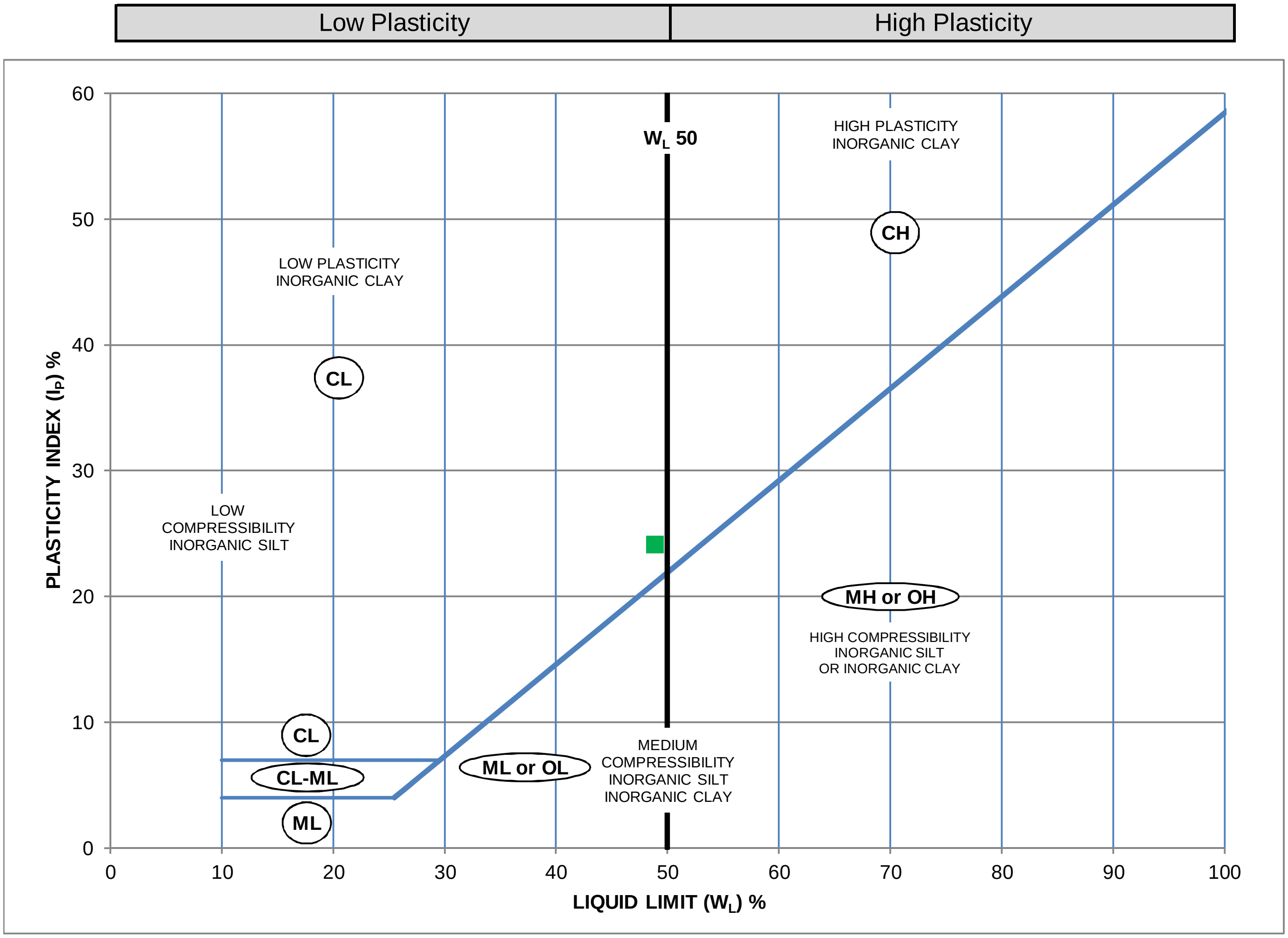
CAMBIUM

CERTIFIED BY CCIL

Plasticity Chart

Project Number:	*7019-001*	**Client:**	*Geo Firma Engineering Limited*
Project Name:	*Geofirma Lab Testing*		
Sampled By:	*Client*	**Sample Date:**	*June 5, 2018*

Hole No.: *B18-08-5* **Depth:** *3 m to 3.7 m* **Lab Sample No:** *S-18-0564*



Symbol	Borehole	Sample	Depth	Description
🟩	B18-08-5	-	3 m to 3.7 m	Clay and Silt

Liquid Limit (%)	Plastic Limit	Plasticity Index (%)
48.9	***24.9***	***24.1***

Issued By: ______________________ *Date Issued:* *June 27, 2018*

(Senior Project Manager)

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Grain Size Distribution Chart

Project Number: *7019-001* **Client:** *Geo Firma Engineering Limited*

Project Name: *Geofirma Lab Testing*

Sample Date: *May 28-29, 2018* **Sampled By:** *Client*

Location: *BH18-08-5* **Depth:** *3 m to 3.7 m* **Lab Sample No:** *S-18-0564*

<table>
<tr><th colspan="6">UNIFIED SOIL CLASSIFICATION SYSTEM</th></tr>
<tr><td rowspan="2">CLAY & SILT (<0.075 mm)</td><td colspan="3">SAND (<4.75 mm to 0.075 mm)</td><td colspan="2">GRAVEL (>4.75 mm)</td></tr>
<tr><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>COARSE</td></tr>
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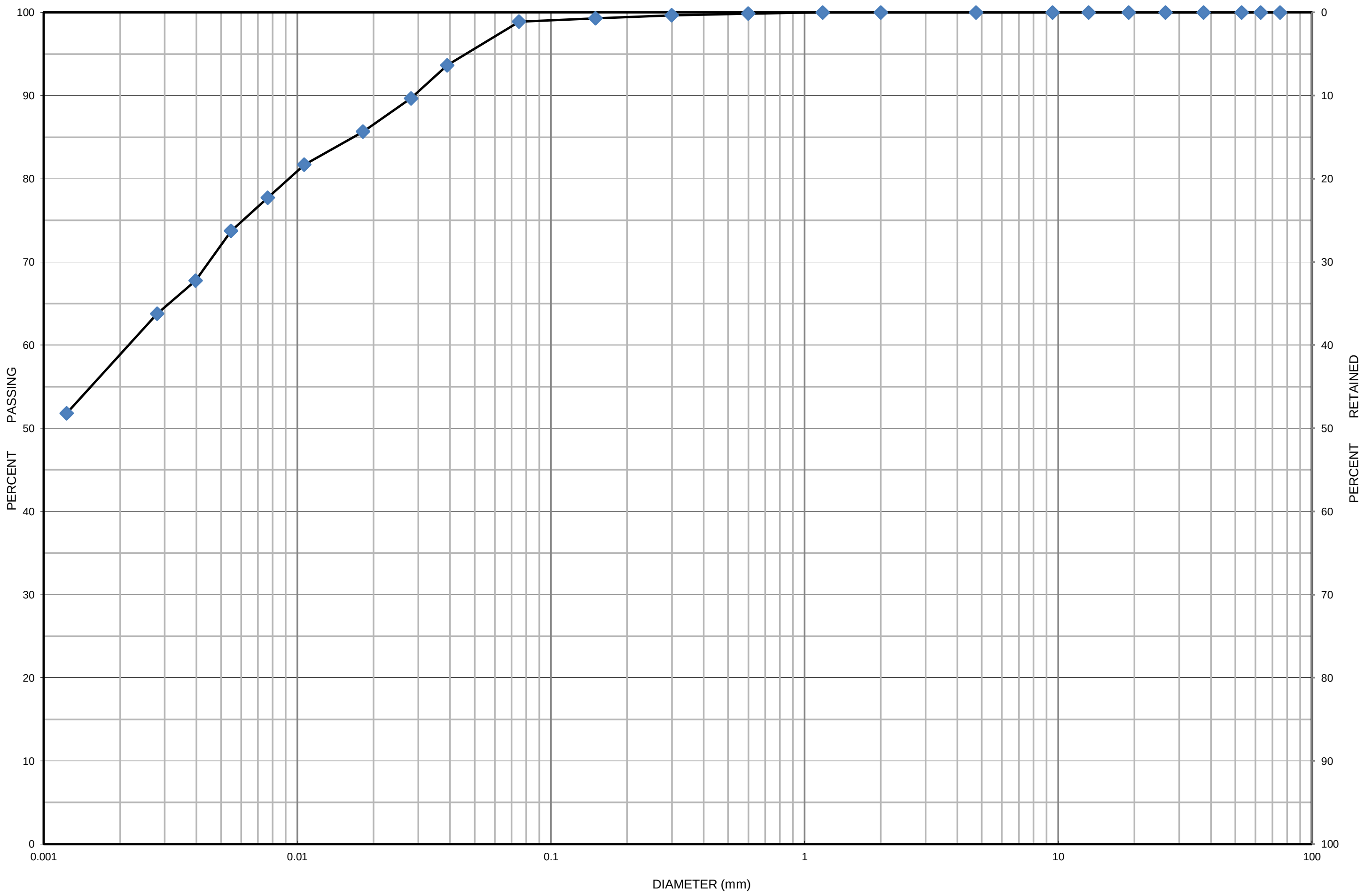

<table>
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<tr><td rowspan="2">CLAY</td><td rowspan="2">SILT</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td rowspan="2">BOULDERS</td></tr>
<tr><td colspan="3">SAND</td><td colspan="3">GRAVEL</td></tr>
</table>

Location	Depth	Gravel	Sand	Silt	Clay	Moisture
BH18-08-5	3 m to 3.7 m	0	1	99		44.6

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Clay and Silt	CL-ML	0.0022	-	-	-	-

Issued By: ______________________ *Date Issued:* *July 2, 2018*

(Senior Project Manager)

CAMBIUM

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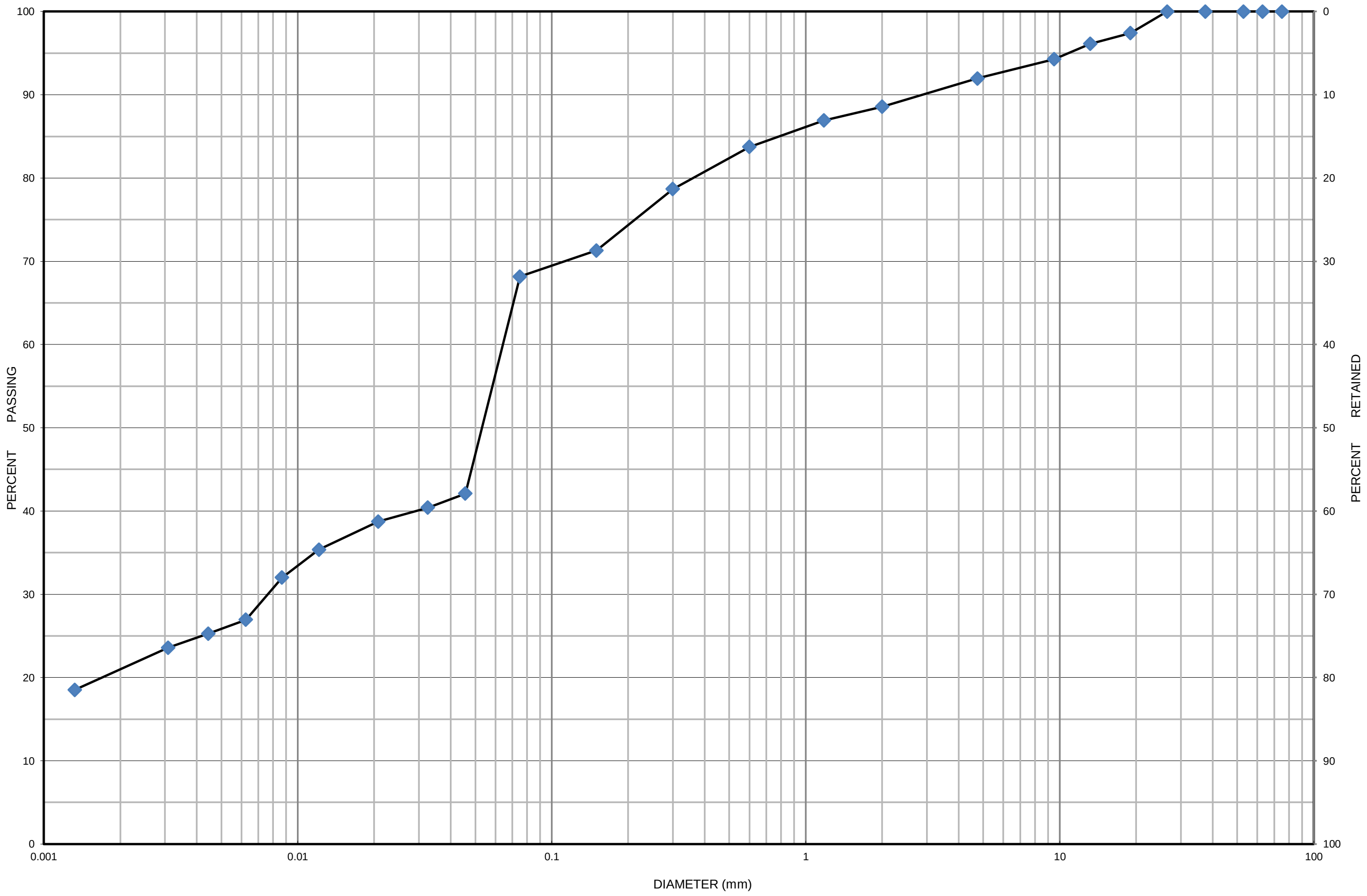
Grain Size Distribution Chart

Project Number:	*7019-001*	**Client:**	*Geo Firma Engineering Limited*		
Project Name:	*Geofirma Lab Testing*				
Sample Date:	*June 5, 2018*	**Sampled By:**	*Client*		
Location:	*BH18-08-7*	**Depth:**	*4.7 m to 5.3 m*	**Lab Sample No:**	*S-18-0565*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Location	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH18-08-7		4.7 m to 5.3 m	8	24	68		22.2

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Clayey Sandy Silt trace Gravel	ML	0.064	0.0077	-	-	-

Issued By: ______________________ *Date Issued:* *July 2, 2018*

(Senior Project Manager)

Cambium Inc. (Laboratory)
866.217.7900 | cambium-inc.com
701 The Queensway | Units 5-6 | Peterborough | ON | K9J 7J6

Form: L6V.2 - Grad.Hydo

GEMTEC Consulting Engineers and
Scientists Limited
32 Steacie Drive
Ottawa, ON
K2K 2A9
Tel.: 613-836-1422
Fax.:613-836-9731

COMPRESSIVE STRENGTH
of ROCK CORE

CLIENT: Geofirma Engineering Ltd. **PROJECT No.:** 62649.02

Project: March Road **REPORT NO:** 1

Date Received: 12-Jun-18 **Date Tested:** 13-Jun-18

Lab no.	1807	1808				
Core ID	18-01	18-05				
Depth (m)	5.74	5.94				
Cut length (mm)	128.22	130.44				
Ground length (mm)	125.15	126.49				
Diameter (mm)	62.92	63.21				
Ground Mass (g)	1044.00	1022.00				
Length:Diameter ratio	1.99	2.00				
Correction factor	1.00	1.00				
Failure load (kN)	570.92	252.84				
Uncorrected Strength (MPa)	183.60	80.60				
Corrected Strength (MPa)	**183.60**	**80.60**				

Remarks

Checked by: Krystle Smith, Laboratory Manager

Reviewed by: Steve Goodman, Ph.D., P.Eng.

Certificate of Analysis

Geofirma Engineering Ltd.

Suite 200, 1 Raymond St.
Ottawa, ON K1R 1A2
Attn: Steve Gaines

Client PO:
Project: 18-206-2
Custody:

Report Date: 13-Jun-2018
Order Date: 8-Jun-2018

Order #: 1823750

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

Paracel ID	Client ID
1823750-01	BH18-04-7
1823750-02	BH18-07-5

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	12-Jun-18	12-Jun-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	11-Jun-18	11-Jun-18
Resistivity	EPA 120.1 - probe, water extraction	12-Jun-18	12-Jun-18
Solids, %	Gravimetric, calculation	12-Jun-18	12-Jun-18

Certificate of Analysis

Client: Geofirma Engineering Ltd.

Client PO:

Report Date: 13-Jun-2018

Order Date: 8-Jun-2018

Project Description: 18-206-2

	Client ID:	BH18-04-7	BH18-07-5	-	-
	Sample Date:	06/06/2018 09:00	06/07/2018 09:00	-	-
	Sample ID:	1823750-01	1823750-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	58.6	73.9	-	-
General Inorganics					
pH	0.05 pH Units	7.75	7.64	-	-
Resistivity	0.10 Ohm.m	8.16	35.6	-	-
Anions					
Chloride	5 ug/g dry	783	143	-	-
Sulphate	5 ug/g dry	175	41	-	-

Method Quality Control: Blank

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

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	184	5	ug/g dry	176			4.1	20	
Sulphate	161	5	ug/g dry	148			8.3	20	
General Inorganics									
pH	7.89	0.05	pH Units	7.91			0.3	10	
Resistivity	113	0.10	Ohm.m	107			5.8	20	
Physical Characteristics									
% Solids	94.2	0.1	% by Wt.	93.2			1.1	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	269	5	ug/g	176	92.6	78-113			
Sulphate	244	5	ug/g	148	96.7	78-111			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Paracel ID: 1823750

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1-800-749-1947
paracel@paracellabs.com

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(Lab Use Only)

Page 1 of 1

Client Name: Geofirma Engineering Ltd
Contact Name: Steve Gaines
Address: 1 Raymond St., Ottawa, ON
Telephone: 613-658-0187

Project Reference: 15-206-2
Quote #
PO #
Email Address: sgaines@geofirma.com

Turnaround Time:
☐ 1 Day ☐ 3 Day
☐ 2 Day ☑ Regular
Date Required: ____________

Criteria: ☑ O. Reg. 153/04 (As Amended) Table 1 ☐ 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: 1823750

Required Analyses

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	ph	resistivity	sulphate	chloride											
1	BH18-04-7	Soil		1	6-Jun-18		☑	☑	☑	☑	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
2	BH18-07-5	soil		1	7-Jun-18		☑	☑	☑	☑	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
3							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
4							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
5							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
6							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
7							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
8							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
9							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
10							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:

Method of Delivery:

Relinquished By (Sign):
Relinquished By (Print): Steve Gaines
Date/Time: 8-Jun-18 / 17:00

Received by Driver/Depot: A Stone
Date/Time: 08/06/18 3:41
Temperature: ______ °C PM.

Received at Lab: KRG
Date/Time: JUNE 8, 18 4 17
Temperature: 38.5 °C

Verified By: KRG
Date/Time: JUNE 8, 18 4 17
pH Verified [ ] By: ________

Chain of Custody (Blank) - Rev 0.4 Feb 2016

Certificate of Analysis

Geofirma Engineering Ltd.

Suite 200, 1 Raymond St.
Ottawa, ON K1R 1A2
Attn: Steve Gaines

Client PO:
Project: 18-206-2
Custody:

Report Date: 25-Jun-2018
Order Date: 19-Jun-2018

Order #: 1825310

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

Paracel ID	Client ID
1825310-01	MW18-01

Approved By:

Dale Robertson, BSc
Laboratory Director

Certificate of Analysis
Client: Geofirma Engineering Ltd.
Client PO:

Report Date: 25-Jun-2018
Order Date: 19-Jun-2018
Project Description: 18-206-2

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC	21-Jun-18	21-Jun-18
pH	EPA 150.1 - pH probe @25 °C	21-Jun-18	21-Jun-18
Resistivity	EPA 120.1 - probe	22-Jun-18	22-Jun-18

Certificate of Analysis

Client: Geofirma Engineering Ltd.

Client PO:

Report Date: 25-Jun-2018

Order Date: 19-Jun-2018

Project Description: 18-206-2

	Client ID:	MW18-01	-	-	-
	Sample Date:	06/15/2018 12:00	-	-	-
	Sample ID:	1825310-01	-	-	-
	MDL/Units	Water	-	-	-
General Inorganics					
pH	0.1 pH Units	7.7	-	-	-
Resistivity	0.01 Ohm.m	14.2	-	-	-
Anions					
Chloride	1 mg/L	115	-	-	-
Sulphate	1 mg/L	41	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	1	mg/L						
Sulphate	ND	1	mg/L						
General Inorganics									
Resistivity	ND	0.01	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	106	1	mg/L	103			3.3	10	
Sulphate	370	1	mg/L	364			1.5	10	
General Inorganics									
pH	8.0	0.1	pH Units	8.0			0.3	10	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	111	1	mg/L	103	89.4	78-112			
Sulphate	26.1	1	mg/L	16.7	94.3	75-111			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

PARACEL
LABORATORIES LTD.

Paracel ID: 1825310

ad Office
0-2319 St. Laurent Blvd
awa, Ontario K1G 4J8
1-800-749-1947
paracel@paracellabs.com

Chain of Custody
(Lab Use Only)

Client Name: GEOFIRMA ENG LTD.

Contact Name: Steve Gaines

Address: 1 Raymond

Telephone: 613-858-0187

Project Reference: 18-206-2

Quote #

PO #

Email Address: sgaines@geofirma.com

Page 1 of 1

Turnaround Time:

☐ 1 Day ☐ 3 Day

☐ 2 Day ☒ Regular

Date Required:

Criteria: ☒ O. Reg. 153/04 (As Amended) Table 1 ☐ 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: 1825310

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	**Sample Taken** Date	**Sample Taken** Time	pH	Resistivity	Sulphate	Chloride							
1	MW18-01	GW		1	15-JUN-18	pm	☒	☒	☒	☒	☐	☐	☐	☐	☐	☐	☐
2							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
3							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
4							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
5							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
6							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
7							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
8							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
9							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
10							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments:

Method of Delivery: Paracel

Relinquished By (Sign):

Relinquished By (Print): STEVE GAINES

Date/Time: 18-JUN-18 / 1420

Received by Driver/Depot: Trouse

Date/Time: 19/06/18 3:25 PM

Temperature: °C

Received at Lab: SUNPOORN DOKMA

Date/Time: JUN 19, 2018 05.13

Temperature: 15.0 °C

Verified By:

Date/Time: Jun 19/18 7:20 pm

pH Verified [ ] By:

Chain of Custody (Blank) - Rev 0.4 Feb 2016

APPENDIX F

Shear Wave Velocity Report (prepared by Geophysics GPR International Inc.)

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7

Tel. : (450) 679-2400
Fax : (514) 521-4128
info@geophysicsgpr.com
www.geophysicsgpr.com

June 13th, 2018

Transmitted by email: sgaines@geofirma.com
Our Ref.: GPR-18-00588

Mr. Steve Gaines, M.A.Sc., P.Eng., P.Geo.
Geological Engineer
Geofirma Engineering Ltd
1 Raymond Street, Suite 200
Ottawa (ON) K1R 1A2

Subject: **Shear Wave Velocity Sounding for Site Class Determination**
788 March Road, Kanata, Ottawa (ON)

[Project: 18-206-2]

Dear Sir,

Geophysics GPR International inc. has been requested by Geofirma Engineering Ltd to carry out seismic shear wave surveys on a vacant field, located at 788 March Road, Kanata, in Ottawa (ON). The geophysical investigations used the Multi-channel Analysis of Surface Waves (MASW), the Extended SPatial AutoCorrelation (ESPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocities values were calculated for the soil and the rock.

The surveys were carried out, on June 8th, by Mr. Alexis Marchand and Mrs. Chloé Gingras, trainee. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the test methods, and the results in graphic and table format.

METHODS PRINCIPLES

MASW Survey

The *Multi-channel Analysis of Surface Waves* (MASW) and the *Extended SPatial AutoCorrelation* (ESPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface waves (“ground roll”). The MASW is considered an “active” method, as the seismic signal is induced at known location and time in the geophones spread axis. Conversely, the ESPAC is considered a “passive” method, using the low frequency “noises” produced far away. The method can also be used with “active” seismic source records. The dispersion properties are expressed as a change of phase velocities with frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding). Figure 3 schematically outlines the basic operating procedure for the MASW method.

Figure 4 illustrates an example of one of the MASW/ESPAC records, the corresponding spectrogram analysis and resulting 1D V_S model. The ESPAC method allows deeper Vs soundings, but generally with a lower resolution for the surface portion. Its dispersion curve can then be merged with the higher frequency one from the MASW to calculate a more complete inversion.

Seismic Refraction Survey

The method consists in measuring the propagation delays of the direct and refracted seismic waves (P and/or S) produced by an artificial source in the axis of a seismic linear spread. The seismic velocities of the materials can be directly calculated, then the refractors depths.

INTERPRETATION METHODS

MASW Surveys

The main processing sequence involved data inspection and edition when required; spectral analysis (“phase shift” for MASW, and “cross-correlation” for ESPAC); picking the fundamental mode; and 1D inversion of the MASW and ESPAC shot records using the SeisImagerSW™ software. The data inversions used a nonlinear least squares algorithm.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_S) is of the order of 15% or better.

Seismic Refraction surveys

The considered seismic wave's arrival times were identified for each geophone. The General Reciprocal Method was used, with signal sources at both ends of the seismic spreads, to consider seismic wave propagation for two opposite directions. The measurements were realised to calculate the rock depth, and its seismic velocity (using P waves). The rock seismic velocities (V_S) were calculated using two methods: the reduced travel-times (the Hobson and Overton method) and the opposite apparent velocities. The first one allows independence from the surface and rock topography effect, as well as the overburden lateral variation of its seismic velocity, but remains limited to common geophones. Its application remains however limited to shallow to intermediate depths refractors. The second one can use longer segments of opposite directions signals, improving the linear regressions accuracy, but remains affected by the surface and rock topography effect, as well as the overburden lateral variation of the seismic velocity. Conversely to the MASW method, the seismic rock velocity calculated by seismic refraction is only representative of its superior part, due to the evanescent nature of the refracted wave.

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015

SURVEY DESIGN

The seismic acquisition spreads were located on a vacant field, south-east of the intersection of March Road and Klondike Road. The seismic spread started close to BH/MW18-07, and ended between BH/MW-03 and 04. The geophone spacing for the main spread was of 3 metres, using 24 geophones. Shorter seismic spreads, with geophone spacings of 0.5 and 1 metre, were dedicated to the near surface materials.

The seismic records counted 4096 data, sampled at 1000 μs for the MASW surveys, and 4096 data, sampled at 50 μs for the seismic refraction. The records included a pre-trig portion of 10 ms. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

Unlike the refraction method, which allows producing a result point beneath each geophone, the shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length. The seismic records were made with a seismograph Terraloc MK6 (from ABEM Instrument), and the geophones were 4.5 Hz. A 10 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic spreads.

RESULTS

From seismic refraction surveys, the rock was calculated between 4.5 and 7.4 metres deep (± 1 metre), with an average value between 4.6 and 6.5 metres, and with an apparent dip North-West. Its seismic velocity was calculated between 1565 and 1585 m/s for the upper portion (cf. Figure 5). These results were used as initial parameters for the basic geophysical model, prior to the MASW dispersion curves inversions.

The MASW calculated velocities of the seismic shear wave (V_S) results are illustrated at Figure 6 and the numerical results are presented at Table 1.

The $\overline{V}_{S30}$ value results from the harmonic mean of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface up to 30 metres. This value represents an equivalent homogeneous single layer response.

Considering an average rock depth of 6 metres, the calculated $\overline{V}_{S30}$ value of the actual site is 668.9 m/s (cf. Table 1), corresponding to the Site Class "C". Nevertheless, low seismic velocities were calculated from the surface to approximately 2.5 to 3.5 metres deep.

At least one underground story parking is actually projected, and the Site Classes A and B are not to be used if there is more than 3 metres of unconsolidated material between the rock and the lower portion of the footings. A $\overline{V}_{S30}{}^{*}$ value of 1056.8 m/s was calculated, corresponding Site Class "B", considering less than 3 metres of unconsolidated materials between the rock surface and the lower portion of the footings (cf. Table 2). In the case the footings would be less than 0.3 metre from the rock surface, the Site Class "A" could be considered.

CONCLUSION

Geophysical surveys were carried out on a vacant field, at 788 March road, Kanata, Ottawa (ON). The seismic surveys used the MASW, ESPAC analysis methods, as well as the complementary seismic refraction method, to calculate the $\overline{V}_{S30}$ value for the Site Class determination. The $\overline{V}_{S30}$ calculation is presented in Table 1.

The calculated $\overline{V}_{S30}$ value of the actual site is 669 m/s, corresponding to the Site Class "C" ($360 < \overline{V}_{S30} \leq 760$ m/s), as determined through the MASW, ESPAC and seismic refraction methods, Table 4.1.8.4.A of the NBC, and the Building Code, O. Reg. 332/12.

In the case the rock surface would be less than 3 metres from the lower part of the footings, the $\overline{V}_{S30}{}^*$ value (1057 m/s) would correspond to a Site Class "B" ($760 < \overline{V}_{S30}{}^* \leq 1500$ m/s). Furthermore, in the case the rock surface would be less than 0.3 metre from the lower part of the footings, the $\overline{V}_{S30}{}^*$ value would correspond to a Site Class "A" ($\overline{V}_{S30}{}^* > 1500$ m/s).

It must be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, soft clays, high moisture content etc. can supersede the Site Classification provided in this report based on the $\overline{V}_{S30}$ value.

The V_S values calculated are representative of the in-situ materials and are not corrected for the total and effective stresses.

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Project Manager

Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

Figure 2: Location of the seismic spread
(source: *Google Earth™*)

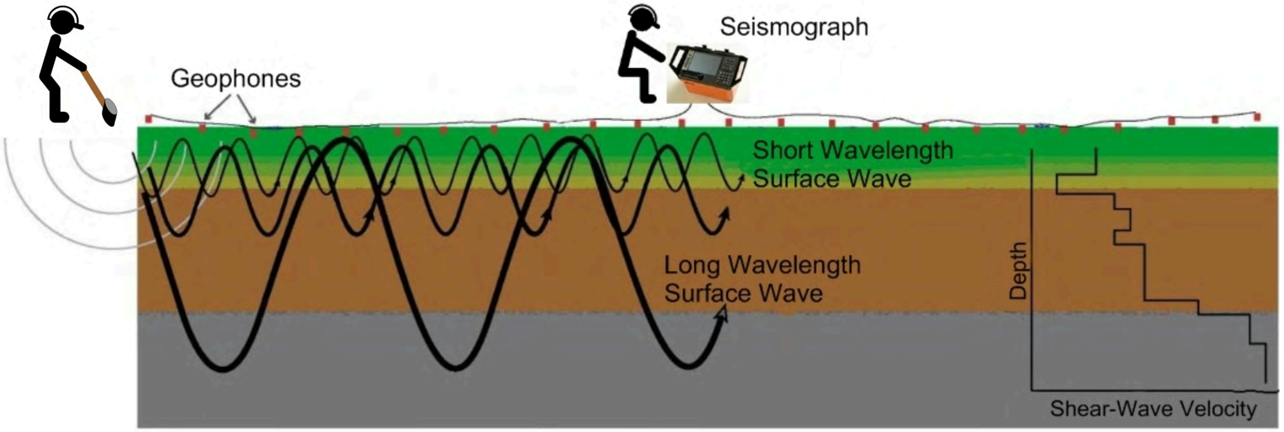
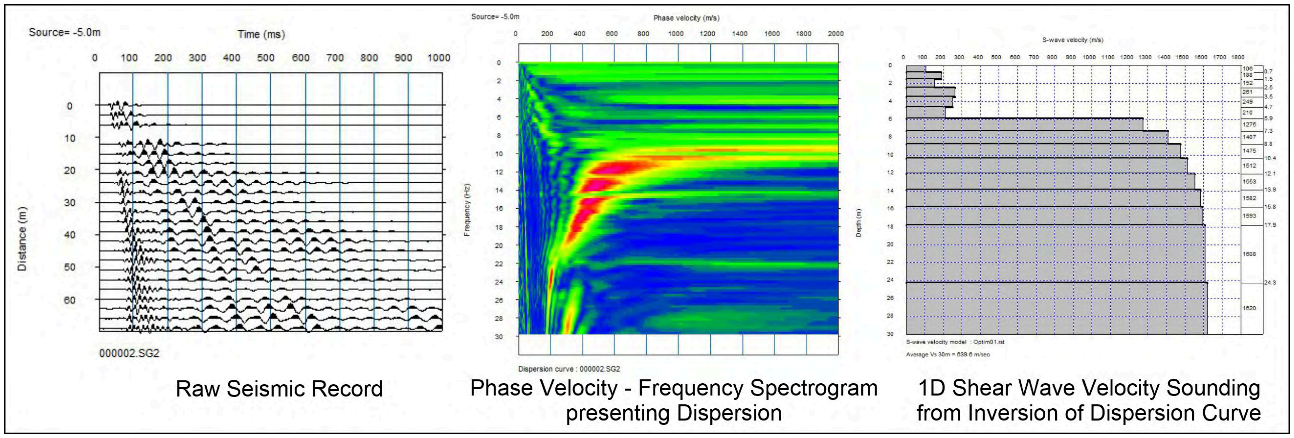


Figure 3: MASW Operating Principle

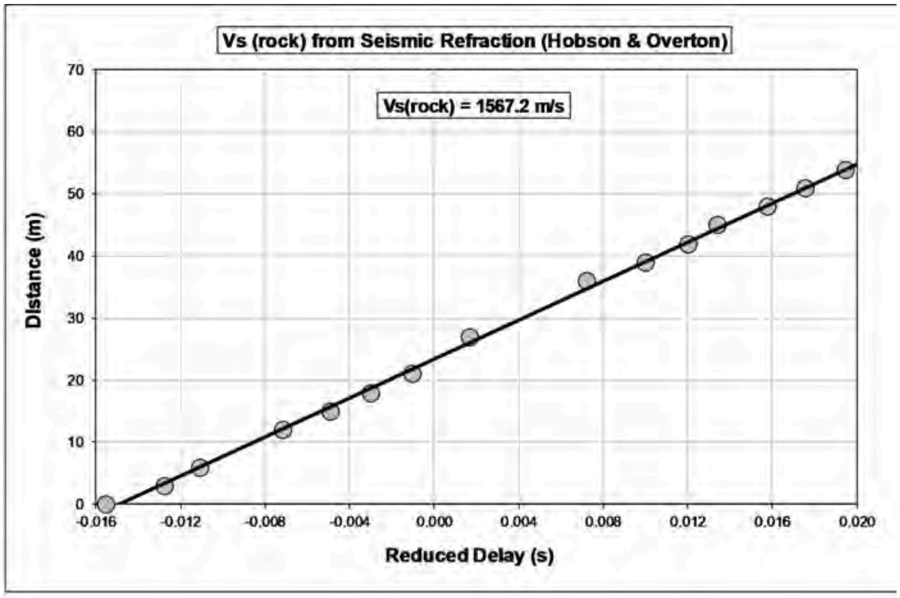


Raw Seismic Record

Phase Velocity - Frequency Spectrogram presenting Dispersion

1D Shear Wave Velocity Sounding from Inversion of Dispersion Curve

Figure 4: Example of a MASW/ESPAC record, Phase Velocity - Frequency curve and resulting 1D Shear Wave Velocity Model



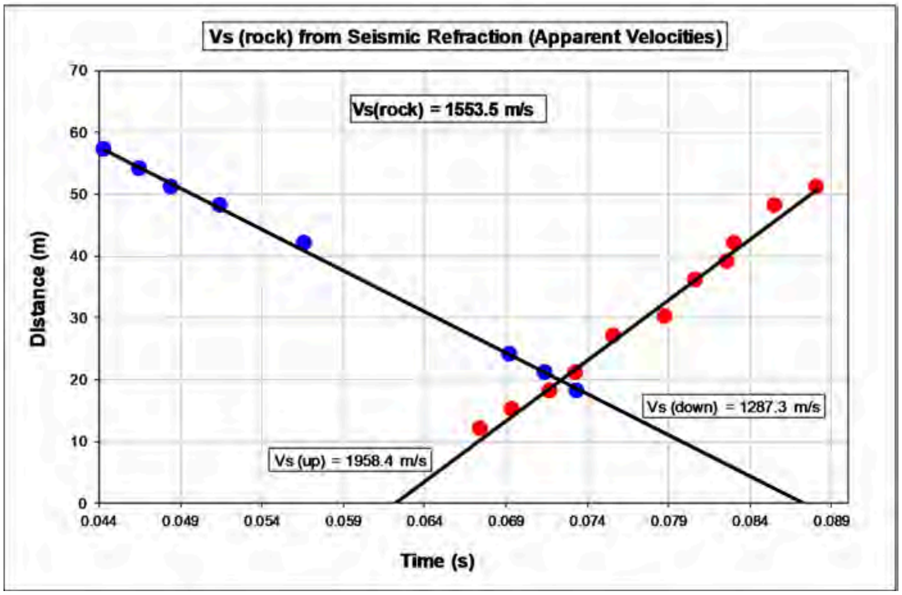


Figure 5: Rock V_S from Seismic Refraction

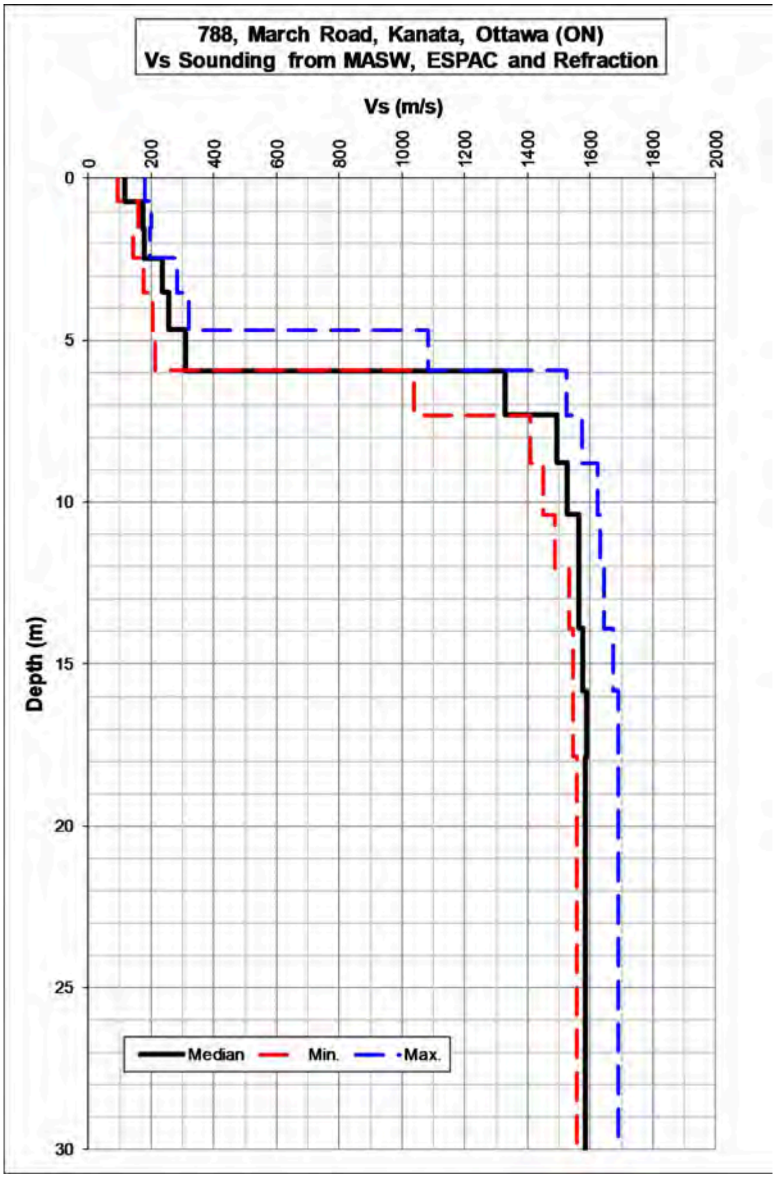


Figure 6: MASW Shear-Wave Velocities Sounding

TABLE 1

V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	**94.1**	**113.8**	180.1					
0.71	**158.0**	**173.1**	201.3	0.71	0.71	0.006276	0.006276	113.8
1.54	**142.1**	**176.9**	196.3	0.82	1.54	0.004761	0.011037	139.4
2.47	**173.3**	235.2	283.2	0.93	2.47	0.005282	0.016318	151.5
3.52	203.4	254.5	319.4	1.04	3.52	0.004438	0.020756	169.4
4.67	211.1	309.0	1082.2	1.15	4.67	0.004533	0.025289	184.7
5.93	1036.9	1327.6	1525.7	1.26	5.93	0.004090	0.029379	202.0
7.31	1409.7	1491.8	1574.6	1.37	7.31	0.001035	0.030414	240.3
8.79	1448.4	1527.2	1622.3	1.48	8.79	0.000994	0.031408	279.9
10.38	1488.7	1563.7	1629.3	1.59	10.38	0.001043	0.032452	320.0
12.09	1533.4	1564.3	1644.0	1.70	12.09	0.001089	0.033541	360.4
13.90	1544.8	1575.6	1671.8	1.81	13.90	0.001159	0.034700	400.6
15.82	1545.5	1586.2	1688.4	1.92	15.82	0.001221	0.035921	440.5
17.86	1557.1	1583.0	1689.7	2.03	17.86	0.001282	0.037202	480.0
24.29	1567.2	1594.1	1712.0	6.43	24.29	0.004061	0.041263	588.6
30				5.71	30.00	0.003585	0.044848	668.9

V_{S30} (m/s)	668.9
Class	**C**

TABLE 2

V_{S30}* Calculation for the Site Class (considering less than 3 metres of soils)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	94.1	113.8	180.1	**considering less than 3 metres of soils**				
0.71	158.0	173.1	201.3					
1.54	142.1	176.9	196.3					
2.47	173.3	235.2	283.2					
2.94	173.3	235.2	283.2					
3.52	203.4	254.5	319.4	0.58	0.58	0.002451	0.002451	235.2
4.67	211.1	309.0	1082.2	1.15	1.73	0.004533	0.006984	247.8
5.93	1036.9	1327.6	1525.7	1.26	2.99	0.004090	0.011074	270.4
7.31	1409.7	1491.8	1574.6	1.37	4.37	0.001035	0.012109	360.7
8.79	1448.4	1527.2	1622.3	1.48	5.85	0.000994	0.013103	446.6
10.38	1488.7	1563.7	1629.3	1.59	7.44	0.001043	0.014146	526.3
12.09	1533.4	1564.3	1644.0	1.70	9.15	0.001089	0.015236	600.4
13.90	1544.8	1575.6	1671.8	1.81	10.96	0.001159	0.016395	668.6
15.82	1545.5	1586.2	1688.4	1.92	12.88	0.001221	0.017615	731.4
17.86	1557.1	1583.0	1689.7	2.03	14.92	0.001282	0.018897	789.4
24.29	1567.2	1594.1	1712.0	6.43	21.35	0.004061	0.022958	929.8
32.94				8.65	30.00	0.005429	0.028387	1056.8

V_{S30}* (m/s)	1056.8
Class	**B (1)**

(1) : The Site Classes A and B are not to be used if there is more than 3 metres of unconsolidated material between the rock surface and the bottom of the spread footing or mat foundation.

APPENDIX G

Hydraulic Testing Results

BH/MW18-01

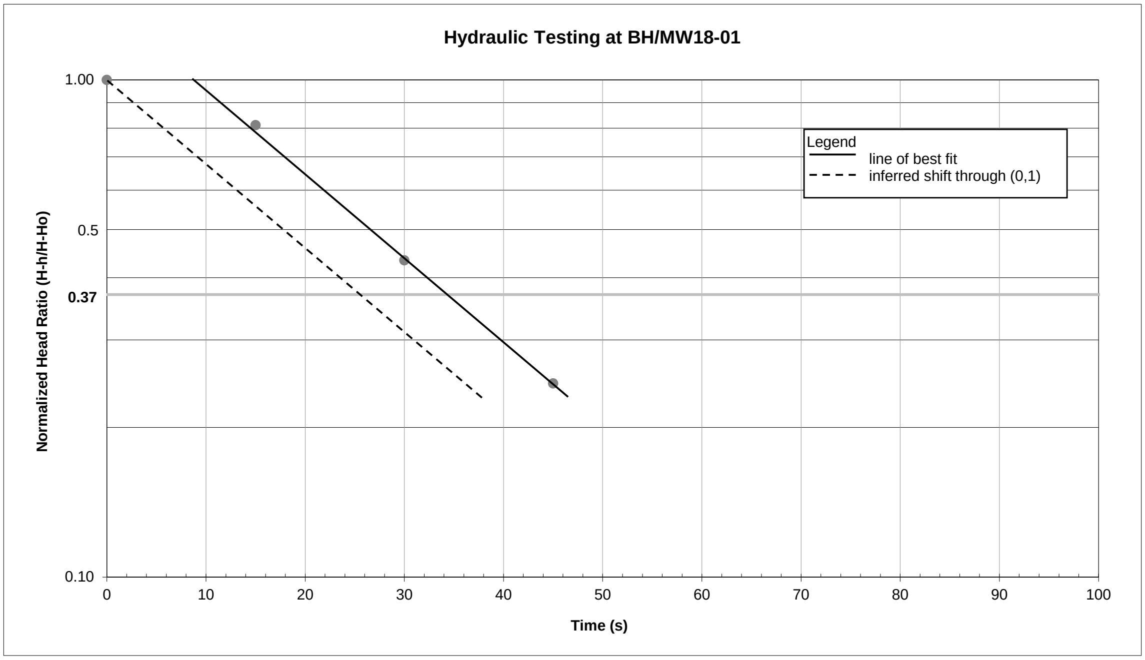
Hvorslev Bail-Test Method

Time (hr:min:sec)	Time (s)	h (m BTOR)	H-h (m)	$(H-h/H-H_o)$	Comments
0:00:00	0	3.39	0.053	1.000	H (m BTOR)) = 3.443
0:00:15	15	3.4	0.043	0.811	H_O (m BTOR) = 3.390
0:00:30	30	3.42	0.023	0.434	
0:00:45	45	3.43	0.013	0.245	BTOR = below top of riser
0:02:30	150	3.445	-0.002	-0.038	H = static water level
0:03:30	210	3.45	-0.007	-0.132	H_O = water level at T = 0
					h = water level (m BTOR)

$$K = \frac{r_c^{\,2} \ln\left(\frac{L}{R}\right)}{2\ L\ T_o}$$

K = 1.4E-05 m/s

R (borehole radius)	0.05	m
L (interval length)	3.60	m
r_c (radius of well casing)	0.025	m
T_O (T = 63% recovery)	26	s



Prepared by: SNG
Reviewed by: RW
Date: 4-Jul-18
18-206-2_Slug Tests_R0.xlsx

BH/MW18-06

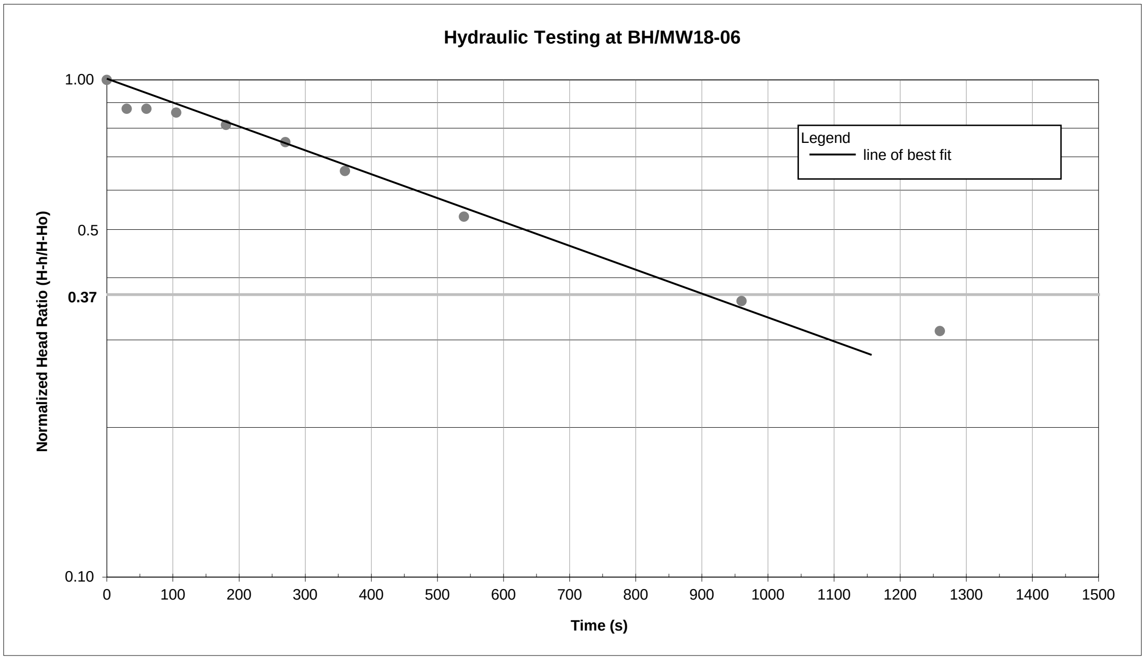
Hvorslev Bail-Test Method

Time (hr:min:sec)	Time (s)	h (m BTOR)	H-h (m)	$(H\text{-}h/H\text{-}H_o)$	Comments
0:00:00	0	3.95	0.320	1.000	H (m BTOR)) = 4.270
0:00:30	30	3.99	0.280	0.875	H_O (m BTOR) = 3.950
0:01:00	60	3.99	0.280	0.875	
0:01:45	105	3.995	0.275	0.859	BTOR = below top of riser
0:03:00	180	4.01	0.260	0.813	H = static water level
0:05:00	270	4.03	0.240	0.750	H_O = water level at T = 0
0:06:00	360	4.06	0.210	0.656	h = water level (m BTOR)
0:09:00	540	4.1	0.170	0.531	
0:16:00	960	4.155	0.115	0.359	
0:21:00	1260	4.17	0.100	0.313	

$$K = \frac{r_c^{\,2} \ln\left(\frac{L}{R}\right)}{2\,L\,T_o}$$

K = 3.7E-07 m/s

R (borehole radius)	0.10	m
L (interval length)	3.30	m
r_c (radius of well casing)	0.025	m
T_O (T = 63% recovery)	900	s



Prepared by: SNG
Reviewed by: RW
Date: 4-Jul-18
18-206-2_Slug Tests_R0.xlsx

Geofirma Engineering Ltd

APPENDIX H

2015 National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

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

June 13, 2018

Site: 45.35 N, 75.93 W User File Reference: 788 March Rd.

Requested by: ,

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

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.403	0.475	**0.399**	0.304	**0.216**	**0.109**	**0.052**	**0.014**	**0.0051**	**0.256**	**0.180**

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

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.038	0.128	0.216
Sa(0.1)	0.054	0.164	0.266
Sa(0.2)	0.050	0.144	0.228
Sa(0.3)	0.040	0.112	0.176
Sa(0.5)	0.029	0.081	0.127
Sa(1.0)	0.014	0.042	0.065
Sa(2.0)	0.0058	0.019	0.031
Sa(5.0)	0.0012	0.0045	0.0076
Sa(10.0)	0.0006	0.0018	0.0031
PGA	0.029	0.090	0.145
PGV	0.020	0.062	0.102

References

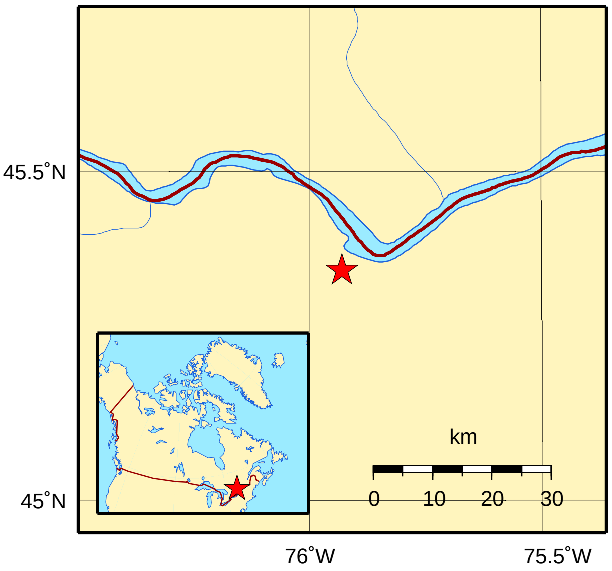
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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

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

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

Aussi disponible en français



Natural Resources Canada Ressources naturelles Canada

Canada