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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1258 MARENGER STREET, ORLEANS WARD
CITY OF OTTAWA, ONTARIO

Project # 200083

Submitted to:

1786302 Ontario Inc.
209 Pretoria Avenue
Ottawa, Ontario
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February 2020

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RE: GEOTECHNICAL INVESTIGATION
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CITY OF OTTAWA, ONTARIO

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the above noted proposed residential development located on the west side of Marenger Street, about 85 metres north of the intersection of Marenger Street and St. Joseph Boulevard (see Key Plan, Figure 1).

The purpose of the investigation was to:

- Identify the subsurface conditions at the site by means of a limited number of boreholes;
- Based on the factual information obtained, provide recommendations and guidelines on the geotechnical engineering aspects of the project design; including bearing capacity and other construction considerations, which could influence design decisions.

2.0 BACKGROUND INFORMATION AND SITE GEOLOGY

2.1 Existing Conditions and Site Geology

The subject site for this assessment consists of about a 0.14 hectare (0.35 acres) rectangular shaped property located at 1258 Marenger Street, Orleans Ward, City of Ottawa, Ontario (see Key Plan, Figure 1).

For the purposes of this assessment, project north lies in a direction parallel to Marenger Street which is located immediately east of the subject site.

Surrounding land use is currently mixed residential and commercial development. The site is bordered on the west by vacant commercial development and Jeanne D'Arc Medical Center, on the south and north by high density residential development and on the east by Marenger Street followed by residential development. The site is currently occupied by a single family dwelling.

The ground surface at the site is currently graded such that surface water drains from south to north across the subject site.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by fine textured glaciomarine deposits. Bedrock geology maps indicate that the bedrock underlying the site consists of Paleozoic dolostone of the Oxford Formation.

Based on a review of overburden thickness mapping for the site area, the overburden is estimated to be between about 50 to 100 metres in thickness above bedrock.

2.2 Proposed Development

It is understood that preliminary plans are being prepared for the construction of two, 3-storey, 12 unit residential buildings. The residential buildings will be serviced by adjacent ground surface parking. It is understood that the buildings will be wood framed with some brick veneer and cast-in-place concrete construction with conventional concrete spread footing foundations and concrete slab-on-grades. The proposed buildings will be provided with an asphaltic concrete and gravel surfaced access roadway and parking area. The proposed buildings will be serviced by municipal water and sanitary services.

Surface drainage for the proposed buildings will be by means of swales, catch basins and storm sewers.

3.0 PROCEDURE

The field work for this investigation was carried out on February 7, 2020, at which time five boreholes, numbered BH1 to BH5 were put down at the site using a truck mounted drill rig equipped with a hollow stem auger owned and operated by CCC Group of Ottawa, Ontario. Two deep boreholes (BH1 and BH2) were put down within the proposed building footprints and three shallow boreholes (BH3 to BH5) were put down within the proposed parking area.

Sampling of the overburden materials encountered at the borehole location was carried out at regular 0.75 metre depth intervals using a 50 millimetre diameter drive open conventional split spoon sampler in conjunction with standard penetration testing (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils) and in situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil). Two of the boreholes (BH1 and BH2) were advanced within the building footprints to depths of about 9.3 metres below the existing ground surface using 200 mm hollow stem augers. Borehole BH2 was continued to a depth of about 31.7 metres below the existing ground surface as a probe hole using dynamic cone penetration testing. Three boreholes (BH3 to BH5) were advanced to depths of about 1.8 metres for pavement design purposes. The soils were classified using the Unified Soil Classification System.

The subsurface soil conditions encountered at the boreholes were classified based on visual and tactile examination of the samples recovered (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), standard penetration tests (ASTM D-1586) as well as laboratory test results on select samples. Groundwater conditions at the boreholes were noted at the time of drilling. The boreholes were loosely backfilled with the auger cuttings upon completion of drilling.

One soil sample (BH1 – SS4 - 3.05 - 3.65m) was submitted for Atterberg Limits (D4318) and Moisture Content (ASTM D2216). One sample (BH2 - SS6 - 6.1 to 6.7m) was submitted for particle size analysis (ASTM D422). The samples were selected based on depth and tactile examination to be representative of the various soil conditions encountered at the site. The soils were classified using the Unified Soil Classification System. A total of 17 soil samples recovered from the boreholes were also tested for moisture content (ASTM D2216).

One soil sample (BH2 - SS2 - 1.5 - 2.1m) was delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack on concrete and corrosivity to buried steel.

The field work was supervised throughout by a member of our engineering staff who located the boreholes in the field, logged the boreholes and cared for the samples obtained. A description of the subsurface conditions encountered at the boreholes are given in the attached Record of Borehole Sheets. The results of the laboratory testing of the soil samples are presented in the Laboratory Test Results section and Attachments A and B following the text in this report. The approximate locations of the boreholes are shown on the attached Site Plan, Figure 2.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, a description of the subsurface conditions encountered at the boreholes is provided in the attached Record of Borehole Sheets following the text of this report. The borehole logs indicate the subsurface conditions at the specific drill locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than borehole locations may vary from the conditions encountered at the boreholes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification was in general completed by visual-manual procedures in accordance with ASTM 2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) with select samples being classified by laboratory testing in accordance with ASTM 2487. Classifications were confirmed by laboratory testing by test methods conforming to ASTM D4318, ASTM D2216 and ASTM D422.

Classification and identification of soil involves judgement and Kollaard Associates Inc. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The groundwater conditions described in this report refer only to those observed at the location and on the date the observations were noted in the report and on the borehole logs. Groundwater conditions may vary seasonally, or may be affected by construction activities on or in the vicinity of the site.

The following is a brief overview of the subsurface conditions encountered at the boreholes.

4.2 Fill

Fill materials consisting of topsoil and yellow brown silty sand with a trace of clay and organics was encountered from the surface at all of the boreholes. The fill materials ranged in thickness from about 0.5 to 0.9 metres at the borehole locations. The fill materials were fully penetrated at all borehole locations.

4.3 Silty Clay

Beneath the fill materials, a deposit of grey brown to grey silty clay was encountered at all of the boreholes. In situ vane shear tests carried out in the silty clay deposit gave undrained shear strength values ranging from about 45 to 86 kilopascals in boreholes BH1 and BH2. The results of the in situ vane shear testing and tactile examination carried out for the silty clay material indicate that the silty clay is firm to stiff in consistency.

Borehole BH2 was advanced through the silty clay by dynamic cone penetration testing to refusal at about 31.69 metres below the existing ground surface. Based on the increase in the standard cone penetration values in blow counts per 300 mm obtained at BH2 at a depth of about 28.96 metres below the existing ground surface, it is considered that the silty clay deposit layer is about 19.67 metres in thickness. Borehole BH2 was terminated on practical refusal to cone penetration on a boulder or bedrock at a depth of about 31.69 metres below the existing ground surface.

The results of Atterberg Limits tests and moisture content (ASTM D422) conducted on one soil sample (BH1 – SS4 - 3.05 - 3.65 metres) of the silty clay are presented in the following table and in Attachment A at the end of the report. The tested silty clay sample classifies as high plasticity in accordance with the Unified Soil Classification System. The results of the laboratory testing are located in Attachment A.

Table I – Atterberg Limit and Water Content Results

Sample	Depth(metres)	LL (%)	PL (%)	PI (%)	W (%)
BH3-SS4	2.28 - 2.88	73.7	25.9	47.8	48.9

LL: Liquid Limit PL: Plastic Limit PI: Plasticity Index w: water content

CH: Inorganic High Plastic Clays

The results of one hydrometer test (ASTM D422 and D2216) on a sample of soil (BH2-SS6) indicates the sample has a sand content of 1.0 percent, silt content of about 28.5 percent and a clay content of 70.5 percent. The results are located in Attachment A.

4.4 Glacial Till

Borehole BH2 was advanced through the silty clay by dynamic cone penetration testing to refusal at about 31.69 metres below the existing ground surface. The dynamic cone penetration test at BH2 gave values ranging from 7 to 220 blows per 0.3 metres. Based on the increase in the standard cone penetration values in blow counts per 300 mm obtained at BH2 at a depth of about 28.96 metres below the existing ground surface, it is considered that the silty clay deposit layer is about 19.67 metres in thickness. Borehole BH2 was terminated with practical refusal to cone penetration on either a boulder or bedrock at a depth of about 31.69 metres below the existing ground surface.

It is considered likely that the increase in blow count at about 28.96 metres depth indicates the possible presence of glacial till materials.

4.5 Moisture Contents

A total of 17 soil samples recovered from the boreholes were also tested for moisture content (ASTM D2216). The measured moisture contents of the soil samples ranged from about 23 to 68 percent. The results of the moisture content are located on the Record of Borehole sheets following the text of this report.

4.6 Groundwater

Some groundwater seepage was encountered within each of the boreholes at the time of drilling at depths of 3.7 metres below the existing ground surface at both boreholes. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

4.7 Corrosivity on Reinforcement and Sulphate Attack on Portland Cement

The results of the laboratory testing of a soil sample for submitted for chemistry testing related to corrosivity is summarized in the following table.

Item	Threshold of Concern	Test Result	Comment
Chlorides (Cl)	Cl > 0.04 %	0.0684	Some concern
pH	5.0 < pH	7.57	Basic Negligible concern
Resistivity	R < 20,000 ohm-cm	5950	Moderately Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	0.0102	Negligible concern

The results were compared with Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and posses a "negligible" risk for sulphate attack on concrete materials and accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements.

The pH value for the soil sample was reported to be at 7.57, indicating a durable condition against corrosion. This value was evaluated using Table 2 of Building Research Establishment (BRE) Digest 362 (July 1991).The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids.

The chloride content of the sample was also compared with the threshold level and presents some concern to concrete corrosion potential. Consideration on increasing the cover over reinforcement bars and using rebar coated by epoxy, having cathodic protection or by use of stainless steel-clay rebar should be considered.

The results of the laboratory testing of a soil sample for resistivity and pH indicates the soil sample tested has an underground corrosion rate of about 0.65 loss-oz./ft^2/yr (5950 ohm-cm). Based on the findings of Fischer and Bue (1981) underground corrosion rates (loss-oz./ft^2/yr) of 0.30 and less are considered nonaggresive, from 0.30 to 0.75 the rate is considered slightly aggressive, from 0.75 to 2.0 the rate is considered aggressive and 2.0 and greater the rate is considered very aggressive. Accordingly, the above mentioned soil sample is considered to have a slightly aggressive corrosion rate to reinforcement steel within below grade concrete walls. Based on the chemical test results, Type GU General use Hydraulic Cement may be used for this proposed development. Special protection is required for reinforcement steel within the concrete walls.

5.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the information from the test holes and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers and is intended for this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from offsite sources are outside the terms of reference for this report.

5.2 Foundations for Proposed Residential Buildings

The site is underlain by a thin layer of fill materials overlying native silty clay. Based on the undrained shear strength measurements within the silty clay deposit, the silty clay has a firm to stiff consistency and has a limited capacity to support loads from footings and grade raise fill. The allowable bearing pressure for any footings depends on the depth of the footings below original ground surface, the width of the footings, and the height above the original ground surface of any landscape grade raise adjacent to the foundations and the thickness of the soils deposit beneath the footings.

5.3 Foundation Design and Bearing Capacity

The subsurface conditions at the site encountered at the boreholes advanced during the investigation consisted of fill materials (topsoil and silty sand fill) followed by native silty clay. With the exception of the fill materials, the subsurface conditions encountered at the test holes advanced during the investigation are suitable for the support of the proposed buildings on conventional spread footing foundations placed on a native subgrade or on engineered fill placed on the native subgrade.

For predictable performance of the proposed foundations, all existing fill materials and any deleterious materials should be removed from within the proposed foundation areas to expose the native silty clay.

Strip and pad footings, a minimum 0.5 metres in width bearing on the native undisturbed silty clay at a founding depth of about 1.2 to 1.3 metres below the existing ground surface and above the groundwater level or on a suitably constructed engineering pad placed on the native silty clay may be designed using a maximum allowable bearing pressure of 75 kilopascals for serviceability limit states and 150 kilopascals for the factored ultimate bearing resistance.

The above allowable bearing pressure is subject to a maximum grade raise of 1.0 metres above the existing ground surface and to maximum strip and pad footing widths of 1.5 metres.

Provided that any loose and/or disturbed soil is removed from the bearing surfaces prior to pouring concrete, the total and differential settlement of the footings should be less than 25 millimetres and 20 millimetres, respectively.

5.4 Engineered Fill

Any fill required to raise the footings for the proposed buildings to founding level should consist of imported granular material (engineered fill). The engineered fill should consist of granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 300 millimetre thick loose lifts to at least 98 percent of the standard Proctor maximum dry density. It is considered that the engineered fill should be compacted using dynamic compaction with a large diameter vibratory steel drum roller or diesel plate compactor. If a diesel plate compactor is used, the lift thickness may need to be restricted to less than 300 mm to achieve proper compaction. Compaction should be verified by a suitable field compaction test method.

To allow the spread of load beneath the footings, the engineered fill should extend down and out from the edges of the footing at 1 horizontal to 1 vertical, or flatter. The excavations for the proposed buildings should be sized to accommodate this fill placement.

The first lift of engineered fill material should have a thickness of 300 millimetres in order to protect the subgrade during compaction. It is considered that the placement of a geotextile fabric between the engineered fill and the subgrade is not necessary where granular materials meeting the grading requirements for OPSS Granular B Type II or OPSS Granular A are placed on a silty clay subgrade above the normal ground water level. It is recommended that trucks are not used to place the engineered fill on the subgrade. The fill should be dumped at the edge of the excavation and moved into place with a tracked bulldozer or excavator.

The native silty clay soils at this site will be sensitive to disturbance from construction operations and from rainwater or snowmelt, and frost. In order to minimize disturbance, construction traffic operating directly on the subgrade should be kept to an absolute minimum and the subgrade should be protected from below freezing temperatures.

5.4.1 Foundation Excavation

Any excavation for the proposed structures will likely be carried out through fill material to bear within the native silty clay subgrade. The sides of the excavations should be sloped in accordance with the requirements of Ontario Regulation 213/91, s. 226 under the Occupational Health and

Safety Act. According to the Act, the native soils at the site can be classified as Type 3 soil, however this classification should be confirmed by qualified individuals as the site is excavated and if necessary, adjusted.

It is expected that the side slopes of the excavation will be stable in the short term provided the walls are sloped at 1H:1V through the fill materials to 1.2 metres or less from the bottom of the excavation and provided no excavated materials are stockpiled within 3 metres of the top of the excavations.

5.4.2 Effect of Foundation Excavation on Adjacent Structures and City of Ottawa Services

As previously indicated, the proposed foundation excavations will be carried out through fill and native silty clay. There will be no bedrock excavation or removal. As such, there will be no excavation processes which could contribute to vibration which could potentially damage adjacent City of Ottawa Services.

5.4.3 Ground Water in Excavation and Construction Dewatering

Groundwater inflow from the native soils into the excavations during construction, if any should be handled by pumping from sumps within the excavation.

Groundwater was observed in BH1 and BH2 at about 3.7 metres below the ground surface at time of drilling. Based on the groundwater levels observed, it is considered that the excavation for the new buildings at the site should not extend below the ground water level. As such a permit to take water is will not be required prior to excavation.

5.4.4 Effect of Dewatering of Foundation or Site Services Excavations on Adjacent Structures

Since the existing ground water level at the site will be below the expected underside of footing elevations, dewatering of the excavation will not remove water from historically saturated soils. The closest building is located about 15 metres north of the proposed buildings on the subject site. As such dewatering of the foundation or site services excavations, if required, will not have a detrimental impact on the adjacent structures.

5.5 Frost Protection Requirements for Spread Footing Foundations

In general, all exterior foundation elements and those in any unheated parts of the proposed buildings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated foundation elements adjacent to surfaces, which are cleared of snow cover during winter months should be provided with a minimum 1.8 metres of earth cover for frost protection purposes.

Where less than the required depth of soil cover can be provided, the foundation elements should be protected from frost by using a combination of earth cover and extruded polystyrene rigid insulation. A typical frost protection insulation detail could be provided upon request, if required.

5.6 Foundation Wall Backfill and Drainage

The native soils encountered at this site are considered to be frost susceptible. As such, to prevent possible foundation frost jacking due to frost adhesion, the backfill against the foundation walls and isolated walls or piers should consist of free draining, non-frost susceptible material. If imported material is required, it should consist of sand or sand and gravel meeting OPSS Granular B Type I grading requirements. Alternatively, foundations could be backfilled with native material in conjunction with the use of an approved proprietary drainage layer system such as "System Platon" against the foundation wall. It is pointed out that there is potential for possible frost jacking of the upper portion of some types of these drainage layer systems if frost susceptible material is used as backfill. This could be mitigated by backfilling the upper approximately 0.6 metres with non-frost susceptible granular material.

Where the backfill material will ultimately support a pavement structure or walkway, it is suggested that the foundation wall backfill material be compacted in 250 millimetre thick lifts to 95 percent of the standard Proctor dry density value. In that case any native material proposed for foundation backfill should be inspected and approved by the geotechnical engineer.

The basement foundation walls should be designed to resist the earth pressure, P, acting against the walls at any depth, h, calculated using the following equation.

$$\mathbf{P = k_0 (\gamma h + q)}$$

Where:

	P	=	the pressure, at any depth, h, below the finished ground surface
	k_0	=	earth pressure at-rest coefficient, 0.5
	γ	=	unit weight of soil to be retained, estimated at 22 kN/m^3
	q	=	surcharge load (kPa) above backfill material
	h	=	the depth, in metres, below the finished ground surface at which the pressure, P, is being computed

This expression assumes that the water table would be maintained at the founding level by the above mentioned foundation perimeter drainage and backfill requirements.

5.7 Slab on Grade Support

As stated above, it is expected that the proposed buildings will be founded on native silty clay or on an engineered pad placed on the native subgrade. For predictable performance of the proposed concrete floor slab all existing fill material, topsoil and any otherwise deleterious material should be removed from below the proposed floor slab 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. Any fill materials consisting of granular material, removed from the proposed concrete floor slab area, could be stockpiled for possible reuse with approval from the geotechnical engineer.

The fill materials beneath the proposed concrete floor slab on grade should consist of a minimum of 150 millimetre thickness of crushed stone meeting OPSS Granular A immediately beneath the concrete floor slab followed by sand, or sand and gravel meeting the OPSS for Granular B Type I, or crushed stone meeting OPSS grading requirements for Granular B Type II, or other material approved by the Geotechnical Engineer. The fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density.

The slab should be structurally independent from walls and columns, which are supported by the foundations. This is to reduce any structural distress that may occur as a result of differential soil movement. If it is intended to place any internal non-load bearing partitions directly on the slab-on-grade, such walls should also be structurally independent from other elements of the building founded on the conventional foundation system so that some relative vertical movement between the floor slab and foundation can occur freely.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage 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 the lesser of 25 times the slab thickness or 4.5 metres. The slab should be cut as soon as it is possible to work on the slab without damaging the surface of the slab.
If any areas of the proposed buildings are to remain unheated during the winter period or under slab insulation is to be used, thermal protection of the foundation may be required. Further details on the insulation requirements could be provided, if necessary.

5.8 Seismic Design for the Proposed Residential Buildings

5.8.1 Seismic Site Classification

Based on the limited information from the boreholes, for seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class D. The assumed basement floor level is about 1.2 metres below the existing ground surface.

Borehole 2					
Layer	Description	Depth (m)	d_i (m)	S_{ui} (kPa)	d_i/S_{ui} (m/kPa)
1	USF	1.2			
2	Silty Clay	28.96	28.8	56.3	0.5
3	Silty Clay	30	1.2	N/A	
	$d_c/(\text{sum}(d_i/S_{ui}))$				56.3

Since $S_u = 50 < 56.3 < 100$ kPa the seismic site response is Site Class D.

5.9 National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.300 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The results of the test are attached following the text of this report.

5.9.1 Potential for Soil Liquefaction

As indicated above, the results of the boreholes and information from geological maps indicate that the native deposits underlying the site consist of fill materials (silty sand, topsoil) followed by a stiff silty clay crust then by firm clays to depths of about 28.96 metres.

C.F.E.M. section 6.6.3.2 (6) recommends that the Bray et al. (2004) criteria be used to determine liquefaction susceptibility of fine-grained soils:

That is fine-grained soils with PI ≤ 12 and $W_c > 0.85LL$ are susceptible to liquefaction, soils with 12 ≤ PI ≤ 20 and $W_c > 0.8LL$ are moderately susceptible to liquefaction and soils with PI > 20 and $W_c < 0.8LL$ are not susceptible to liquefaction.

Seed et al. (2003) proposed liquefaction susceptibility criteria that are similar to those by Bray et al. (2004) except that they include slightly different Wc / LL ratios and include constraints on LL. The criteria by Seed et al. (2003) are described by three zones on the Atterberg limits chart, which are bounded by the following PI and LL values: Zone A soils have PI ≤ 12 and LL ≤ 37 and are considered potentially susceptible to “classic cyclically induced liquefaction” if the water content is greater than 80% of the LL; Zone B soils have PI ≤ 20 and LL ≤ 47 and are considered potentially liquefiable with detailed laboratory testing recommended if the water content is greater than 85% of the LL; and Zone C soils with PI > 20 or LL >47 are considered generally not susceptible to classic cyclic liquefaction, although they should be checked for potential sensitivity.

From the laboratory test results, the silty has a plasticity index PI = of 47.8 and a liquid limit of 73.7 indicating an inorganic highly plastic clay. As such the silty clay is not prone to liquefaction.

6.0 SITE SERVICES

6.1 Excavation

The excavations for the site services will be carried out through fill materials (silty sand/topsoil) and silty clay. For the purposes of Ontario Regulation 213/91 the soils at the site can be considered to be Type 3 soil. The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupational Health and Safety Act. That is, open cut excavations with overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box.

Based on the depths at which groundwater was measured within borehole BH2, significant groundwater flow into any excavation is unlikely. Any groundwater inflow into the service trenches should be handled by pumping from sumps from within the excavations.

6.2 Pipe Bedding and Cover Materials

It is suggested that the service pipe bedding material consist of at least 150 millimetres of granular material meeting OPSS requirements for Granular A. A provisional allowance should, however, be made for sub-excavation of any existing fill or disturbed material encountered at sub-grade level. 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 should not be permitted.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A or Granular B Type I (with a maximum particle size of 25 millimetres).

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

6.3 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 sub-grade 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 materials 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 recompaction 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. Any boulders larger than 300 millimetres in size should not be used as service trench backfill. 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. If the native material is not suitable for backfill, imported granular material may have to be used. If imported granular materials are used, suitable frost tapers should be used OPSD 802.013.

To minimize future settlement of the backfill and achieve an acceptable sub-grade for the roadways, sidewalks, etc., the trench should be compacted in maximum 300 millimetre 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 or in close proximity to existing or future roadways, driveways, sidewalks, or any other type of permanent structure.

7.0 ACCESS ROADWAY AND PARKING LOT PAVEMENTS

7.1 Subgrade Preparation

In preparation for pavement construction at this site any fill and topsoil and any soft, wet or deleterious materials should be removed from the proposed access roadway and parking lot areas. The exposed subgrade surface should then be proof inspected and approved by geotechnical personnel. Based on the results of the boreholes, the subsurface conditions in the access roadway and parking areas consist of existing fill materials (silty sand with a trace of organics and silty clay) overlying native silty clay. Any soft or unacceptable areas evident should be subexcavated and replaced with suitable earth borrow material. The subgrade should be shaped and crowned to promote drainage of the roadway and parking area granulars. Following approval of the preparation of the subgrade, the pavement granulars may be placed.

For any areas of the site that require the subgrade to be raised to proposed roadway and parking area subgrade level, the material used should consist of OPSS select subgrade material or OPSS Granular B Type I or Type II. Materials used for raising the subgrade to proposed roadway and parking area subgrade level should be placed in maximum 300 millimetre thick loose lifts and be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

If the subgrade surface consists of native silty clay for pavement areas subject to cars and light trucks the pavement should consist of:

50 millimetres of Superpave 12.5 asphaltic concrete over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase over
(50 or 100 millimetre minus crushed stone)

If the subgrade surface consists of existing fill materials:

50 millimetres of Superpave 12.5 asphaltic concrete over
150 millimetres of OPSS Granular A base over
450 millimetres of OPSS Granular B, Type II subbase

Non-woven geotextile fabric (4oz/sy) such as Terrafix 270R or Thrace-Ling 130EX or approved alternative

Performance grade PG 58-34 asphaltic concrete should be specified. Compaction of the granular pavement materials should be carried out in maximum 300 millimetre thick loose lifts to 100 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

The above pavement structures will be adequate on an acceptable sub-grade, that is, one where any roadway fill and service trench backfill has been adequately compacted. If the roadway sub-grade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a non-woven geotextile separator between the roadway sub-grade surface and the granular subbase material. The adequacy of the design of the pavement thickness should be assessed by the geotechnical personnel at the time of construction.

8.0 CONSTRUCTION CONSIDERATIONS

It is suggested that the final design drawings for the project, including the proposed site grading plan, be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended and to re-evaluate the guidelines provided in the report with respect to the actual project plans. Items such as actual foundation wall/column loads, whether or not the basement or below grade parking structure is heated, etc could have significant impacts on foundation type, frost protection requirements, etc.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed development do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

All foundation areas and any engineered fill areas for the proposed residential buildings should be inspected by Kollaard Associates Inc. to ensure that a suitable sub-grade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the site services, access roadways and driveway should be inspected and approved by geotechnical personnel. In situ density testing should be carried out on the service pipe bedding and backfill and the pavement granular materials to ensure the materials meet the specifications from a compaction point of view.

The native silty clay deposits at this site will be sensitive to disturbance from construction operations, from rainwater or snow melt, and frost. In order to minimize disturbance, construction traffic operating directly on the subgrade should be kept to an absolute minimum and the subgrade should be protected from below freezing temperatures.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact our office.

Regards,
Kollaard Associates Inc.

LICENSED PROFESSIONAL ENGINEER
21.02.2020
S.E. deWit
100079612
PROVINCE OF ONTARIO

Dean Tataryn, B.E.S., EP.

Steve DeWit, P.Eng.

RECORD OF BOREHOLE BH1

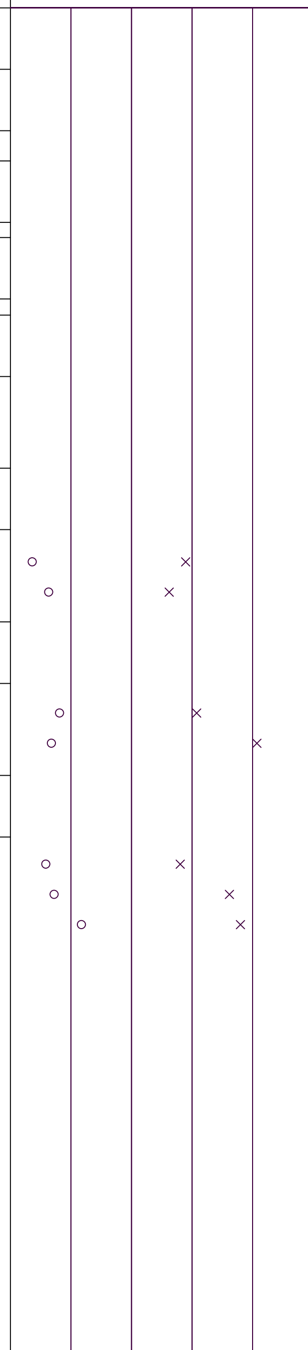
PROJECT: Proposed Residential Development
CLIENT: 1786302 Ontario Inc.
LOCATION: 1258 Marenger Street, Orleans, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 200083
DATE OF BORING: February 7, 2020
SHEET 1 of 1
DATUM: LOCAL

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING %M	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface						
0	Tospoil (FILL)	0.00					
	Yellow brown silty sand, trace clay and organics (FILL)						
	Firm grey brown SILTY CLAY	0.60	1	SS	8	39	
			2	SS	9	36	
			3	SS	7	37	
			4	SS	4	48	
	Grey SILTY CLAY	3.65	Obtained Shelby				
			5	SS	2		Water observed in borehole at approximately 3.7 metres below the existing ground surface on February 10, 2020.
			6	SS	2	61	
			7	SS	2	64	
	End of Borehole	9.29					

UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80

REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80



DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90

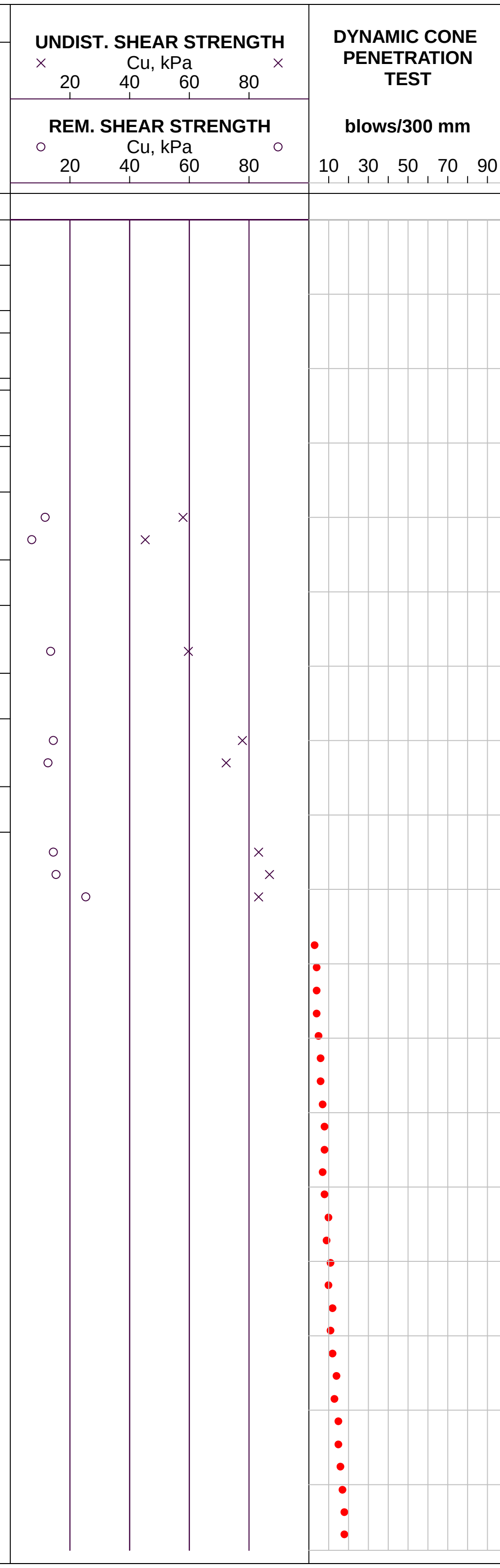
DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: 1786302 Ontario Inc.
LOCATION: 1258 Marenger Street, Orleans, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 200083
DATE OF BORING: February 7, 2020
SHEET 1 of 2
DATUM: LOCAL

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING (%M)	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface						
0	TOPSOIL (FILL)	0.00					
	Stiff grey brown SILTY CLAY	0.45					
1			1	SS	6	40	
2			2	SS	7	39	
			3	SS	4	44	
3			4	SS	2	54	
4	Grey SILTY CLAY	3.65					
5			5	SS	2	68	Water observed in borehole at approximately 3.7 metres below the existing ground surface on February 10, 2020.
6			6	SS	2	60	
7							
8			7	SS	4	60	
9	Borehole continued as Probe Hole, probably grey SILTY CLAY	9.29					
10							
11							
12							
13							
14							
15							
16							
17							



DEPTH SCALE: 1 to 100
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

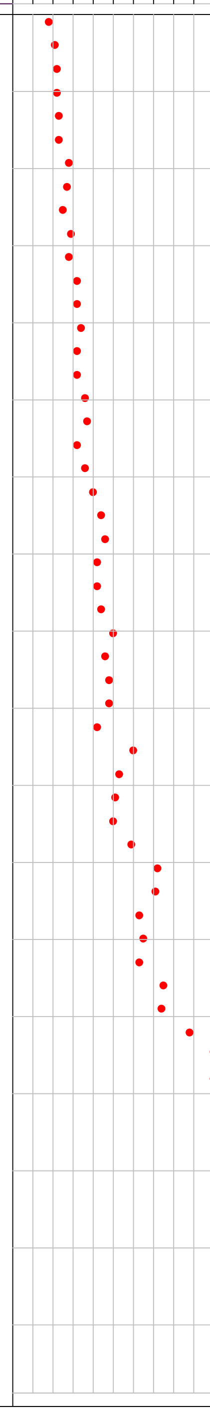
RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: 1786302 Ontario Inc.
LOCATION: 1258 Marenger Street, Orleans, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 200083
DATE OF BORING: February 7, 2020

DATUM: LOCAL

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
18–29										
29	Probably grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)		28.96							
31.69	End of Borehole, refusal on large boulder or bedrock		31.69							



DEPTH SCALE: 1 to 100
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH3

PROJECT: Proposed Residential Development
CLIENT: 1786302 Ontario Inc.
LOCATION: 1258 Marenger Street, Orleans, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 200083
DATE OF BORING: February 7, 2020
SHEET 1 of 1
DATUM: LOCAL

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa (20 40 60 80) / REM. SHEAR STRENGTH ○ Cu, kPa (20 40 60 80)	DYNAMIC CONE PENETRATION TEST blows/300 mm (10 30 50 70 90)	ADDITIONAL LAB TESTING %M	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface									
0	Tospoil (FILL)		0.00							
	Yellow brown silty sand, trace clay and organics (FILL)			1	SS	4				
1	Grey brown SILTY CLAY		0.90	2	SS	8			23	
				3	SS	14			36	
2	End of Borehole		1.80							
3										
4										Borehole dry, February 10, 2020.
5										
6										
7										
8										
9										
10										
11										
12										
13										

DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH4

PROJECT: Proposed Residential Development
CLIENT: 1786302 Ontario Inc.
LOCATION: 1258 Marenger Street, Orleans, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 200083
DATE OF BORING: February 7, 2020
SHEET 1 of 1
DATUM: LOCAL

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa (20 40 60 80) / REM. SHEAR STRENGTH ○ Cu, kPa (20 40 60 80)	DYNAMIC CONE PENETRATION TEST blows/300 mm (10 30 50 70 90)	ADDITIONAL LAB TESTING (%M)	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface									
0	Tospoil (FILL)		0.00							
	Yellow brown silty sand, trace clay and organics (FILL)			1	SS	4				
	Grey brown SILTY CLAY		0.65	2	SS	8			44	
				3	SS	14			34	
2	End of Borehole		1.80							
4										Borehole dry, February 10, 2020.

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13

DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH5

PROJECT: Proposed Residential Development
CLIENT: 1786302 Ontario Inc.
LOCATION: 1258 Marenger Street, Orleans, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 200083
DATE OF BORING: February 7, 2020
SHEET 1 of 1
DATUM: LOCAL

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING %M	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface									
0	Tospoil (FILL)		0.00							
	Yellow brown silty sand, trace clay and organics (FILL)			1	SS	4				
1	Grey brown SILTY CLAY		0.85	2	SS	8			25	
				3	SS	14			37	
2	End of Borehole		1.80							
4										Borehole dry, February 10, 2020.

DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
MS manual sample
RC rock core
ST slotted tube .
TO thin-walled open Shelby tube
TP thin-walled piston Shelby tube
WS wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance, N

The number of blows by a 63.5 kg hammer dropped 760 millimeter required to drive a 50 mm drive open sampler for a distance of 300 mm. For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance

The number .of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter, 60° cone attached to 'A' size drill rods for a distance of 300 mm.

WH

_Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drih rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH sieve and hydrometer analysis
U unconfined compression test
Q undrained triaxial test
V field vane, undisturbed and remolded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to100
Very Stiff	over100

LIST OF COMMON SYMBOLS

Cu undrained shear strength
e void ratio
Cc compression index
Cv coefficient of consolidation
k coefficient of permeability
Ip plasticity index
n porosity
u pore pressure
w moisture content
wL liquid limit
Wp plastic limit
$1 effective angle of friction
r unit weight of soil
y1 unit weight of submerged soil
cr normal stress

KEY PLAN	FIGURE 1

NOT TO SCALE

Kollaard Associates
Engineers

Project No. 200083

Date February 2020

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

REFERENCE: PLAN SUPPLIED BY CITY OF OTTAWA EMAPS.

SPECIAL NOTE: THIS DRAWING TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING REPORT.

REV.	NAME	DATE	DESCRIPTION

Kollaard Associates
Engineers

PO, BOX 189, 210 PRESCOTT ST
KEMPTVILLE ONTARIO
K0G 1J0 FAX (613) 258-0475
http://www.kollaard.ca
(613) 860-0923
info@kollaard.ca

CLIENT:
1786302 ONTARIO INC.

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION:
1258 MARENGER STREET
ORLEANS WARD
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
--	FEB 21, 2020
DRAWN BY:	SCALE:
DT	N.T.S

KOLLAARD FILE NUMBER:
200083

ATTACHMENT A

Laboratory Test Results for Physical Properties

Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

March 3, 2020
File: 122410003

Attention: **Dean Tataryn, Kollaard Associates Engineers**

Reference: **Kollaard File #200083**
ASTM D4318 Atterberg Limit & ASTM D2216 Moisture Content

The table below summarizes Atterberg Limit & Moisture Content results for BH-1 SS4.

Source	Depth	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index
BH-1 SS4	10'-12'	48.9%	73.7	25.9	47.8

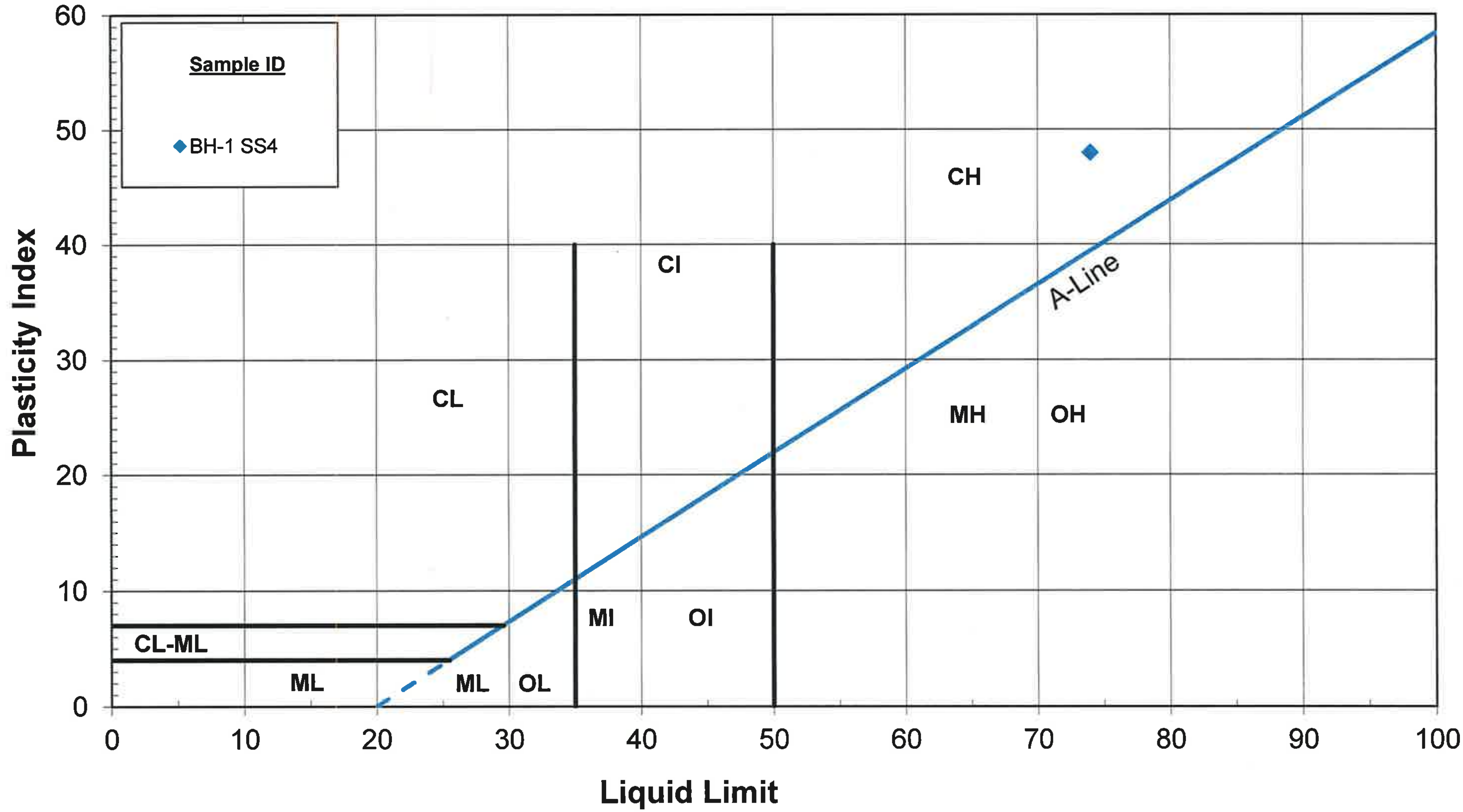
Sincerely,

Stantec Consulting Ltd

Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-722-2799
brian.prevost@stantec.com

Attachments: Atterberg Limit Plasticity Chart



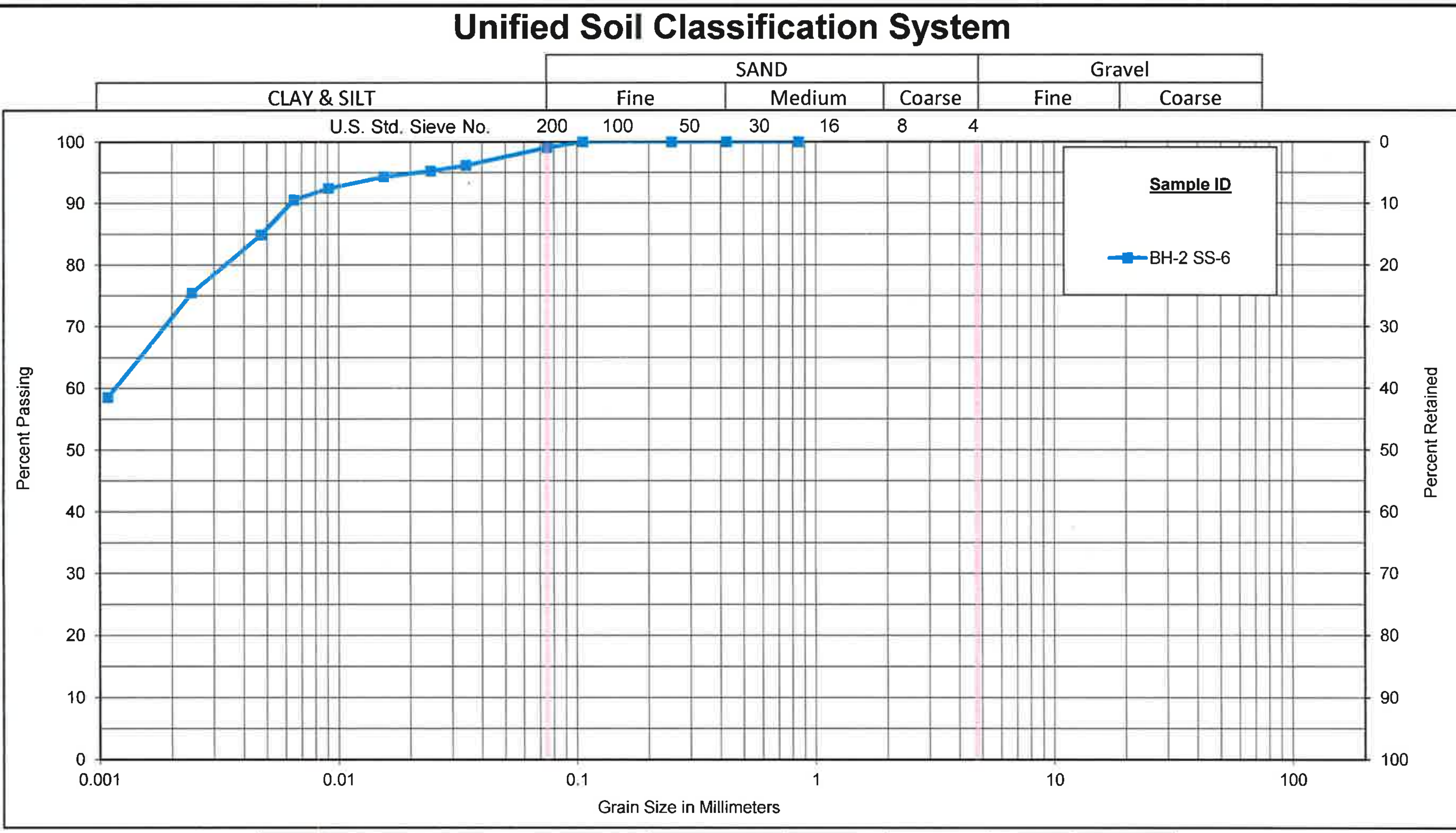
Stantec

Kollaard Associates, File # 200083

1258 Meranger Street. Ottawa

PLASTICITY CHART

Figure No.
Project No. 122410003



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH-2 SS-6	20'-22'	0.0	1.0	28.5	70.5

GRAIN SIZE DISTRIBUTION

Kollaard Associates, File # 200083

1258 Meranger Street

Figure No.

Project No. 122410003

Stantec

Particle-Size Analysis of Soils
LS702
AASHTO T88

PROJECT DETAILS

Client:	Kollaard Associates, File # 200083	Project No.:	122410003
Project:	1258 Meranger Street	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH-2	Date Sampled:	February 20, 2020
Sample No.:	SS-6	Tested By:	Denis Rodriguez
Sample Depth	20'-22'	Date Tested:	February 24, 2020

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	51.90
Sample Weight after Hydrometer and Wash (g)	0.50
Percent Passing No. 200 Sieve (%)	99.0
Percent Passing Corrected (%)	99.04

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	161.00
Sample Weight After Sieve (g)	160.40
Percent Loss in Sieve (%)	0.37

SOIL INFORMATION

Liquid Limit (LL)		
Plasticity Index (PI)		
Soil Classification		
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	48	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	52.26
Air Dried Mass (W_a), (g)	52.78
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9901
Air Dried Mass in Analysis (M_a), (g)	52.42
Oven Dried Mass in Analysis (M_o), (g)	51.90
Percent Passing 2.0 mm Sieve (P_{10}), (%)	100.00
Sample Represented (W), (g)	51.90

SIEVE ANALYSIS

Sieve Size mm	Cum. Wt. Retained	Percent Passing
75.0		100.0
63.0		100.0
53.0		100.0
37.5		100.0
26.5		100.0
19.0		100.0
13.2		100.0
9.5		100.0
4.75		100.0
2.00	0.0	100.0
Total (C + F)[1]	160.40	
0.850	0.00	100.00
0.425	0.00	100.00
0.250	0.00	100.00
0.106	0.00	100.00
0.075	0.50	99.04
PAN	0.50	

Note 1: (C + F) = Coarse + Fine

HYDROMETER DETAILS

Volume of Bulb (V_B), (cm^3)	63.0
Length of Bulb (L_2), (cm)	14.47
Length from '0' Reading to Top of Bulb (L_1), (cm)	10.29
Scale Dimension (h_s), (cm/Div)	0.155
Cross-Sectional Area of Cylinder (A), (cm^2)	27.25
Meniscus Correction (H_m), (g/L)	1.0

START TIME 9:27 AM

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
24-Feb-20	9:28 AM	1	59.0	8.0	22.5	51.0	96.13	7.06904	9.50295	0.012894	0.03428
24-Feb-20	9:29 AM	2	58.5	8.0	22.5	50.5	95.19	7.14654	9.50295	0.012894	0.02437
24-Feb-20	9:32 AM	5	58.0	8.0	22.5	50.0	94.25	7.22404	9.50295	0.012894	0.01550
24-Feb-20	9:42 AM	15	57.0	8.0	22.0	49.0	92.36	7.37904	9.61570	0.012970	0.00910
24-Feb-20	9:57 AM	30	56.0	8.0	22.0	48.0	90.48	7.53404	9.61570	0.012970	0.00650
24-Feb-20	10:27 AM	60	53.0	8.0	22.0	45.0	84.82	7.99904	9.61570	0.012970	0.00474
24-Feb-20	1:37 PM	250	48.0	8.0	22.0	40.0	75.3992	8.77404	9.61570	0.012970	0.00243
25-Feb-20	9:27 AM	1440	39.0	8.0	22.5	31.0	58.4344	10.16904	9.50295	0.012894	0.00108

Remarks:

Reviewed By: Brian Prevost
Date: March 3/2020

ATTACHMENT B

Laboratory Test Results for Chemical Properties

Kollaard Associates (Kemptville)
ATTN: Dean Tataryn
210 Prescott Street Unit 1
P.O. Box 189
Kemptville ON K0G 1J0

Date Received: 13-FEB-20
Report Date: 21-FEB-20 09:48 (MT)
Version: FINAL

Client Phone: 613-860-0923

Certificate of Analysis

Lab Work Order #: L2417075

Project P.O. #: NOT SUBMITTED
Job Reference: 200083
C of C Numbers:
Legal Site Desc:

Emily Smith
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2417075-1 BH2 SS2 5-7							
Sampled By: CLIENT on 10-FEB-20							
Matrix: SOIL							
Physical Tests							
Conductivity	0.168		0.0040	mS/cm		19-FEB-20	R4998389
% Moisture	25.5		0.25	%	14-FEB-20	14-FEB-20	R4997138
pH	7.57		0.10	pH units		18-FEB-20	R4997924
Redox Potential	253		-1000	mV		18-FEB-20	R4997914
Resistivity	5950		1.0	ohm*cm		19-FEB-20	
Leachable Anions & Nutrients							
Chloride	0.0684		0.00050	%	18-FEB-20	19-FEB-20	R4998934
Anions and Nutrients							
Sulphate	0.0102		0.0020	%	15-FEB-20	19-FEB-20	R4998934
Inorganic Parameters							
Acid Volatile Sulphides	<0.20		0.20	mg/kg	19-FEB-20	19-FEB-20	R4998272

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-R511-WT	Soil	Chloride-O.Reg 153/04 (July 2011)	EPA 300.0

5 grams of dried soil is mixed with 10 grams of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

EC-WT	Soil	Conductivity (EC)	MOEE E3138

A representative subsample is tumbled with de-ionized (DI) water. The ratio of water to soil is 2:1 v/w. After tumbling the sample is then analyzed by a conductivity meter.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

MOISTURE-WT	Soil	% Moisture	CCME PHC in Soil - Tier 1 (mod)
PH-WT	Soil	pH	MOEE E3137A

A minimum 10g portion of the sample is extracted with 20mL of 0.01M calcium chloride solution by shaking for at least 30 minutes. The aqueous layer is separated from the soil and then analyzed using a pH meter and electrode.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580

This analysis is carried out in accordance with the procedure described in the "APHA" method 2580 "Oxidation-Reduction Potential" 2012. Samples are extracted at a fixed ratio with DI water. Results are reported as observed oxidation-reduction potential of the platinum metal-reference electrode employed, in mV.

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	APHA 2510 B

The reported Resistivity value is calculated as the inverse of the conductivity of a 2:1 water:soil leachate. This method does not use direct measurement of Soil Resistivity using a resistivity meter.

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	MOECC E3138

The reported Resistivity value is calculated as the inverse of the conductivity of a 2:1 water:soil leachate. This method does not use direct measurement of Soil Resistivity using a resistivity meter.

SO4-WT	Soil	Sulphate	EPA 300.0

5 grams of soil is mixed with 50 mL of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

SULPHIDE-WT	Soil	Sulphide, Acid Volatile	APHA 4500S2J

This analysis is carried out in accordance with the method described in APHA 4500 S2-J. Hydrochloric acid is added to sediment samples within a purge and trap system. The evolved hydrogen sulphide (H_2S) is carried into a basic solution by inert gas. The acid volatile sulfide is then determined colourimetrically.

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Chain of Custody Numbers:

Reference Information

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid weight of sample
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

ALS Environmental

www.alsglobal.com

Chain of Custody (COC) / Analytical Request Form

Canada Toll Free: 1 800 668 9878

L2417075-COFC

COC Number: 17 -

Page of

Report To	Contact and company name below will appear on the final report
Company:	Kollaard Associates (27196)
Contact:	Dean Tataryn
Phone:	613.860.0923, ext.225
	Company address below will appear on the final report
Street:	210 Prescott Street, Unit 1 P.O. Box 189
City/Province:	Kemptville, Ontario
Postal Code:	K0G 1J0
Invoice To	Same as Report To ☐ YES ☐ NO
	Copy of Invoice with Report ☐ YES ☐ NO
Company:	
Contact:	

Report Format / Distribution

Select Report Format: ☑ PDF ☑ EXCEL ☐ EDD (DIGITAL)

Quality Control (QC) Report with Report ☐ YES ☐ NO

☐ Compare Results to Criteria on Report - provide details below if box checked

Select Distribution: ☐ EMAIL ☐ MAIL ☐ FAX

Email 1 or Fax dean@kollaard.ca

Email 2

Email 3

Invoice Distribution

Select Invoice Distribution: ☐ EMAIL ☐ MAIL ☐ FAX

Email 1 or Fax mary@kollaard.ca

Email 2

Select Service Level Below - Contact your AM to confirm all E&P TATs (surcharges may apply)

Regular [R] ☒ Standard TAT if received by 3 pm - business days - no surcharges apply

PRIORITY (Business Days): 4 day [P4-20%] ☐ 3 day [P3-25%] ☐ 2 day [P2-50%] ☐

EMERGENCY: 1 Business day [E1 - 100%] ☐ Same Day, Weekend or Statutory holiday [E2 -200% (Laboratory opening fees may apply)] ☐

Date and Time Required for all E&P TATs:

For tests that can not be performed according to the service level selected, you will be contacted.

Project Information

ALS Account # / Quote #: Q71021

Job #: 200803

PO / AFE:

LSD:

ALS Lab Work Order # (lab use only): L2417075

Oil and Gas Required Fields (client use)

AFE/Cost Center: | PO#

Major/Minor Code: | Routing Code:

Requisitioner:

Location:

ALS Contact: Melanie M. | Sampler:

Analysis Request

Indicate Filtered (F), Preserved (P) or Filtered and Preserved (F/P) below

ALS Sample # (lab use only)	Sample Identification and/or Coordinates (This description will appear on the report)	Date (dd-mmm-yy)	Time (hh:mm)	Sample Type	Corrosivity	SAMPLES ON HOLD	Sample is hazardous (please provide further details)	NUMBER OF CONTAINERS
	BH 2 SS2 5-7	Feb 10/2020		Soil	+			

Drinking Water (DW) Samples[1] (client use)

Are samples taken from a Regulated DW System? ☐ YES ☐ NO

Are samples for human consumption/ use? ☑ YES ☐ NO

Special Instructions / Specify Criteria to add on report by clicking on the drop-down list below (electronic COC only)

SAMPLE CONDITION AS RECEIVED (lab use only)

Frozen ☐ SIF Observations Yes ☐ No ☐

Ice Packs ☐ Ice Cubes ☐ Custody seal intact Yes ☐ No ☐

Cooling Initiated ☐

INITIAL COOLER TEMPERATURES °C: 20.9

FINAL COOLER TEMPERATURES °C: 5.7

SHIPMENT RELEASE (client use)			INITIAL SHIPMENT RECEPTION (lab use only)			FINAL SHIPMENT RECEPTION (lab use only)		
Released by:	Date: Feb 13/20	Time:	Received by: Costas Farassoglou	Date: 2/13/20	Time: 12:20	Received by:	Date: Feb 14/20	Time:

REFER TO BACK PAGE FOR ALS LOCATIONS AND SAMPLING INFORMATION WHITE - LABORATORY COPY YELLOW - CLIENT COPY

SEPT 2017 FRONT

Failure to complete all portions of this form may delay analysis. Please fill in this form LEGIBLY. By the use of this form the user acknowledges and agrees with the Terms and Conditions as specified on the back page of the white - report copy.

1. If any water samples are taken from a Regulated Drinking Water (DW) System, please submit using an Authorized DW COC form.

ATTACHMENT C

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

Site: 45.464N 75.543W **User File Reference:** 1258 Marenger Street, Orleans, Ottawa, Ontario 2020-02-14 18:04 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.484	0.269	0.161	0.047
Sa (0.1)	0.562	0.323	0.201	0.065
Sa (0.2)	0.468	0.272	0.172	0.058
Sa (0.3)	0.354	0.207	0.131	0.045
Sa (0.5)	0.249	0.145	0.092	0.032
Sa (1.0)	0.123	0.072	0.046	0.016
Sa (2.0)	0.058	0.034	0.021	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.300	0.175	0.109	0.035
PGV (m/s)	0.206	0.116	0.071	0.022

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" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. 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.**

References

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

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
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