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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED LIGHT INDUSTRIAL BUILDING
6793 HIRAM DIRVE
OSGOODE WARD, GREELY
CITY OF OTTAWA, ONTARIO

Project # 180938

Submitted to:

Mr. Nat Giust
3226 Woodroffe Avenue
Nepean, Ontario
K2C 4G5

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January 29, 2020 180938

Mr. Nat Giust
3226 Woodroffe Avenue
Nepean, Ontario
K2C 4G5

RE: GEOTECHNICAL INVESTIGATION
PROPOSED LIGHT INDUSTRIAL BUILDING
6793 HIRAM DRIVE, GREELY
CITY OF OTTAWA, ONTARIO

Dear Sir:

This report presents the results of a geotechnical investigation carried out for the above noted proposed light industrial building. The purpose of the investigation was to identify the subsurface conditions at the site based on a limited number of boreholes. Based on the factual information obtained, Kollaard Associates Inc. was to provide guidelines on the geotechnical engineering aspects of the project design, including construction considerations, which could influence design decisions.

BACKGROUND INFORMATION AND SITE GEOLOGY

Plans are being prepared to construct a light industrial building to be used as a auto mechanic shop and accessory office, with a footprint of approximately 514 square metres at the site. It is understood that the building will contain three bays, office spaces, a reception area and washrooms within about a 0.4 hectare (1 acre), rectangular shaped property located on the west side of Hiram Drive, in the City of Ottawa, Ontario (see Key Plan, Figure 1).

Preliminary plans indicate that the proposed light industrial building will consist of a single storey steel frame metal clad structure with an attached wood frame auxiliary office structure. The proposed building will be placed on a conventional concrete spread footing foundation with a concrete slab-on-grade construction. The proposed building will be provided with an asphaltic concrete surfaced access roadway and parking area as well as a gravel surfaced rear yard area.

The proposed building will be serviced by a drilled cased well and an onsite septic system.

The site is located within a commercial / industrial park. The site is bordered on the north by other light industrial development, on the south by vacant lots, on the west by undeveloped land and on the east by Hiram Drive, followed by existing industrial and commercial development within the industrial development. Surface drainage for the proposed building will be directed to the existing municipal drain west of the site and to the roadside ditch east of the site by means of sheet flow, swales and a stormwater management system.

Based on a review of the surficial geology map for the site area (*Surficial Geology Map*: Geological Survey of Canada, Surficial Geology, Ottawa, Ontario, Map 1506A, published 1982, scale 1:50,000.), it is expected that the site is generally underlain by coarse textured glaciomarine deposits consisting of sand gravel, silt and clay. A review of the bedrock geology map indicates that the bedrock underlying the site consists of dolomite and limestone of the Oxford Formation (*Bedrock Geology Map*: Geological Survey of Canada, Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Map 1508A, published 1979, scale 1:125,000.). Based on a review of the topographical map for the site area, it is expected that the upper groundwater flow at the site is towards Shields Creek that exists about 230 metres south of the site.

A geotechnical report prepared by Kollaard Associates Inc for a property two lots north (6805 Hiram Drive) of the site was completed on November 10, 2017. A review of that report indicates that the property was underlain by silty sand followed by silty clay, silt then glacial till. Refusal to advance the borehole was encountered at a depth of approximately 8.1 metres below the existing ground surface level at that site.

A drilled cased water well was installed on the subject site for a hydrogeological investigation completed by Kollaard Associates Inc. The drilled cased well was installed by a company licensed to install drinking water wells by MECP in accordance with Ontario Regulation 903. The well will be used to provide drinking water for the proposed building once development is complete. From the water well record (see attachment), the surface soils are indicated to consist of sandy clay and gravel to about 15.8 metres followed by bedrock. It is considered that limestone bedrock is underlying the site at 15.8 metres below the ground surface.

PROCEDURE

The field work for this investigation was carried out on April 2, 2019, at which time two boreholes numbered BH1 and BH2 were put down at the site using a track mounted drill rig equipped with a hollow stem auger owned and operated by Marathon Drilling of Greely, Ontario.

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 – Penetration Test and Split Barrel Sampling of Soils as well as laboratory test results on select samples. In situ vane shear testing (ASTM D2573 - Standard test method for Field vane shear test in cohesive soil) was carried out in the cohesive materials encountered within the boreholes. The soils were classified using the Unified Soil Classification System. 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.

Two soil samples (BH1 - SS4 and BH2 - SS8) were submitted for particle size analysis (ASTM D422). One soil sample (BH2 - SS3) was submitted for Atterberg Limits Determination (ASTM D4318). One sample of soil obtained from BH2 was also delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack and soil corrosion on buried concrete and 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 Attachment A following the text in this report. The approximate location of the boreholes are shown on the attached Site Plan, Figure 2.

SUBSURFACE CONDITIONS

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 location 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. 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 ground surface elevation at the boreholes were determined, in the field, relative to a site topographic survey completed by Kollaard Associates Inc using a geodetic datum. The site benchmark is described as a nail in a hydro pole located at the north/east corner of the site, west of Hiram drive. The elevation of the benchmark is 99.79 metres geodetic datum. This benchmark was transferred to this site during the topographic survey from the former benchmark used for 6811 & 6805 Hiram Drive. The former benchmark was described as a nail in a hydro pole located east of Hiram Drive, opposite 6811 Hiram Drive. The elevation of the former benchmark is 99.91 metres geodetic datum.

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

Fill

From the surface, fill materials were encountered at both boreholes. At borehole BH1, the fill materials consisted of about 600 millimetres of grey crushed stone. Beneath the crushed stone and from the surface at borehole BH2, fill materials consisting of yellow brown silty sand and/or grey brown sand with a trace of clay, brick, concrete and organics was encountered. The fill materials ranged in thickness from about 0.3 to 0.55 metres and were encountered from the ground surface to about 0.9 metres below the existing ground surface. The fill materials were augered through at borehole BH1. The results of standard penetration testing carried out in the fill materials at borehole BH2 was 23 blows per 0.3 metres, indicating a compact state of packing. The fill materials were fully penetrated at both borehole locations.

Topsoil

Beneath the fill materials at borehole BH2, a layer of topsoil was encountered. The topsoil consists of dark brown to black sandy silt and has a thickness of about 0.3 metres. The material was classified as topsoil based on the colour and the presence of organic materials. The identification of the topsoil layer is for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustainable plant growth.

Silty Sand

A thin layer of grey brown silty sand was encountered below the topsoil at borehole BH2 with a thickness of about 0.3 metres below the existing ground surface.

Silty Clay

A deposit of grey silty clay was encountered below the silty sand layer at borehole BH2 with a thickness of 1.15 metres below the existing ground surface.

The results of the in situ vane shear testing (ASTM D2573 - Standard test method for Field vane shear test in cohesive soil) gave undrained shear strength values of greater than 120 kilopascals. The field testing apparatus was limited to a maximum undrained shear of 120 kilopascals. Since the capacity of the silty clay tested was in excess of this limit of the field testing apparatus, no residual shear strength values were obtained.

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 very stiff in consistency.

The results of Atterberg Limits tests conducted on a soil sample of silty clay (BH2-SS3-5'-7') are presented in the following table and in Attachment A at the end of the report. The tested silty clay sample classifies as inorganic clays of medium plasticity (CI) 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 (%)**
BH2-SS3	1.5 - 2.1	47.8	22.2	25.7	28.1

LL: Liquid Limit PL: Plastic Limit PI: Plasticity Index w: water content
CI: Silty Clay of Medium Plasticity

Silt

A deposit of grey silt was encountered below the fill materials at borehole BH1 and beneath the silty clay layer at borehole BH2 at depths of about 0.9 and 4.8 metres, respectively, below the existing ground surface. The silt layer was fully penetrated at both boreholes at depths of about 7.18 and 7.46 metres and found to be about 2.6 to 4.9 metres in thickness. The results of the standard penetration tests carried out in the silt gave N values of about 4 to 23 blows per 0.3 metres of penetration, indicating a loose to compact state of compaction.

It is noted that the standard penetration tests were carried out with a drill rig equipped with a safety hammer with automatic release. The safety hammer was calibrated by the manufacturer to have an energy correction ratio of 1.0 that is: C_E = ER/60 = 1.0. Since the energy correction ratio is 1, the N values measured in the field correspond to N(60) values.

The results of two hydrometer tests (ASTM D422 and D2216) of soil samples (BH1-SS4 and BH2-SS8) indicate the samples have a silt/clay content of about 89 to 93 percent of which about 7 and 6 percent, respectively, is clay sized particles. The results indicate a sand content of 6.3 and 9.6 percent. The results are located in Attachment A.

Sand

Beneath the silt at borehole BH1, grey fine to medium was encountered at a depth of about 7.2 metres below the existing ground surface. The results of the standard penetration tests carried out in the sand gave N values of about 15 to 21 blows per 0.3 metres of penetration, indicating a compact state of compaction. Borehole BH1 was terminated in the sand layer.

Glacial Till

Glacial till was encountered beneath the silt layer at BH2. The glacial till consisted of gravel, cobbles and boulders, in a matrix of grey silty sand with a trace to some silty clay.

BH2 encountered glacial till at a depth of about 7.46 metres and was continued by dynamic cone penetration testing. The dynamic cone penetration test carried out at BH2 gave values ranging from 17 to 150 blows per 0.3 metres between the depths of 7.46 and 11.51 metres below the existing ground surface. At a depth of some 11.51 metres below the existing ground surface at borehole 2, refusal to cone penetration was encountered. It is considered likely that the refusal to cone penetration indicates either large boulders or bedrock in borehole BH2 at about 11.51 metres.

Groundwater

All of the boreholes encountered water at the time of the field work. A trace to some water was encountered at the boreholes at depths of about 1.4 to 1.5 metres below the existing ground surface. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

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 %	<0.0005	Negligible concern
pH	5.0 < pH	7.69	Neutral / Slightly Basic Negligible concern
Resistivity	R < 1500 ohm-cm	6090	Moderately corrosive
Sulphates (SO_4)	SO_4 > 0.1%	0.0046	Negligible concern

The results of the laboratory testing of a soil sample for sulphate gave a percent sulphate of less than 0.01. The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete based on percent sulphate in soil. From 0 to 0.10 percent the potential is negligible, from 0.10 to 0.20 percent the potential is mild but positive, from 0.20 to 0.50 percent the potential is considerable and 0.50 percent and greater the potential is severe. Based on the above, the soils are considered to have a negligible potential for sulphate attack on buried 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 7.69, 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 present negligible concrete corrosion potential.

Corrosivity Rating for soils ranges from extremely corrosive with a resistivity rating <1000 ohm-cm to moderately corrosive with a resistivity of 5000 to 10,000 ohm-cm to non-corrosive with a resistivity of >20,000 ohm-cm. The Soil resistivity was found to be 6090 ohm-cm for the sample analyzed making the soil corrosive for buried steel. Consideration to increasing the specified strength and/or adding air entrainment into any reinforced concrete in contact with the soil should be given. Protection is required for reinforcement steel within the concrete foundation walls. Based on the chemical test results, Type GU General use Hydraulic Cement may be used for this proposed development. Special protection may be required for reinforcement steel within the concrete walls.

The laboratory results are presented at the end of this report.

PROPOSED LIGHT INDUSTRIAL BUILDING FOUNDATIONS

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.

Foundations for Proposed Light Industrial Building

A review of the structural drawings for the proposed building indicates that the foundation will consist of conventional cast in place concrete foundation bearing on spread footings. The strip footings will be 0.6 metres in width and the pad footings will be a maximum of 1.37 metres in width. A review of the proposed site grading plan indicates that the footings will be set at about 0.6 to 0.9 metres below the existing ground surface. The site grading plan indicates that the proposed grade raise at the site will be limited to about 1.0 metres.

Subsurface Conditions at the Underside of Footing Level

With the exception of the topsoil materials, the subsurface conditions encountered at the test holes advanced during the investigation are suitable for the support of the proposed building on conventional spread footing foundations placed on a native subgrade or on engineered fill placed on the native subgrade. The excavations for the foundation should be taken through any topsoil or otherwise deleterious material to expose the native, undisturbed silty sand, silty clay and/or silt.

It is expected that the subgrade immediately below the proposed footing level will consist of a thin layer of loose silt / sand underlain by compact silt / sand or stiff to very stiff silty clay. Once the excavation for the foundation is complete, the exposed subgrade should be inspected by a qualified geotechnical person. Should the subgrade consist of loose silt or sand, the subgrade should be sub-excavated to remove the loose material to a depth of 0.4 metres below the underside of footing elevation.

Conventional Spread Footing Foundations

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 foundation.

The proposed light industrial building, a maximum allowable bearing pressure of 90 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 180 kilopascals using ultimate limit states design, may be used for the design of conventional strip footings or pad footings, a minimum of 0.6 metres in width, founded on the compact silt, sand and/or silty clay or on a suitably constructed engineered pad placed on the silt, sand and/or silty clay.

The maximum total and differential settlement of the footings are expected to be less than 25 millimetres and 20 millimetres, respectively, using the above allowable bearing pressure and resistance. The above allowable bearing pressures and resistance are acceptable for a maximum grade raise of less than 1.5 metres above the existing ground surface.

The allowable bearing pressure was determined by the engineer in consideration of Burland and Burbidge method where the compressibility of the material underlying a foundation to a depth of 2B below the foundation is related to the foundation width by a compressibility index (I_c) and the average settlement s_i of a foundation is related to the foundation loading, footing size and compressibility index.
where:
$I_c = 1.71 / N^{1.4}$ and N = average N'(60) value over the depth range not corrected for overburden pressure.
$s_i = q\, B^{0.7}\, I_c$ where q is the bearing pressure and B is the footing width.
Rearranging the above equations gives $q = si / (B^{0.7} \times (1.71/N^{1.4}))$

Assuming any loose silt or sand immediately below the proposed underside of footing elevation has been removed and replaced with compacted granular material, the average corrected N value for the subsurface material encountered within 2 times the width of the largest footing below the USF elevation is N'(60) = 20. The minimum N'(60) value within this depth range is 5.

With a maximum allowable settlement for serviceability limit states design of 25 mm and a maximum footing width of 1.37 metres, an average N of 12 provides an allowable bearing capacity of 380 kPa. The minimum N value of 5 provides an allowable bearing capacity of 115 kPa. Assuming 80 percent of the allowable bearing capacity to accommodate long term settlement, the allowable bearing capacity for serviceability limit states design is recommended by the geotechnical engineer to be 90 kPa as indicated above.

The subgrade surface should be inspected and approved by geotechnical personnel prior to placement of any granulars.

Engineered Fill

Should the complete removal of all fill materials and topsoil and any otherwise deleterious material result in a subgrade below the proposed founding level, any fill required to raise the footings for the proposed building to founding level 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 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 foundations, the engineered fill should extend out from the outside edges of the footings for a horizontal distance of 0.5 metres and then down and out at a slope of 1 horizontal to 1 vertical, or flatter. The excavations for the structure 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 I or Type II are placed on a sand or silty clay subgrade above the normal ground water level. Should the subgrade surface consist of silt, a 4 ounce per square yard non woven geotextile fabric should be placed between the engineered fill and the silt subgrade. 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 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.

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.

The depth of frost cover could be reduced for footings bearing on engineered fill over silty clay. In this case, the combined thickness of earth cover and the engineered fill should be at least 1.5 metres for frost protection purposes. Alternatively, the required frost protection could be provided using a combination of earth cover and extruded polystyrene insulation. Detailed guidelines for footing insulation frost protection could be provided upon request.

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.

Foundation Wall Backfill and Drainage

Provided the proposed finished floor surfaces are everywhere above the exterior finished grade and the granular materials beneath the proposed floor slab are properly compacted no perimeter foundation drainage system is required.

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

The native soils encountered at this site are considered to be frost susceptible. As such, to prevent possible foundation frost jacking, the backfill against any unheated or insulated walls or isolated walls or piers should consist of free draining, non-frost susceptible material consisting of sand or sand and gravel meeting OPSS Granular B Type I grading requirements.

Alternatively, foundations could be backfilled on the exterior with native material in conjunction with the use of an approved proprietary drainage layer system (such as Platon System Membrane) against the foundation wall.

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. To mitigate this potential, the upper approximately 0.6 metres of the foundation should be backfilled with non-frost susceptible granular material.

Where the granular backfill will ultimately support a pavement structure or walkway, it is suggested that the 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.

Slab on Grade Support

As stated above, it is expected that the proposed building will be founded on native silty sand, silty clay and/or silt 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. Under slab drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level.

Seismic Design for the Proposed Light Industrial Building

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 C. The subsurface conditions below the proposed footing design level are indicated to consist of the following:

At borehole BH2 – Very stiff silty clay followed by about 2.6 m of compact Silt then glacial till with refusal to further penetration at a depth of 11.5 metres.

At borehole BH1 – Compact Silt followed by about 3.1 m of loose Silt then compact Sand to the depths explored.

Bedrock is indicated to be at a depth of 15.8 metres.

The shear strengths of the silty clay in excess of 50 kPa indicate that the silty clay has a seismic site response of Site Class C.

Consideration of non cohesive materials in Boreholes BH1 and BH2 provides the following:

Seismic Site Response Site Class Calculation

Borehole BH1						
Layer	Description	Depth (m)	d_i (m)	$N(60)_i$ (blows/0.3 m)	d_i/N_i (blows/0.3m)	Estimated Shear Wave Velocity (m/s)
	USF	0.9				
1	SILT	0.9	2.9	22	0.133	
2	SILT	3.8	3.1	7	0.428	
3	SILT	6.9	0.7	20	0.428	
4	SAND/TILL	7.6	3.9	15	0.307	
5	TILL	11.5	4.3	100	0.043	
6	Bedrock	15.8	15.1	N/A		
$sum(d_i/N(60)_i)$					0.899	
$d_c/(sum(d_i/N(60)_i)$					16.6	
Since N(60) = 15 < 16.6 < 50 (site class D)						180
Limestone Bedrock						800 to 1800
Sandstone Bedrock						2000 to 3300

Since the seismic site classification is based on the average properties of the top 30 metres below the underside of the foundation, the site classification will include both the overburden and the bedrock. In order to correlate the overburden and bedrock information, the minimum shear wave velocity provided for a site class D soil was obtained from Table 4.1.8.4.A of the Ontario Building Code 2012 (as updated) to be 180 m/s. The bedrock at the site is limestone with sandstone. The minimum expected shear velocity for the bedrock at the site is 800 m/s. The weighted average shear wave velocity of the upper 30 metres is equal to 473 m/s which corresponds to a seismic site classification of site class C.

National Building Code Seismic Hazard Calculation

The online 2015 National Building Code Seismic Hazard Calculation was used to verify the seismic conditions at the site. The design Peak Ground Acceleration (PGA) for the site was calculated as 0.309 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The seismic site classification for the site is indicated to be Seismic Site Class C. The results of the calculation are attached following the text of this report.

Potential for Soil Liquefaction

Consideration for the potential for soil liquefaction was determined by considering the ratio between the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR) for the soils between the proposed underside of footing level and the depth explored by standard penetration testing. The CRR value was determined from a mathematical expression as determined by Rauch (1998) (copied below).

Rauch (1998) proposed the following equation for determining CRR based on SPT N-value $(N_1)_{60}$ for an earthquake of magnitude 7.5.

$$\mathrm{CRR} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{\left[10(N_1)_{60}+45\right]^2} - \frac{1}{200} \qquad (6)$$

where $(N_1)_{60}$ refers to SPT blow count normalized to an overburden pressure of approximately 100 kPa and a hammer efficiency of 60%.

$(N_1)_{60}$ must also be corrected for fines as per the following equation proposed by Seed and Idriss(1971):

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \qquad (5)$$

where α and β = coefficients determined from the following relationships:

$$\alpha = 0 \quad \text{for FC} \leq 5\% \qquad (6a)$$

$$\alpha = \exp[1.76 - (190/\text{FC}^2)] \quad \text{for } 5\% < \text{FC} < 35\% \qquad (6b)$$

$$\alpha = 5.0 \quad \text{for FC} \geq 35\% \qquad (6c)$$

$$\beta = 1.0 \quad \text{for FC} \leq 5\% \qquad (7a)$$

$$\beta = [0.99 + (\text{FC}^{1.5}/1{,}000)] \quad \text{for } 5\% < \text{FC} < 35\% \qquad (7b)$$

$$\beta = 1.2 \quad \text{for FC} \geq 35\% \qquad (7c)$$

And

$$\mathrm{CRR}_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (4)$$

This equation is valid for $(N_1)_{60} < 30$. For $(N_1)_{60} \geq 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable. This equation may be used in spreadsheets and other analytical techniques to approximate the clean-sand base curve for routine engineering calculations.

The CSR was determined from Seed and Idriss (1971).

$$CSR_{peak} = \left(\frac{a_{\max}}{g}\right) \cdot \left(\frac{\sigma_v}{\sigma'_v}\right) \cdot (r_d) \quad \text{(Eq. 1)}$$

where

a_{max} = the peak horizontal ground surface acceleration,
g = the acceleration of gravity,
σ_v = total vertical stress,
σ'_v = effective vertical stress, and
r_d = the nonlinear shear mass participation factor.

r_d = 1 – 0.0.015(z-4) for z > 4 m and r_d= 1 for z <= 4 m. (Idriss & Boulanger 2006)

The equation for Factor of safety is written as follows:

$$\mathrm{FS} = (\mathrm{CRR}_{7.5}/\mathrm{CSR})\mathrm{MSF}$$

Where $CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes and MSF is the magnitude scaling factor

$$\mathrm{MSF} = 10^{2.24}/M_w^{2.56}$$

The calculations for the non-cohesive materials encountered at the boreholes are summarized in the following table:

Depth	Average Raw N	N(60)	N(60)cs	CSR	$CRR_{7.5}$	MSF M = 6.5	$CRR_{6.5}$	FS
0.76	6	9	16	0.205	0.170	1.44	0.245	1.2
1.52	24	35	47					
2.29	15	20	29	0.254	0.393	1.44	0.566	2.2
3.05	19	23	33	0.288	1.222	1.44	1.762	6.1
3.81	6	7	14	0.313	0.150	1.44	0.217	0.7
4.57	12.5	15	22	0.328	0.242	1.44	0.350	1.1
5.33	15.5	18	26	0.339	0.309	1.44	0.446	1.3
6.10	13.5	15	23	0.347	0.255	1.44	0.367	1.1
6.86	15	17	25	0.352	0.285	1.44	0.412	1.2
7.62	18	20	29	0.356	0.398	1.44	0.573	1.6
8.38	29	32	44					
Average				0.309	0.380		0.549	1.8

It is noted that the CSR and CRR values were not computed for N(60) > 30
The average factor of safety against liquefaction for the soils assessed for an earthquake with a magnitude of 7.5 is 0.380 / 0.309 = 1.2.

Since the factor of safety against liquefaction of the soils below the proposed building is 1.2, there is no danger to the proposed development resulting from liquefaction of the subsurface soils.

ACCESS ROADWAY AND PARKING LOT PAVEMENTS

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 area. The exposed subgrade surface should then be proof inspected and approved by geotechnical personnel. 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.

Parking Area Structure

A review of the proposed site grading plan indicates that the finished elevation of the asphalt surfaced area and granular surfaced yard area will be between an estimated 0.2 and 0.7 metres above the existing ground surface. In addition, the borehole logs indicate that there is between 0.5 and 0.6 metres of fill overlying a 0.3 m thick topsoil layer at the site. In general, the fill material consists of a mixture of granular materials consisting of sand, gravel and crushed stone.
The fill materials should be stripped from the proposed parking, roadway and yards areas. The fill material can be stockpiled for reuse upon approval by the geotechnical engineer or qualified representative of the geotechnical engineer. Following removal of the fill, the underlying topsoil layer should be stripped to expose the native undisturbed subgrade surface.
Following approval of the subgrade surface by the geotechnical engineer or qualified representative of the geotechnical engineer, the granular fill materials can be replaced below the proposed asphalt

and granular surfaced structures to raise the subgrade to the underside of subbase elevation. The fill should be placed and compacted to a minimum of 95 percent standard proctor maximum dry density.

Granular Surfaced Yard Area

It is suggested that provision be made for the following minimum pavement structure:

200 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase over
(50 or 100 millimetre minus crushed stone)
Non-woven geotextile fabric (4 oz/sqy) such as Terrafix 270R or Thrace-Ling 130EX or approved alternative.

Asphalt Surfaced Areas

For pavement areas subject to cars and light trucks the pavement should consist of:

50 millimetres of Superpave 12.5 hot mix 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)
Non-woven geotextile fabric (4 oz/sqy) 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 subgrade, that is, one where the subgrade has been inspected and approved and any fill placed over the subgrade has been adequately compacted. If the subgrade 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.

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.

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 building should be inspected by Kollaard Associates Inc. to ensure that a suitable subgrade 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 access roadway and parking areas should be inspected and approved by geotechnical personnel. In situ density testing should be carried out on the roadway granular materials to ensure the materials meet the specifications from a compaction point of view.

The native topsoil, silty sand, silt and 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
29.JAN.2020
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

Attachments: Table I - Record of Boreholes
Key Plan, Figure 1
Site Plan, Figure 2
Laboratory Test Results for Sulphate, Resistivity and pH
Attachment A – Stantec Laboratory Test Results for Soils
Attachment B - National Building Code Seismic Hazard Calculation
Attachment C - Ontario Water Well Record

Reference Papers:
T.L.Youd and I.M.Idriss, "Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" Journal of Geotechnical and Geoenvironmental Engineering April 2001.

Syed M. Ali Jawaid "Comparison of Liquefaction Potential Evaluation Based on Different Field Tests" 2010 – Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics.

Allan F. Rauch, EPOLLS: An Empirical Method for Predicting Surface Displacements Due to Liquefaction-Induced Lateral Spreading in Earthquakes Chapter 7 – Analysis of Soil Borings for Liquefaction Resistance" Virginia Polytechnic Institute and State University, May 5 1997

RECORD OF BOREHOLE BH1

PROJECT: Proposed Light Industrial Development
CLIENT: Mr. Nat Guist
LOCATION: 6793 Hiram Drive, Greely, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180938
DATE OF BORING: April 2, 2019
SHEET 1 of 1
DATUM: Geodetic

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
	Ground Surface		99.75							
0	Grey crushed stone (FILL)		0.00 / 99.15							
	Yellow brown silty sand, trace clay and organics (FILL)		0.60 / 98.85							
1	Grey brown SILT, trace sand		0.90	1	SS	6				
2			97.45	2	SS	25				Water observed in borehole at approximately 1.5 metres below the existing ground surface on April 2, 2019.
	Grey SILT, trace to some sand and clay seams		2.30	3	SS	15				
3				4	SS	19				
4				5	SS	6				
5				6	SS	4				
6				7	SS	8				
				8	SS	7				
7			92.57	9	SS	15				
	Grey fine to medium SAND, trace silt		7.18							
8			91.53	10	SS	21				
	End of Borehole		8.22							

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 Light Industrial Development
CLIENT: Mr. Nat Guist
LOCATION: 6793 Hiram Drive, Greely, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

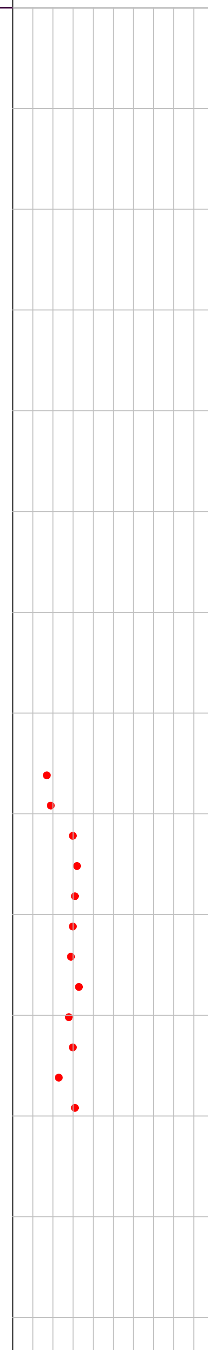
PROJECT NUMBER: 180938
DATE OF BORING: April 2, 2019
SHEET 1 of 1
DATUM: Geodetic

DEPTH SCALE (meters)	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (M)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface	99.74				
0	Grey brown sand, gravel, trace brick and concrete, organics (FILL)	0.00 99.19	1	SS	23	
	TOPSOIL	0.55 98.89				
1	Grey brown SILTY SAND	0.85 98.59	2	SS	6	
	Grey brown SILTY CLAY	1.15 97.44	3	SS	14	
2–4	Grey SILTY CLAY	2.30 94.92	4	SS	12	Water observed in borehole at approximately 1.4 metres below the existing ground surface on April 2, 2019.
			5	SS	10	
			6	SS	14	
5–7	Grey SILT, trace sand, and clay	4.82 92.28	7	SS	21	
			8	SS	23	
			9	SS	20	
			10	SS	15	
8–11	Borehole continued as Probe Hole, probably grey silt, then grey silty sand with some gravel, cobbles and boulders (GLACIAL TILL)	7.46 88.23				
12–13	End of Borehole, Practical refusal on large boulder or bedrock	11.51				

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

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

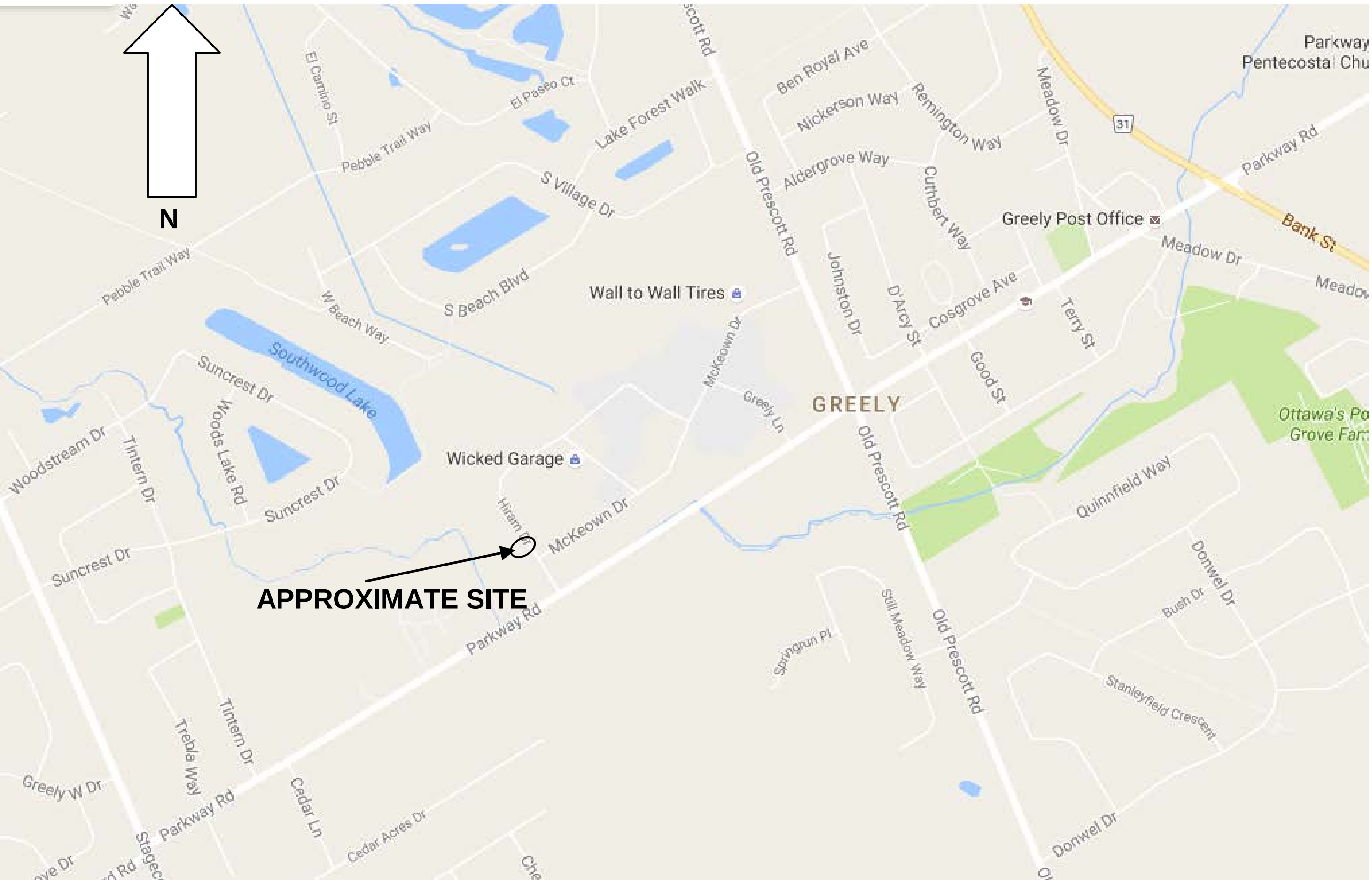
Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to100
Very Stiff	over100

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. 180938

Date April 2019

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:
MR. NAT GIUST

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED WAREHOUSE BUILDING

LOCATION:
6793 HIRAM DRIVE, GREELY
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
--	MARCH 18, 2019
DRAWN BY: DT	SCALE: N.T.S

KOLLAARD FILE NUMBER:
180938

Attachment A
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: 04-APR-19
Report Date: 09-APR-19 13:53 (MT)
Version: FINAL

Client Phone: 613-860-0923

Certificate of Analysis

Lab Work Order #: L2254055

Project P.O. #: NOT SUBMITTED
Job Reference: 180938
C of C Numbers:
Legal Site Desc:

Melanie Moshi
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2254055-1 BH1 SS2 5'-7' Sampled By: CLIENT on 02-APR-19 Matrix: WATER							
Physical Tests							
Conductivity	0.164		0.0040	mS/cm		09-APR-19	R4592787
% Moisture	14.0		0.10	%	05-APR-19	05-APR-19	R4591310
pH	7.69		0.10	pH units		05-APR-19	R4591292
Resistivity	6090		1.0	ohm*cm		09-APR-19	
Leachable Anions & Nutrients							
Chloride	<0.00050		0.00050	%	08-APR-19	08-APR-19	R4592880
Anions and Nutrients							
Sulphate	0.0046		0.0020	%	05-APR-19	05-APR-19	R4592140

* 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).

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	APHA 2510 B

Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	MOECC E3138

Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.

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.

** 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:

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.

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

Stantec

April 8, 2019
File: 122410003

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

Reference: **Kollaard File #180938**
ASTM D4318 Atterberg Limit & ASTM D2216 Moisture Content

The table below summarizes Atterberg Limit & Moisture Content results.

Source	Depth	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index
BH-2 SS3	5'-7'	28.1	47.8	22.2	25.7

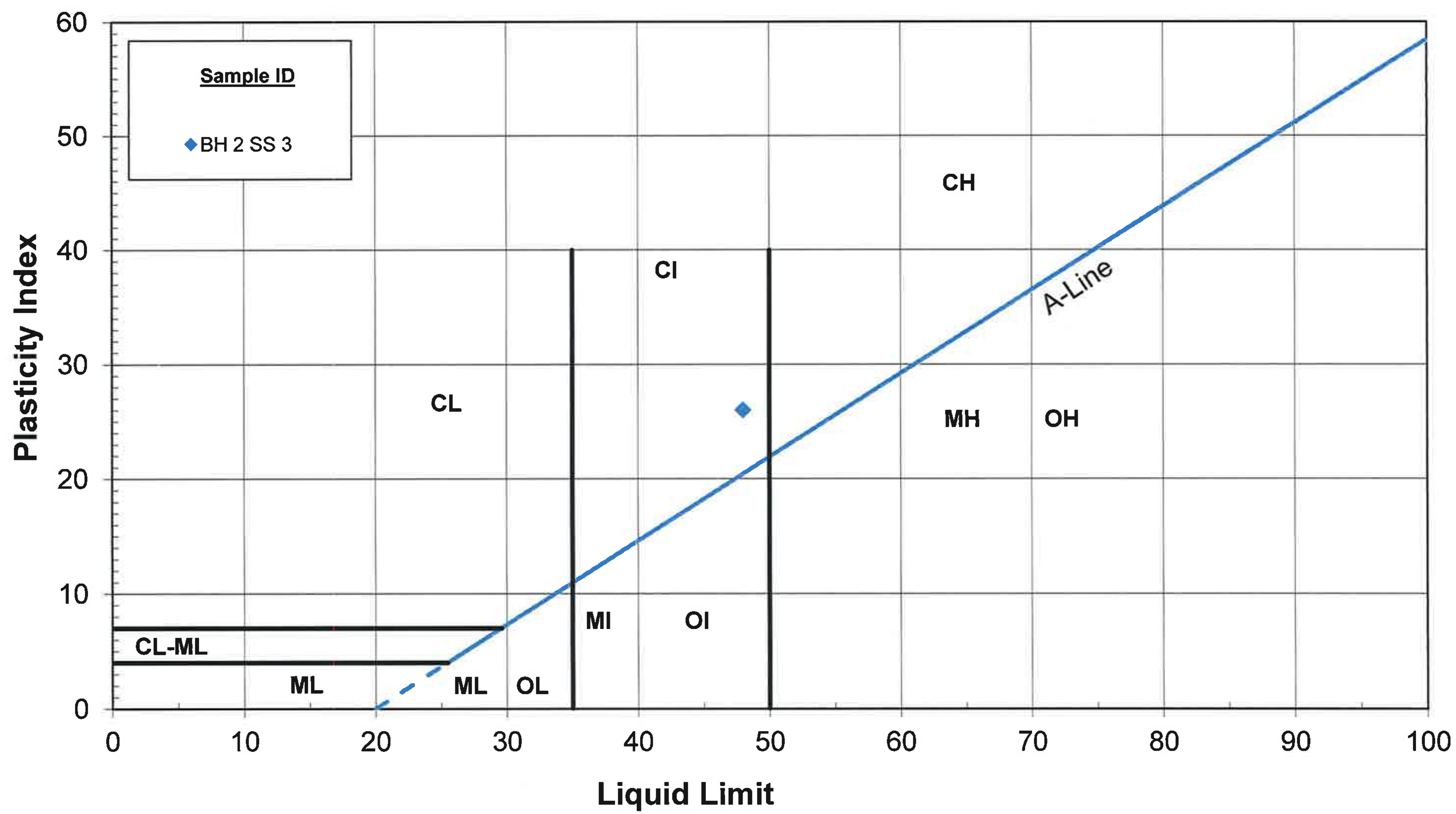
Sincerely,

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 #180938

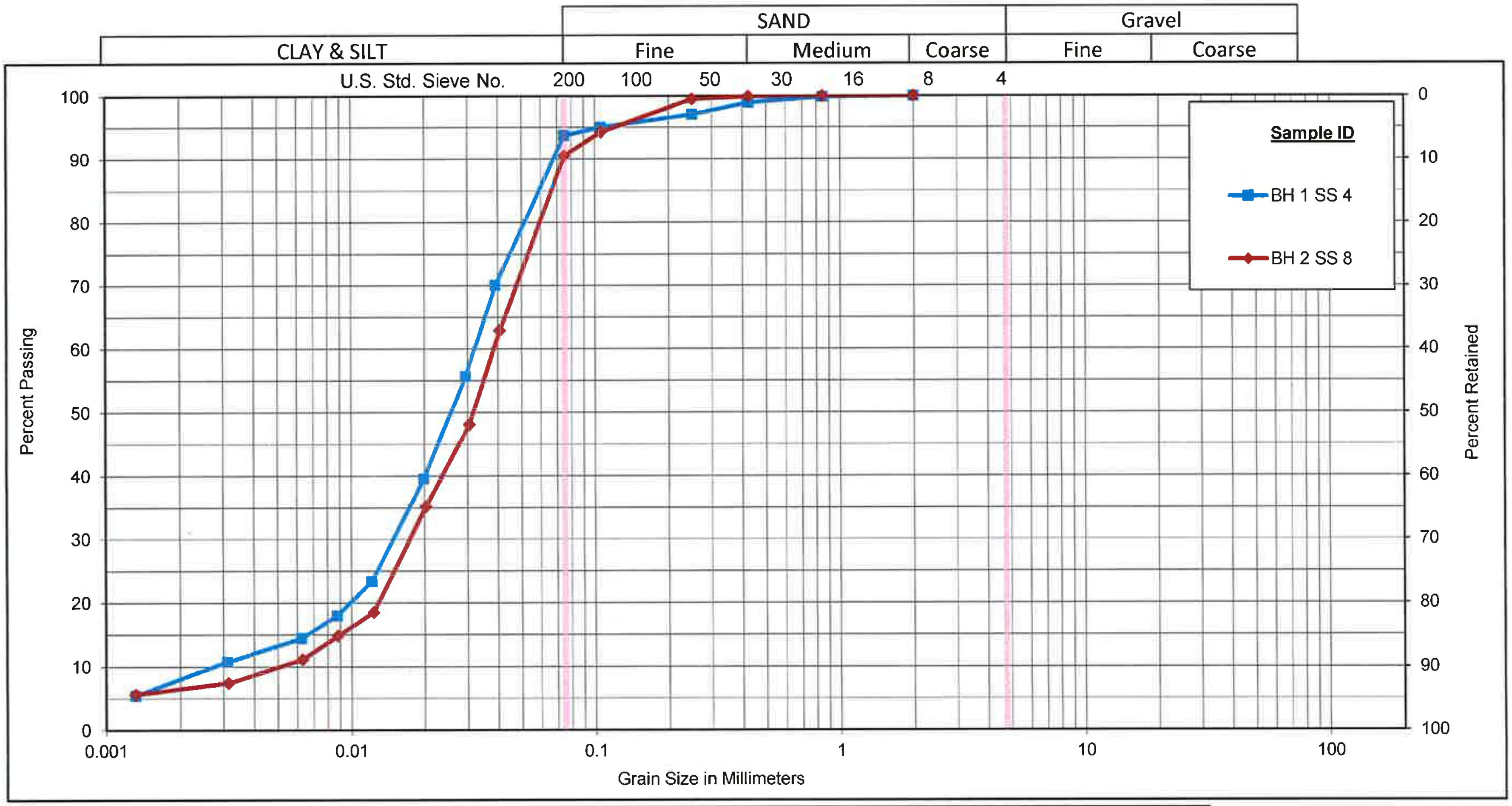
6793 Hiram Drive, Greely, ON

PLASTICITY CHART

Figure No.

Project No. 122410003

Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH 1 SS 4	10-12 ft.	0.0	6.3	86.7	7.0
BH 2 SS 8	17½-19½ ft.	0.0	9.6	84.4	6.0

Stantec

GRAIN SIZE DISTRIBUTION

Kollaard Associates File# 180938

6793 Hiram Drivbe, Greely ON

Figure No.

Project No. 122410003

Stantec

Particle-Size Analysis of Soils

LS702

ASTM D422

PROJECT DETAILS

Client:	Kollaard Associates, File #180938	Project No.:	122410003
Project:	6793 Hiram Drive, Greely, ON	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates
Source:	BH 1	Date Sampled:	April 3, 2019
Sample No.:	SS 4	Tested By:	Daniel Boateng
Sample Depth	10-12 ft.	Date Tested:	April 5, 2019

WASH TEST DATA

Oven Dry Mass in Hydrometer Analysis (g)	54.52
Sample Weight after Hydrometer and Wash (g)	3.62
Percent Passing No. 200 Sieve (%)	93.4
Percent Passing Corrected (%)	93.36

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	166.50
Sample Weight After Sieve (g)	166.50
Percent Loss in Sieve (%)	0.00

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	40	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	111.53
Air Dried Mass (W_a), (g)	111.73
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9982
Air Dried Mass in Analysis (M_a), (g)	54.62
Oven Dried Mass in Analysis (M_o), (g)	54.52
Percent Passing 2.0 mm Sieve (P_{10}), (%)	100.00
Sample Represented (W), (g)	54.52

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]	166.50	
0.850	0.08	99.85
0.425	0.58	98.94
0.250	1.63	97.01
0.106	2.73	94.99
0.075	3.46	93.65
PAN	3.60	

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 8:00 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
05-Apr-19	8:01 AM	1	46.0	7.0	22.0	39.0	69.98	9.08404	9.61570	0.012970	0.03909
05-Apr-19	8:02 AM	2	38.0	7.0	22.0	31.0	55.63	10.32404	9.61570	0.012970	0.02947
05-Apr-19	8:05 AM	5	29.0	7.0	22.0	22.0	39.48	11.71904	9.61570	0.012970	0.01986
05-Apr-19	8:15 AM	15	20.0	7.0	22.0	13.0	23.33	13.11404	9.61570	0.012970	0.01213
05-Apr-19	8:30 AM	30	17.0	7.0	22.0	10.0	17.94	13.57904	9.61570	0.012970	0.00873
05-Apr-19	9:00 AM	60	15.0	7.0	22.0	8.0	14.36	13.88904	9.61570	0.012970	0.00624
05-Apr-19	12:10 PM	250	13.0	7.0	21.5	6.0	10.7667	14.19904	9.73081	0.013047	0.00311
06-Apr-19	8:00 AM	1440	10.0	7.0	21.5	3.0	5.3833	14.66404	9.73081	0.013047	0.00132

Remarks:

Reviewed By: Brian Prevost

Date: April 8/2019

V:\01216\active\laboratory_standing_offers\2019 Laboratory Standing Offers\122410003 Kollaard Associates Engineers\April 3, One MC, Lim, & Two Hydros, Kollaard# 180938\Hydrometer Analysis MTO Projects May2014.xlsx

Stantec

Particle-Size Analysis of Soils
LS702
ASTM D422

PROJECT DETAILS

Client:	Kollaard Associates, File #180938	Project No.:	122410003
Project:	6793 Hiram Drive, Greely, ON	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates
Source:	BH 2	Date Sampled:	April 3, 2019
Sample No.:	SS 8	Tested By:	Daniel Boateng
Sample Depth	17½-19½ ft.	Date Tested:	April 5, 2019

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	52.96
Sample Weight after Hydrometer and Wash (g)	5.60
Percent Passing No. 200 Sieve (%)	89.4
Percent Passing Corrected (%)	89.43

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	191.50
Sample Weight After Sieve (g)	191.50
Percent Loss in Sieve (%)	0.00

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	40	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	138.30
Air Dried Mass (W_a), (g)	138.38
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9994
Air Dried Mass in Analysis (M_a), (g)	52.99
Oven Dried Mass in Analysis (M_o), (g)	52.96
Percent Passing 2.0 mm Sieve (P_{10}), (%)	100.00
Sample Represented (W), (g)	52.96

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]	191.50	
0.850	0.01	99.98
0.425	0.06	99.89
0.250	0.28	99.47
0.106	3.07	94.20
0.075	5.06	90.45
PAN	5.60	

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 8:02 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
5-Apr-19	8:03 AM	1	41.0	7.0	22.0	34.0	62.81	9.85904	9.61570	0.012970	0.04072
5-Apr-19	8:04 AM	2	33.0	7.0	22.0	26.0	48.03	11.09904	9.61570	0.012970	0.03055
5-Apr-19	8:07 AM	5	26.0	7.0	22.0	19.0	35.10	12.18404	9.61570	0.012970	0.02025
5-Apr-19	8:17 AM	15	17.0	7.0	22.0	10.0	18.47	13.57904	9.61570	0.012970	0.01234
5-Apr-19	8:32 AM	30	15.0	7.0	22.0	8.0	14.78	13.88904	9.61570	0.012970	0.00882
5-Apr-19	9:02 AM	60	13.0	7.0	22.0	6.0	11.08	14.19904	9.61570	0.012970	0.00631
5-Apr-19	12:12 PM	250	11.0	7.0	21.5	4.0	7.39	14.50904	9.73081	0.013047	0.00314
6-Apr-19	8:02 AM	1440	10.0	7.0	21.5	3.0	5.54	14.66404	9.73081	0.013047	0.00132

Remarks:

Reviewed By: Brian Prevost
Date: April 9/2019

ATTACHMENT B

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: 45.255982N 75.578729W **User File Reference:** 6793 Hiram Drive

2019-04-12 19:10 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)	0.503	0.272	0.158	0.044
Sa (0.1)	0.584	0.327	0.198	0.061
Sa (0.2)	0.483	0.275	0.169	0.055
Sa (0.3)	0.364	0.208	0.130	0.044
Sa (0.5)	0.255	0.146	0.091	0.031
Sa (1.0)	0.124	0.072	0.046	0.015
Sa (2.0)	0.058	0.033	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)	0.309	0.176	0.107	0.033
PGV (m/s)	0.211	0.117	0.070	0.021

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

Natural Resources Canada Ressources naturelles Canada

Canada

ATTACHMENT C

Ontario Water Well Record

Ontario | Ministry of the Environment and Climate Change

Measurements recorded in: ☐ Metric ☒ Imperial

Well Tag Number (Place Sticker and Print Number Below): Tag#:A260947

A260947

Well Record

Regulation 903 Ontario Water Resources Act

Page ____ of ____

Well Owner's Information

First Name	Last Name / Organization	E-mail Address	☐ Well Constructed by Well Owner
	Watko Construction Ltd.		

Mailing Address (Street Number/Name)	Municipality	Province	Postal Code	Telephone No. (inc. area code)
811 Kennedy Road	Kemptville	ON	K0G 1J0	

Well Location

Address of Well Location (Street Number/Name)	Township	Lot	Concession
6793 Hiram Drive	Osgoode	P/L 5	4

County/District/Municipality	City/Town/Village	Province	Postal Code
Ottawa Carleton	Greely	Ontario	

UTM Coordinates	Zone	Easting	Northing	Municipal Plan and Sublot Number	Other
NAD 8 3	18	454576	5011571	4M-351	PT Blk 7

Overburden and Bedrock Materials/Abandonment Sealing Record *(see instructions on the back of this form)*

General Colour	Most Common Material	Other Materials	General Description	Depth (m/ft) From	To
	Sand	+ Clay Gravel		0'	52'
Grey	Limestone			52'	120'
Grey	Limestone	W/ Grey Sandstone Mix		120'	152'
Grey	Limestone	W/ Grey Sandstone Mix		152'	162'

Annular Space

Depth Set at (m/ft) From	To	Type of Sealant Used (Material and Type)	Volume Placed (m³/ft³)
58'	48'	Neat cement	9.36
48'	0'	Bentonite slurry	16.8

Results of Well Yield Testing

After test of well yield, water was: ☐ Clear and sand free ☐ Other, specify Not tested

If pumping discontinued, give reason: X

Pump intake set at (m/ft): 100

Pumping rate (l/min / GPM): 15

Duration of pumping: 1 hrs + 0 min

Final water level end of pumping (m/ft): 38.3'

If flowing give rate (l/min / GPM): X

Recommended pump depth (m/ft): 100'

Recommended pump rate (l/min / GPM): 15

Well production (l/min / GPM): [redacted] 15

Disinfected? ☒ Yes ☐ No

Draw Down Time (min)	Water Level (m/ft)	Recovery Time (min)	Water Level (m/ft)
Static Level	17'4"		38.3'
1	25.6	1	31.5
2	28.1	2	26.7
3	29.8	3	22.3
4	30.7	4	19.7
5	31.8	5	18.1
10	34.7	10	17.4
15	36.2	15	17.4
20	37.9	20	17.4
25	38	25	17.4
30	38.1	30	17.4
40	38.3	40	17.4
50	38.3	50	17.4
60	38.3'	60	17.4'

Method of Construction

☐ Cable Tool ☐ Diamond ☐ Rotary (Conventional) ☐ Jetting ☐ Rotary (Reverse) ☐ Driving ☐ Boring ☐ Digging ☒ Air percussion ☐ Other, specify

Well Use

☐ Public ☒ Domestic ☐ Livestock ☐ Irrigation ☐ Industrial ☐ Other, specify ☐ Commercial ☐ Municipal ☐ Test Hole ☐ Cooling & Air Conditioning ☐ Not used ☐ Dewatering ☐ Monitoring

Construction Record - Casing

Inside Diameter (cm/in)	Open Hole OR Material (Galvanized, Fibreglass, Concrete, Plastic, Steel)	Wall Thickness (cm/in)	Depth (m/ft) From	To
6 1/4"	Steel	.188"	+2'	58'
5 7/8"	Open Hole		58'	162'

Status of Well

☒ Water Supply ☐ Replacement Well ☐ Test Hole ☐ Recharge Well ☐ Dewatering Well ☐ Observation and/or Monitoring Hole ☐ Alteration (Construction) ☐ Abandoned, Insufficient Supply ☐ Abandoned, Poor Water Quality ☐ Abandoned, other, specify ☐ Other, specify

Construction Record - Screen

Outside Diameter (cm/in)	Material (Plastic, Galvanized, Steel)	Slot No.	Depth (m/ft) From	To

Water Details

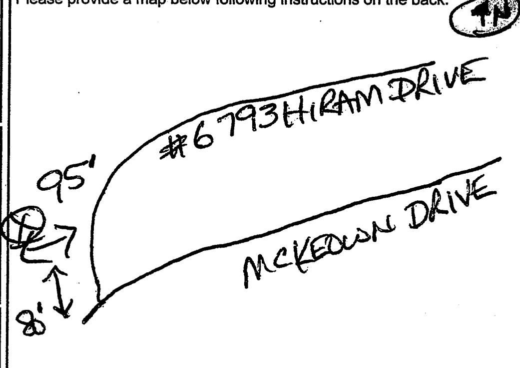
Water found at Depth	Kind of Water
152 (m/ft) ☐ Gas	☐ Fresh ☒ Untested ☐ Other, specify
(m/ft) ☐ Gas	☐ Fresh ☐ Untested ☐ Other, specify
(m/ft) ☐ Gas	☐ Fresh ☐ Untested ☐ Other, specify

Hole Diameter

Depth (m/ft) From	To	Diameter (cm/in)
0'	58'	9 3/4"
58'	162'	5 7/8"

Map of Well Location

Please provide a map below following instructions on the back.



Well Contractor and Well Technician Information

Business Name of Well Contractor	Well Contractor's Licence No.
Air Rock Drilling Co. Ltd.	1119

Business Address (Street Number/Name)	Municipality
6659 Franktown Road, RR#1	Richmond

Province	Postal Code	Business E-mail Address
ON	K0A 2Z0	air-rock@sympatico.ca

Bus. Telephone No. (inc. area code)	Name of Well Technician (Last Name, First Name)
6138382170	Hogan, Dan

Well Technician's Licence No.	Signature of Technician and/or Contractor	Date Submitted
T3058		2019 03 31

Comments: 3/4 HP 15 GPM Set @ 100 Ft

Well owner's information package delivered: ☒ Yes ☐ No

Date Package Delivered: 2019 03 22

Date Work Completed: 2019 03 19

Ministry Use Only

Audit No. Z302592

Received

0506E (2014/11)

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