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REPORT ON

**GEOTECHNICAL INVESTIGATION
PROPOSED LIGHT INDUSTRIAL DEVELOPMENT
2375 ST. LAURENT BOULEVARD
CITY OF OTTAWA, ONTARIO**

Project # 170549

Submitted to:

2465070 Ontario Ltd.
c/o Luc Sabourin
1100 Meadowshire Way
Manotick, Ontario K4M 0A5

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August 9, 2017

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2465070 Ontario Ltd.
c/o Luc Sabourin
1100 Meadowshire Way
Manotick, Ontario K4M 0A5

RE: GEOTECHNICAL INVESTIGATION
PROPOSED LIGHT INDUSTRIAL DEVELOPMENT
2375 ST. LAURENT BOULEVARD
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed light industrial development. 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 929 square metre (10,000 square feet) pre-engineered building with about a 100 square foot mezzanine with a future expansion of 929 square metre (10,000 square feet) at 2375 St. Laurent Boulevard in the City of Ottawa, Ontario (see Key Plan, Figure 1). The site has a total of about 2.28 hectares (5.64 acres) and is currently undeveloped. The property is grassed surfaced. The site has about 179 metres of frontage onto St. Laurent Boulevard.

Preliminary plans indicate that the proposed building will be of steel framed construction with cast in place concrete foundations and a concrete slab on grade floor.

The site is located within an area of light industrial development. The site is bordered on the west and east by industrial development (Promaxis and Advanced Business Interiors, respectively), on the north by St. Laurent Boulevard followed by industrial development (City of Ottawa Public health laboratory and Canadian Army 33rd Service Battalion armoury) and on the south by a hydro corridor that is also used as park space. Currently, the site is in an undeveloped condition and consists of an open grassy area.

The site is located on the south side of St. Laurent Boulevard about 220 metres west of the intersection of St. Laurent Boulevard and Thurston Drive, City of Ottawa, Ontario.

Based on a review of available borehole and well record information from the Ecolog ERIS report for the Phase I Environmental Site Assessment for the site, the overburden at and near the site likely consists of a thin sand layer (0.3 metres in thickness) followed by red brown to grey silty clay to depths of up to 12 metres, followed by glacial till with bedrock occurring at about 14.6 metres below the ground surface. A review of the bedrock geology map indicates that the bedrock underlying the site consists of grey shale of the Carlsbad Formation.

PROCEDURE

The field work for this investigation was carried out on July 27, 2017 at which time three boreholes, numbered BH1, BH2 and BH3 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.

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 to depths ranging from about 5.5 to 7.0 metres below the existing ground surface (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).

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

Two soil samples (BH1 and BH2) were submitted for Atterberg Limits and Moisture Content testing (ASTM D4318 and ASTM D2216). A sample of soil obtained from BH1 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 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. 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.

Topsoil

A thin layer of topsoil (about 150 millimetres in thickness) was encountered, from the ground surface, at all of the boreholes. The topsoil 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. The topsoil was fully penetrated at the borehole locations.

Silty Clay

A deposit of grey brown to grey silty clay was encountered below the topsoil 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 23 kilopascals to 48 kilopascals. 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 soft to firm in consistency. The boreholes were terminated in the silty clay at depths of 9.75 metres below the existing ground surface. Borehole BH1 was advanced through the silty clay by dynamic cone penetration testing to refusal at 14.88 metres below the existing ground surface. Based on the increase in blow counts per 300 mm it is assumed that the silty clay deposit is underlain by glacial till at about 13.4 metres below the existing ground surface.

The results of Atterberg Limits and moisture content tests conducted on two soil samples of silty clay are presented in Table I and in Attachment A at the end of the report. The tested silty clay sample classifies as inorganic clays of high plasticity (CH) in accordance with the Unified Soil Classification System.

Table I – Atterberg Limit and Water Content Results

Sample	Depth(metres)	LL (%)	PL (%)	PI (%)	W (%)
BH1-SS6	4.57 - 5.18	68.3	25.4	43.0	74.5
BH2-SS3	1.52 - 2.13	74.2	28.9	45.3	45.4

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

CH: Clay of High Plasticity

The results are located in Attachment A.

Glacial Till / Bedrock

BH1 was continued by dynamic cone penetration testing beginning at a depth of 9.75 metres below the existing ground surface. The dynamic cone penetration test carried out at BH1 gave values ranging from WH to greater than 50 blows per 0.3 metres. The dynamic cone penetration test values increased with depth below 13.4 metres and ranged from 6 to greater than 50 blows per 0.3 metres. At a depth of some 14.9 metres below the existing ground surface, refusal to cone penetration was encountered and indicates either large boulders or bedrock. It is considered likely that the increase in blow count at about 13.4 metres depth indicates the possible presence of glacial till materials.

Groundwater

Groundwater was encountered in boreholes BH2 and BH3 at time of drilling on July 27, 2017 at about 7.0 metres below the existing ground surface. Groundwater was measured in stand pipes installed within these boreholes at depths of 5.5 metres below the existing ground surface on August 3, 2017.

=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 %	0.003	Negligible
pH	5.0 < pH	7.8	Negligible concern
Resistivity	R < 20,000 ohm-cm	7690	Moderately Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	<0.01	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 7.8, 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. Soil resistivity was found to be 7.69 ohm-m for the sample analyzed. Consideration to increasing the specified strength and/or adding air entrainment into any reinforced concrete in contact with the soil should be given. Special protection is required for reinforcement steel within the concrete walls.

GEOTECHNICAL DESIGN GUIDELINES

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.

Foundation for Proposed Light Industrial Building

The site is underlain by a deposit of sensitive silty clay. Based on the undrained shear strength measurements within the silty clay deposit, this material has a soft to firm 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 foundation and the thickness of the soils deposit beneath the footings. It is considered that the subsurface soils have insufficient capacity to support the proposed concrete foundation on conventional shallow foundations.

It is considered that the proposed light industrial building may be founded on:

1) On pile foundations such as driven piles deriving support in end bearing on bedrock in combination with perimeter grade beams and a concrete slab on grade floor.

With the exception of the topsoil, the soils encountered at the site are suitable to support a cast in place concrete floor slab. The excavations for the foundations should be taken through any surficial fill, topsoil or otherwise deleterious material to expose the native, undisturbed silty clay. The excavation for the grade beam should be extended to a sufficient depth to provide frost protection for the grade beam.

Pile Foundations

It is considered that the proposed light industrial building, could be founded on deep foundations such as driven piles deriving support in end bearing on the bedrock. The following comments are provided for this option:

- All existing topsoil should be removed throughout the building area

It is common practice in the Ottawa area to use pipeline steel for piling. We suggest that a similar approach be taken for this project and that closed ended, concrete filled, steel pipe piles be used. The following pile design example is provided:

Pile Type	Geotechnical Reaction at Serviceability Limit States (kilonewtons)	Factored Geotechnical Resistance at ULS (kilonewtons)
Pipe pile 244 mm diameter by 12 mm wall thickness	1,100	1,350

Note: The SLS and ULS loads assume that the yield strength of the steel is at least 340 MPa and that the piles are filled with concrete having a compressive strength of 30 MPa.

Pipe piles should be driven closed ended and fitted with 20 millimetre (minimum) thick end plates.

All of the piles should be driven to refusal. The refusal criteria will be highly dependent on the contractor's pile driving equipment. Typically, for the drop hammer type piling rigs available in the Ottawa area, a refusal criteria of 20 blows for the last 25 millimetres of penetration would be sufficient to achieve the above loads, assuming that about 35 kilojoules of energy are transferred to the pile per blow.

The dynamic cone penetration testing within borehole BH1 encountered practical refusal on possibly very dense glacial till containing cobbles and boulders and/or bedrock. It is possible that some of the piles may encounter refusal to driving on or within the bouldery glacial till. The use of a pile with a thick wall may allow penetration of the glacial till with less damage. Notwithstanding, some problems with misalignment, plumbness, bending and/or sweeping of the piles, and hard driving conditions could occur due to the presence of cobbles and boulders above the bedrock surface. As such, allowance should be made to drive additional piles and to enlarge some of the pile caps, etc., as required. The requirement for this, if any, would have to be evaluated at the time of construction.

The contractor should be required to submit a copy of the proposed pile type and driving criteria for review prior to the start of actual construction. An allowance should be made in the specifications for re-striking all of the piles at least once, after adjacent piles within 4.0 metres distance have been installed to confirm the permanence of the pile set and to check for upward displacement caused by driving adjacent piles. Piles that do not meet the design set criteria on the first re-strike should receive additional re-striking at 2 day intervals. Furthermore, the specifications should make provision for dynamic testing of selected piles during the early stages of the pile driving operations to verify the transferred energies and pile capacities. In accordance with Ontario Building Code requirements (refer to Clause 4.1.9.4), the pile caps should be interconnected with tie beams.

Frost Protection Requirements for Perimeter Grade Beam and Pile Caps

All exterior foundation elements and those in any unheated parts of the proposed building 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.

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.

Slab on Grade Support

For predictable performance of the proposed concrete floor slab all existing 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.

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 a minimum of 0.3 metres thickness of 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 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.

If any areas of the proposed building are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

Foundation Wall Backfill and Drainage

To prevent possible foundation frost jacking due to frost adhesion, the backfill against the foundation walls or isolated walls or piers should consist of free draining, non-frost susceptible material such as 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.

Provided everywhere the proposed finished floor surfaces are above the exterior finished grade and provided the exterior grade is adequately sloped away from the proposed building addition, no perimeter foundation drainage system is required.

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 E.

Seismic Site Response Site Class Calculation

Borehole 2							
Layer	Description	Depth (m)	d_i (m)	$N(60)_i$ (blows / 0.3m)	d_i/N_i	S_{ui} (kPa)	D_i/S_{ui} (m/kPa)
	Foundation	1.5					
1	Silty Clay	1.5	11.9			36.3	0.327
2	Glacial Till	13.4	1.5	63	0.024		
3	Bedrock	14.9	16.6	100	0.166		
	$sum(d_i/N(60)_i)$				0.190		
	$d_c/(sum(d_i/N(60)_i)$				95		
	$sum(d_i/S_{ui})$						0.327
	$d_c/(sum(d_i/S_{ui}))$						36.3

Since Su = 36.3 < 50 kPa and there is greater than 3 metres of soil with a plasticity index of PI > 20, the seismic site class is Site Class E.

National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.294 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.

Potential for Soil Liquefaction

As indicated above, the results of the boreholes indicate that the native deposits underlying the site consist of a stiff silty clay crust followed by firm to soft clays to depths of about 13.4 metres. The dynamic cone penetration test values increased with depth below 13.4 metres and ranged from 6 to greater than 50 blows per 0.3 metres. At a depth of some 14.9 metres below the existing ground surface, refusal to cone penetration was encountered and indicates either large boulders or bedrock in BH1 at about 14.9 metres. It is considered likely that the increase in blow count at about 13.4 metres depth indicates the possible presence of glacial till materials.

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.

C.F.E.M. section 6.6.3.2 (7) discusses residual strength for silts and clays, that is it recommends that the residual strength for silt and clay zones be determined as per the following guidelines given below:

a) W_c/LL ≥ 0.85 and PI ≤ 12: Sr = remolded shear strength,

b) W_c/LL ≥ 0.8 and 12 < PI < 20 Sr = 0.85 Su where Su = static undrained shear strength

c) W_c/LL < 0.80 and PI ≥ 20: Sr = Su

Since the soil samples tested had plastic indexes of greater than 20 and liquid limits of greater than 47, the silty clay at the site would be in Zone C with reference to the Seed et al. criteria and category c) with reference to C.F.E.M. section 6.6.3.2 (7). As such it is considered that the silty clay at the site is not subject to classic cyclic liquefaction and that the static undrained shear strength will remain unchanged following a seismic event.

SITE SERVICES

Excavation

The excavations for the site services will be carried out through 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 the standpipes installed in boreholes BH2 and BH3 as well as the plasticity of the soil, 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.

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 subgrade 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.

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 pavement areas, acceptable native materials should be used as backfill between the pavement subgrade level and the depth of seasonal frost penetration (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. 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 subgrade for the parking areas, 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 to 90 percent 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.

Seepage Barriers

The permanent lowering of the groundwater level at the site can be caused by drainage through the granular bedding and cover materials within the sewer trenches. Groundwater lowering can cause stress within the silty clay materials which underlie the site and in turn result in settlement of the concrete slab on grade floor. To minimize the possibility of groundwater lowering at this site due to the presence of the proposed sewers, it is considered that clay dykes should be provided within sewer trenches at about 90 metre spacing. Details for construction of the proposed clay dykes are shown in the attached Figure 3.

ACCESS ROADWAY AND PARKING AREA PAVEMENTS

Based on the results of the boreholes, the subsurface conditions in the access roadway and parking areas consist of topsoil overlying grey brown silty clay. For predictable performance of the pavement structures, it is considered that all of the topsoil will have to be removed in preparation for pavement construction at this site.

Once existing topsoil and all deleterious material has been removed, the exposed sub-grade should be inspected and approved by geotechnical personnel and any soft areas evident should be sub-excavated and replaced with suitable earth borrow or granular crushed stone approved by the geotechnical engineer. The sub-grade should be shaped and crowned to promote drainage of the roadway area granular. Following approval of the preparation of the sub-grade, the pavement granulars may be placed.

For any areas of the site that require the sub-grade to be raised to proposed pavement sub-grade level, the material used should consist of OPSS select sub-grade material or OPSS Granular B Type I or Type II. Recycled crushed concrete meeting the grading specifications for Granular B Type II could also be used. Materials used for raising the sub-grade to proposed roadway area sub-grade 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.

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

For pavement areas subject to heavy truck loading the pavement should consist of:

40 millimetres of hot mix asphaltic concrete (HL3) over
40 millimetres of hot mix asphaltic concrete (HL8) over
150 millimetres of OPSS Granular A base over
350 millimetres of OPSS Granular B, Type II subbase
(50 or 100 millimetre minus crushed stone)
Non-woven geotextile fabric (4 oz/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. The adequacy of the design of the pavement thickness should be assessed by the geotechnical personnel at the time of construction.

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, 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 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 site services, access roadway and parking areas 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
Aug 9.2017
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

Attachments: Record of Boreholes
Figures 1 and 2
Laboratory Test Results for Chemical Properties
Laboratory Test Results for Physical Properties – Stantec Laboratory Test Results for Soils

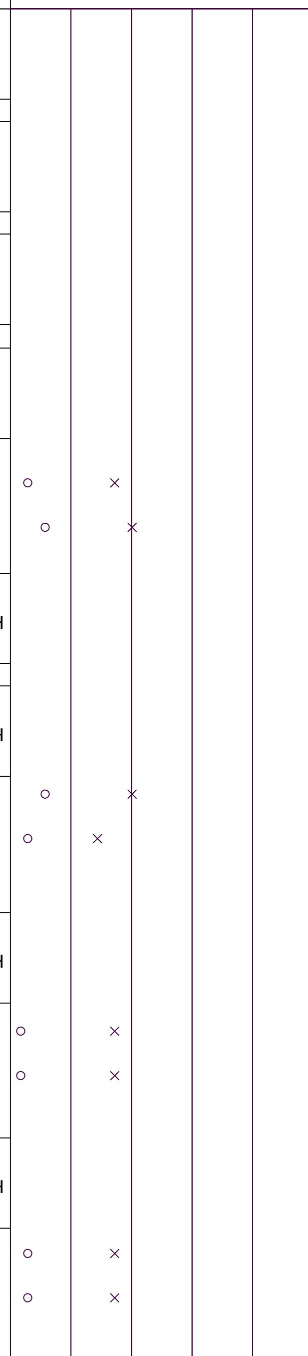
RECORD OF BOREHOLE BH1

PROJECT: Proposed Light Industrial Development
CLIENT: 2465070 Ontario Ltd.
LOCATION: 2375 St. Laurent Boulevard, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 170549
DATE OF BORING: July 27, 2017
SHEET 1 of 2
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m
0	Ground Surface				
	TOPSOIL	0.00			
	Stiff grey brown to grey SILTY CLAY		1	SS	2
1			2	SS	13
2			3	SS	8
			4	SS	2
3					
4			5	SS	WH
5			6	SS	WH
6			7	SS	WH
7					
8			8	SS	WH
9					

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

Water observed in borehole at approximately 3.8 metres below the existing ground surface on July 27, 2017.

DEPTH SCALE: 1 to 50
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH1

PROJECT: Proposed Light Industrial Development
CLIENT: 2465070 Ontario Ltd.
LOCATION: 2375 St. Laurent Boulevard, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 170549
DATE OF BORING: July 27, 2017
SHEET 2 of 2
DATUM:

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	PIEZOMETER OR STANDPIPE INSTALLATION
				9	SS	WH				
10	Borehole continued as Probe Hole through silty clays, sand, silts and glacial till		9.75							
11										
12										
13	Assuming changing to glacial till based on increased blow counts		13.41							
14										
15	End of Borehole, refusal on large boulders or bedrock		14.88							
16										
17										
18										

DEPTH SCALE: 1 to 50
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2

PROJECT: Proposed Light Industrial Development
CLIENT: 2465070 Ontario Ltd.
LOCATION: 2375 St. Laurent Boulevard, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 170549
DATE OF BORING: July 27, 2017
SHEET 1 of 1
DATUM:

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									
0	TOPSOIL		0.00							
	Stiff grey brown to grey SILTY CLAY			1	SS	2				
1				2	SS	11				
2				3	SS	8				
				4	SS	2				
3				5	SS	1				
4				6	SS	WH				
6				7	SS	WH				
8				8	SS	WH				
9				9	SS	WH				
	End of Borehole		9.75							
10										
11										
12										
13										

Water observed in borehole at approximately 7.0 metres below the existing ground surface on July 27, 2017. Water level measured in standpipe at about 5.5 metres below existing ground surface, **August 3, 2017.**

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

RECORD OF BOREHOLE BH3

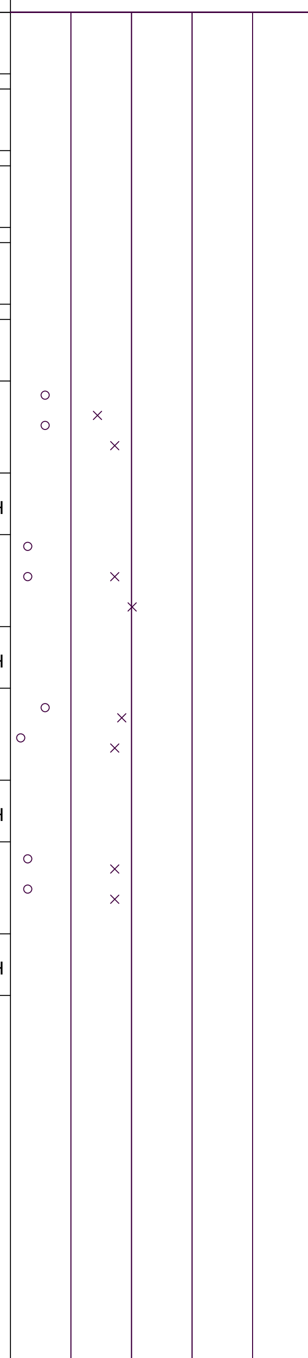
PROJECT: Proposed Light Industrial Development
CLIENT: 2465070 Ontario Ltd.
LOCATION: 2375 St. Laurent Boulevard, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 170549
DATE OF BORING: July 27, 2017
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)
	Ground Surface	
0	TOPSOIL	0.00
	Stiff grey brown to grey SILTY CLAY	
9.75	End of Borehole	9.75

SAMPLES: NUMBER	TYPE	BLOWS/0.3m
1	SS	2
2	SS	9
3	SS	8
4	SS	3
5	SS	1
6	SS	WH
7	SS	WH
8	SS	WH
9	SS	WH

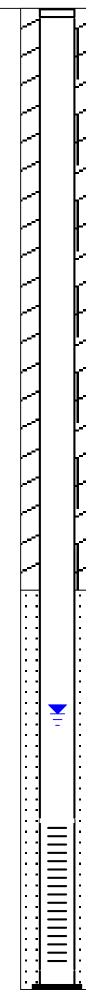
UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80
REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80



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

ADDITIONAL LAB TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



Water observed in borehole at approximately 7.0 metres below the existing ground surface on July 27, 2017. Water level measured in standpipe at about 5.5 metres below existing ground surface, **August 3, 2017.**

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

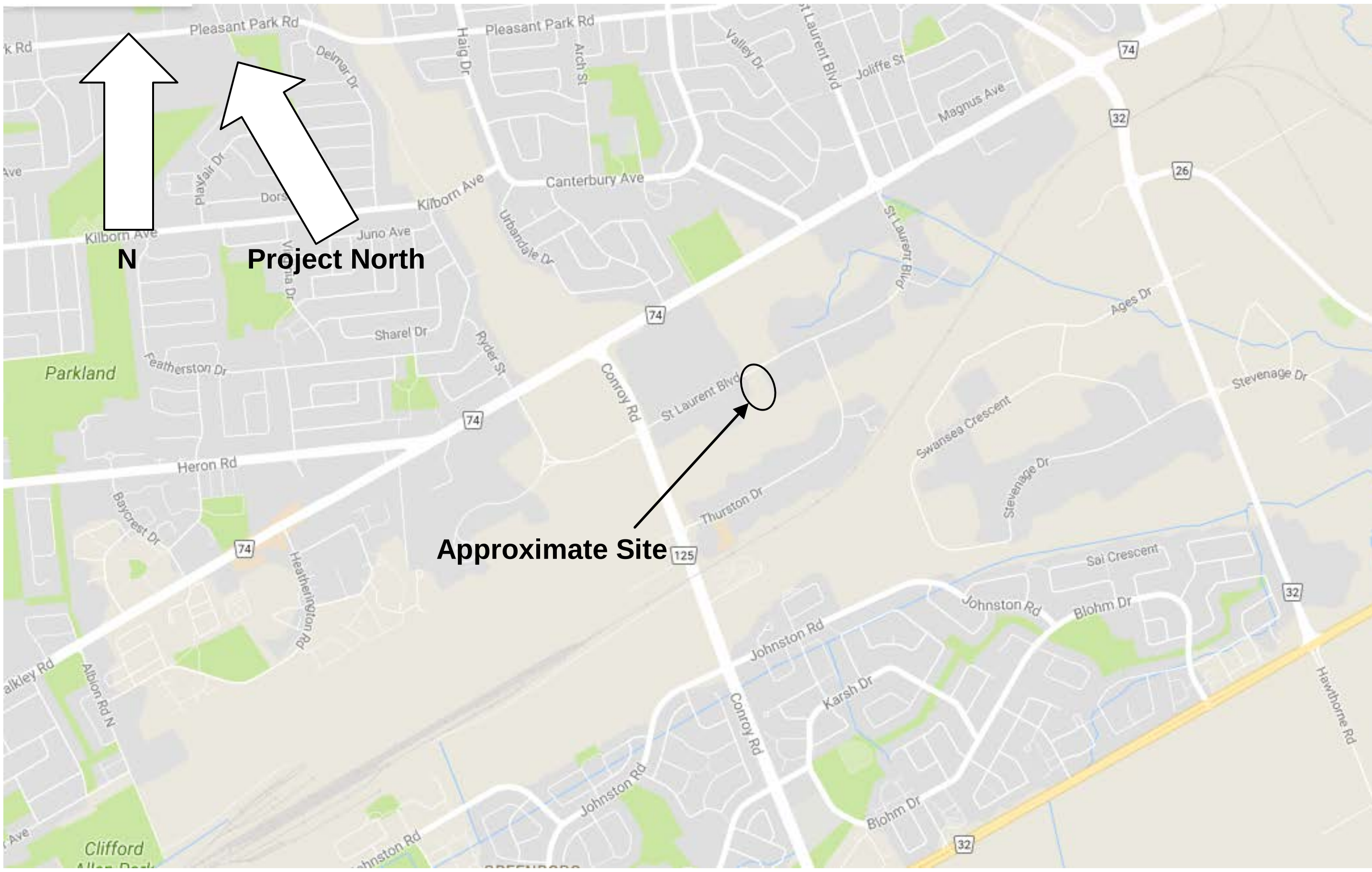
Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

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

c_u undrained shear strength
e void ratio
C_c compression index
C_v coefficient of consolidation
k coefficient of permeability
I_p plasticity index
n porosity
u pore pressure
w moisture content
w_L liquid limit
w_p plastic limit
$\$^1$ effective angle of friction
r unit weight of soil
y^1 unit weight of submerged soil
cr normal stress

KEY PLAN	FIGURE 1

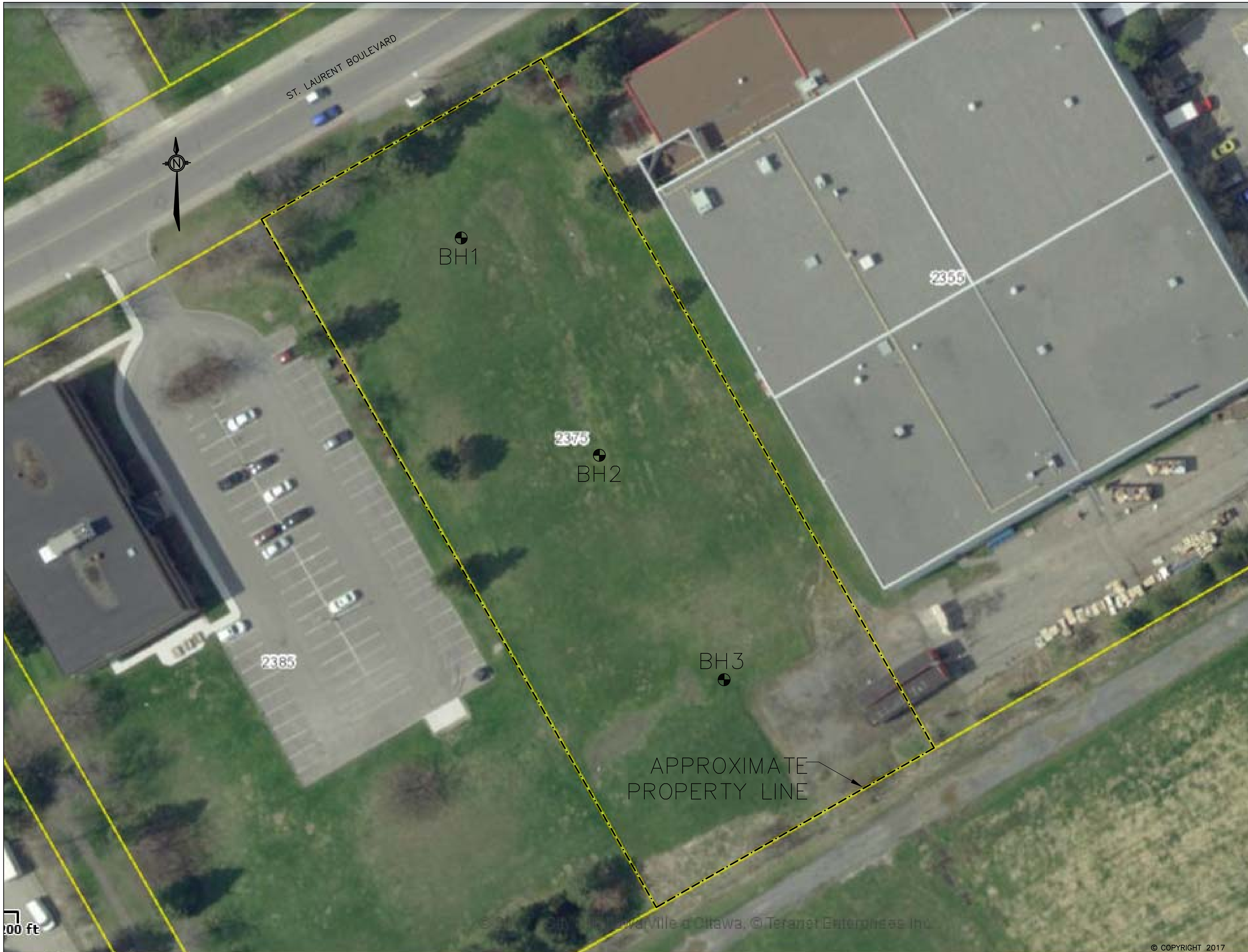


NOT TO SCALE

Kollaard Associates
Engineers

Project No. 170549

Date July 2017



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:
YONAKA MOTORSPORTS OTTAWA

PROJECT:
GEOTECHNICAL INVESTIGATION FOR LIGHT INDUSTRIAL BUILDING

LOCATION:
2375 ST. LAURENT BOULEVARD
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
--	JULY 11, 2017
DRAWN BY: DT	SCALE: N.T.S

KOLLAARD FILE NUMBER:
170549

Laboratory Test Results for Chemical Properties

Certificate of Analysis

Eurofins Environment Testing

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1714364
		Date Submitted:	2017-08-01
		Date Reported:	2017-08-09
		Project:	1705219
		COC #:	192247
Attention:	Mr. Dean Tataryn		
PO#:	1705249		
Invoice to:	Kollaard Associates Inc.		

Dear Dean Tataryn:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Addrine Thomas
Team Leader, Inorganics

All analysis is completed in Ottawa, Ontario (unless otherwise indicated).

Eurofins Ottawa is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on our CALA scope of accreditation. It can be found at http://www.cala.ca/scopes/2602.pdf.

Eurofins(Ottawa) is certified and accredited for specific parameters by OMAFRA, Ontario Ministry of Agriculture, Food and Rural Affairs (for farm soils). Licensed by Ontario MOE for specific tests in drinking water.

Eurofins(Mississauga) is accredited for specific parameters by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required.

Certificate of Analysis

eurofins | Environment Testing

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1714364
Attention:	Mr. Dean Tataryn	Date Submitted:	2017-08-01
PO#:	1705249	Date Reported:	2017-08-09
Invoice to:	Kollaard Associates Inc.	Project:	1705219
		COC #:	192247

Group	Analyte	MRL	Units	Guideline	Lab I.D. 1309611 Sample Matrix Soil Sample Type Sampling Date 2017-07-27 Sample I.D. BH 1 SS3 5-7
Agri. - Soil	pH	2.0			7.8
	SO4	0.01	%		<0.01
General Chemistry	Cl	0.002	%		0.003
	Electrical Conductivity	0.05	mS/cm		0.13
	Resistivity	1	ohm-cm		7690

Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1714364
Attention:	Mr. Dean Tataryn	Date Submitted:	2017-08-01
PO#:	1705249	Date Reported:	2017-08-09
Invoice to:	Kollaard Associates Inc.	Project:	1705219
		COC #:	192247

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 331002 **Analysis/Extraction Date** 2017-08-02 **Analyst** C F			
Method Ag Soil			
pH	<2.0	100	90-110
Method Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	85-115
Method Resistivity - soil			
Resistivity			
Run No 331080 **Analysis/Extraction Date** 2017-08-02 **Analyst** C F			
Method AG SOIL			
SO4	<0.01 %		70-130
Run No 331322 **Analysis/Extraction Date** 2017-08-09 **Analyst** AET			
Method C CSA A23.2-4B			
Chloride		99	90-110

Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

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Laboratory Test Results for Physical Properties

Stantec

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

August 4, 2017
File: 122410003

Attention: **Kollaard Associates Engineers, File #170549**

Reference: **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-1 SS-6	15'-17'	74.5%	68.3	25.4	43.0
BH-2 SS-3	5'-7'	45.4%	74.2	28.9	45.3

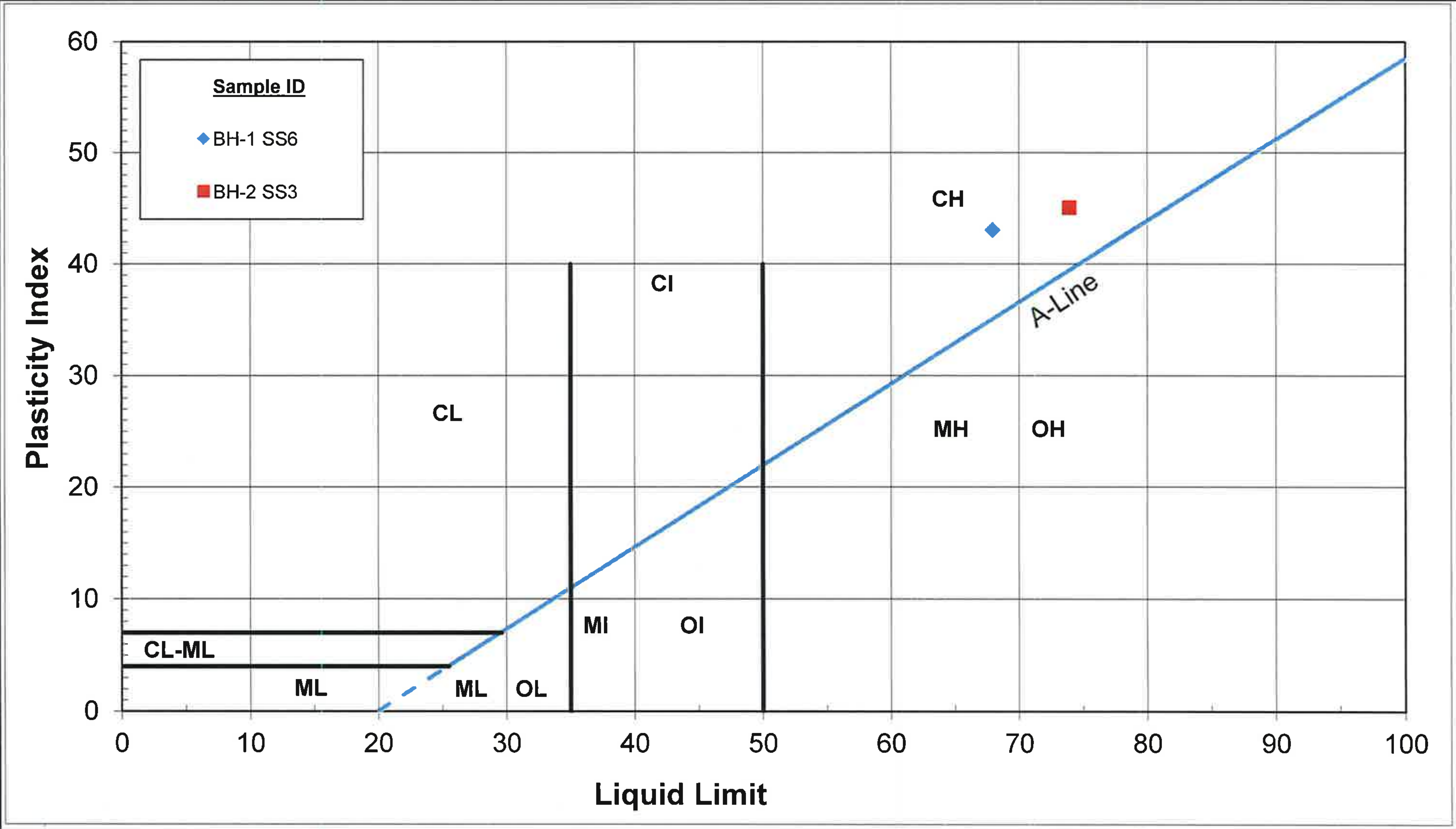
Sincerely,

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



Kollaard Associates Engineers

2375 St. Laurent Blvd, File #170549

PLASTICITY CHART

Figure No.

Project No. 122410003

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

August 15, 2017

Site: 45.3825 N, 75.6229 W   User File Reference: 2375 St. Laurent Boulevard

Requested by: Dean Tataryn, Kollaard Associates Inc.

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

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.472	0.550	**0.459**	0.348	**0.246**	**0.121**	**0.057**	**0.015**	**0.0055**	**0.294**	**0.203**

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

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.045	0.155	0.260
Sa(0.1)	0.062	0.194	0.314
Sa(0.2)	0.056	0.167	0.265
Sa(0.3)	0.044	0.128	0.202
Sa(0.5)	0.031	0.090	0.143
Sa(1.0)	0.015	0.045	0.071
Sa(2.0)	0.0061	0.021	0.033
Sa(5.0)	0.0013	0.0048	0.0082
Sa(10.0)	0.0006	0.0019	0.0032
PGA	0.033	0.105	0.170
PGV	0.022	0.070	0.114

References

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

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

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

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

Aussi disponible en français