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Civil • Geotechnical •
Structural • Environmental •
Hydrogeology

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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED HEAVY MACHINERY MOVERS WAREHOUSING FACILITY
6622 BANK STREET
CITY OF OTTAWA, ONTARIO

Project # 170035

Submitted to:

CAMM Warehousing and Rentals Inc.
3460 Rideau Road
Gloucester, Ontario
K1G 3N4

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June 26, 2017

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CAMM Warehousing and Rentals Inc.
3460 Rideau Road
Gloucester, Ontario
K1G 3N4

RE: GEOTECHNICAL INVESTIGATION
PROPOSED HEAVY MACHINERY MOVERS WAREHOUSING FACILITY
6622 BANK STREET
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed warehousing facility. The purpose of the investigation was to identify the subsurface conditions at the site based on a limited number of boreholes and test pits. 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 single storey, metal clad, warehouse type building some 60.9 metres deep by 38.1 metres wide (200' x 125') at 6622 Bank Street in the City of Ottawa, Ontario (see Key Plan, Figure 1). The site has a total of about 7.6 hectares (18.8 acres) and is currently vacant. The site has about 233.2 metres of frontage onto Bank Street.

Preliminary plans indicate that the proposed building will be of steel frame construction with conventional spread footing foundations and a concrete slab on grade floor.

The site is located on the west side of Bank Street and is locally described as 6622 Bank Street, Ottawa, Ontario (PIN 043151369). The site is bordered on the north by a rural dwelling and on the south by other commercial/light industrial development.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by deposits of sand and/or glacial till. Bedrock geology maps indicate that the bedrock underlying the site consists of dolomite and limestone of the Ottawa Formation, and possibly sandstone underlying limestone at depth.

PROCEDURE

The field work for this investigation consisted of boreholes and test pits. Two boreholes, numbered BH1 and BH2, were put down at the site on April 20, 2017 using a track mounted drill rig equipped with a hollow stem auger owned and operated by Marathon Drilling of Greely, Ontario. Three test pits, numbered TP1 to TP3 inclusive, were put down at the site on June 8, 2017 using a rubber tire mounted backhoe supplied by the owner of the site.

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 3.4 to 4.1 metres below the existing ground surface (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils). In situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil) was not carried out as no cohesive materials were encountered at any of the boreholes.

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.

The test pits were advanced, through the fill which had been placed within the proposed parking area, to depths ranging from about 1.2 to 1.6 metres below the existing ground surface. The soil conditions observed in the test pits were classified based on visual and tactile examination of the materials in the walls and bottom of the test pits (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) and an assessment of the difficulty of excavation. No in situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil) was carried out as no cohesive materials were encountered within the test pits. The soils were classified using the Unified Soil Classification System. The groundwater conditions were observed in the open test pits at the time of the field work.

The test pits were loosely backfilled upon completion of excavation.

One soil sample from BH1 was submitted for hydrometer testing (ASTM D422). 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 and test pits in the field, logged the boreholes and test pits and cared for the samples obtained. A description of the subsurface conditions encountered at the boreholes and test pits are given in the attached Record of Borehole and Table I, Record of Test Pits 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 and test pits 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 and test pits is provided in the attached Record of Borehole and Record of Test Pit Sheets following the text of this report. The borehole and test pit 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 and test pit locations may vary from the conditions encountered at the boreholes and test pits.

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 and test pit 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 borehole and test pit locations were determined, in the field, relative to a geodetic benchmark provided by Kollaard Associates. The geodetic benchmark is described as a nail in a hydro pole located on the west side of Bank Street, approximately 140 south of the entrance way to the site. The elevation of the of the nail is referenced as 94.53 metres.

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

Granular Fill

Fill materials were encountered from the ground surface at both boreholes in the proposed building area. The fill materials consisted of grey brown silty sand, some gravel, cobbles and boulders, with a trace to some concrete and wood debris. The results of standard penetration testing carried out in the fill materials, ranged from 11 to 35 blows per 0.3 metres with an average value of 23 blows per 0.3 metres, indicating a compact to dense state of packing. The fill materials had a thickness of about 1.2 to 1.4 metres. The fill materials were fully penetrated at the borehole locations.

Fill materials were also encountered in the test pits put down within the proposed parking lot areas. The fill materials consisted of a mixture of crushed concrete, sand, gravel, brick, metal and asphalt debris. Based on the difficulty of digging of the test pits, the fill materials were observed to be in a compact state of packing.

Topsoil

A layer of topsoil with a thickness of about 0.3 metres was encountered beneath the fill materials at TP3. The material was classified as topsoil based on colour only and based on visual and tactile examination. 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.

Glacial Till

Glacial till was encountered beneath the fill materials at both of the boreholes and all of the test pit locations. The glacial till consisted of gravel and cobbles, in a matrix of grey brown sand, with a trace of clay. It is noted that large cobbles and boulders were also encountered within the glacial till. The results of standard penetration testing carried out in the glacial till material, range from 11 to 83 blows per 0.3 metres with an average value of 36 blows per 0.3 metres, indicating a dense state of packing.

One soil sample of glacial till (BH1 - SS4 - 2.3 to 2.89 metres) was submitted to Stantec for hydrometer testing (ASTM D422). The results of the hydrometer testing indicated that the samples consists of about 9 percent clay, about 31 percent silt, 42 percent fine to coarse sand and about 18 percent gravel. Both samples are indicated to have between about 42 percent silt and clay size particles. The results are located in Attachment A.

Both of the boreholes encountered refusal to further advancement of the standard penetration split spoon on either large boulders or bedrock at depths of 4.1 metres and 2.1 metres below the existing ground surface for boreholes BH1 and BH2, respectively.

The test pits were terminated within the glacial till at depths ranging between 1.2 to 1.6 metres below the existing ground surface. Based on the difficulty of digging, the glacial till was observed to be in a compact to dense state of packing.

Bedrock

Both boreholes were terminated on the surface of bedrock or large boulders with practical refusal at depths ranging from approximately 4.1 and 2.1 metres, respectively, below the existing ground surface level.

The boreholes were advanced into the bedrock by coring to verify the existence of bedrock or boulders and assess depth to sound bedrock. In general, refusal to further advancement with depth was encountered between about 0.6 to 0.9 metres below the surface of relatively sound bedrock.

A review of the bedrock information contained in a report entitled *Final Geotechnical Investigation Report – Kenny U-Pull Development – 6650, 6638 & 6622 Bank Street, Ottawa, (Greely) Ontario DST Reference No: IN-SO-026670, dated January 2017* by DST Consulting Engineers Inc. indicated the following:

Siltstone was encountered in two of the boreholes put down at the site. Rock Quality Designation (RQD) was found to be between 60% and 88% at one borehole indicating fair to good rock and 47% and 96% and indicating poor to excellent rock.

Groundwater

No groundwater seepage was observed within boreholes at the time of drilling. 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 > 400 ug/g	13	Negligible
pH	5.0 < pH	8.0	Neutral/Slightly Basic Negligible concern
Resistivity	R < 1500 ohm-cm	4760	Slightly 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 8.0, 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 4.7 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. Since the chloride and sulphate concentrations are low, the soil resistivity indicates only a slightly corrosive environment and proper concrete cover is sufficient to protect the reinforcement in buried concrete.

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.

Foundations for Proposed Light Industrial Building

With the exception of any surficial fill, the subsurface conditions encountered at the boreholes advanced during the investigation are suitable for the support of the proposed warehouse facility on conventional spread footing foundations bearing on an engineering granular pad placed on a native subgrade. The excavations for the foundation should be taken through any fill materials or otherwise deleterious material to expose the native, undisturbed glacial till.

For predictable performance of the proposed foundations, all existing fill materials (+/-1.2 to 1.4 metres) and any deleterious materials should be removed from within the proposed foundation areas and should be replaced to the proposed founding level using suitable engineered fill. It is expected that the subgrade, beneath the fill materials, consists of native undisturbed glacial till.

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

Compaction should be verified by a suitable field compaction test method. To allow the spread of load beneath the foundations, the engineered fill should extend out from the outside edges of the thickened edge slab 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 excavations within the glacial till above any groundwater level should not present any serious constraints.

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 dwelling foundation.

Strip and pad footings, a minimum 0.5 metres in width bearing on the native undisturbed glacial till at a founding depth of a minimum of 1.5 metres below the original ground surface and above the groundwater level or on an engineered pad may be designed using a maximum allowable bearing pressure of 150 kilopascals for serviceability limit states and 250 kilopascals for the factored ultimate bearing resistance.

The above allowable bearing pressure is subject to a maximum grade raise of 3.0 metres above the original ground surface and to maximum strip and pad footing widths of 2.0 metres.

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

The native glacial till 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

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.

The depth of frost cover could be reduced for footings bearing on engineered fill over glacial till. 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.

Slab on Grade Support

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 of the walls 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.

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. Landscaped areas of areas that are not settlement sensitive may be compacted to 90% 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, no perimeter foundation drainage system is required.

National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.312g 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.

Both boreholes encountered relatively shallow refusal on possible bedrock at depths of about 2.1 and 4.1 metres. Bedrock was confirmed in BH1. Based on the geotechnical profile encountered in the boreholes, Table 4.1.8.4.A, National Building Code 2015) the site classification for seismic site response is Site Class C.

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 at which refusal to further advancement using standard penetration testing was attained. The CRR value was determined from a mathematical expression as determined by Rauch (1997) of the base curve obtained from Robertson and Fear (1996). The CSR was determined from Seed and Idriss (1971). It is considered that a soil with a normalized SPT of greater than 30 is non-liquefiable. It is also considered that a soil with a CRR/CSR ratio of greater than one is not liquefiable.

The average SPT values for the glacial till encountered at the site was above 30 and the glacial till was of limited thickness. As such, the underlying soils below the proposed foundation are not considered to be liquefiable.

Parking Lot Pavements

Subgrade Preparation

Based on the information obtained during the excavation and drilling of the test holes, the subsurface conditions across the majority of the proposed access roadway and parking areas consist of imported fill materials consisting of crushed concrete with some sand, gravel, brick and metal and asphalt debris overlying native undisturbed glacial till. The fill materials were observed to have been compacted during placement with the use of a large vibratory roller. The lift thickness and frequency of compaction is unknown. Topsoil was encountered at one test hole location. It is understood that the topsoil materials had been stripped from the majority of the site and pushed to the sides of the site.

For predictable performance of the pavement structures, it is considered that additional test pits could be put down along the sides of the proposed parking area to further delineate the extent of the topsoil below the fill material.

It is considered that the imported fill materials of crushed concrete with some sand, gravel, brick and metal and asphalt debris that is free of topsoil or organic debris may be left in place to form the subgrade and base structure of the proposed roadway and parking areas. The existing fill materials should be re-compacted with a large (greater than 15 tonnes) steel drum vibratory roller.

It is considered that in some locations, the fill materials contain relatively large pieces of concrete. Large pieces of concrete when placed together in the sub-grade of an access roadway or parking area may result in voids beneath the roadway or parking area structure. Finer granular materials may migrate down into these voids over time resulting in a minor "sink hole" on the surface. Since the voids area of limited size and extent, the sink hole will be small and will not propagate or cause a loss of structural support for vehicles in the area. Should a "sink hole" form it could be repaired using OPSS Granular A material.

Prior to placement of any base materials, the existing fill materials should be proof rolled with the large roller and with a fully loaded tri-axel dump truck. Any areas where movement or deflection of the existing fill is observed should be sub-excavated. The fill material at these locations should be re-compacted in lifts or replaced depending on the consistency of the fill material.

Any areas of the site that require the sub-grade to be raised to the proposed roadway area sub-base level should be filled with granular materials consisting of OPSS select sub-grade material or OPSS Granular B Type 1 or Type 2. Alternatively crushed concrete and gravel fill meeting the grading requirements for one of the above specified materials may be used. Materials used for raising the sub-grade to proposed roadway area base 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.

The pavement structure for the site has been divided into three areas consisting of the granular surfaced yard area, the granular surfaced access roadway and parking area, and the proposed asphaltic surfaced areas. The granular surfaced yard area will have limited traffic and will be primarily used for the temporary parking of trailers (both loaded and unloaded) and equipment used in the heavy moving industry. The granular surfaced access roadway and parking area will be used to access the building and parking of frequently used equipment. The asphaltic surfaced areas will consist of the entrance roadway and customer parking areas.

Granular Surfaced Yard Area:

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

50 millimetres of OPSS Granular A base, over

1.0 to 1.5 metres of imported granular and crushed concrete fill materials

Granular Surfaced Access Roadway and Parking Area

200 millimetres of OPSS Granular A base over

6 oz/sq-yd Non-woven geotextile cloth such as US Fabrics US 160NW, Layfield LP6 or Terrafix 360R or approved alternative, over

Existing imported granular and crushed concrete fill materials

Asphaltic Concrete Surfaced Areas:

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

50 millimetres of hot mix asphaltic concrete, Superpave 12.5 (HL3) over

150 millimetres of OPSS Granular A base over

OPSS Granular B, Type II subbase (50 or 100 millimetre minus crushed stone)
(if required to raise subgrade to base) over

6 oz/sq-yd Non-woven geotextile cloth such as US Fabrics US 160NW, Layfield LP6 or Terrafix 360R or approved alternative, over

Existing imported granular and crushed concrete fill materials

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

40 millimetres of hot mix asphaltic concrete, Superpave 12.5 (HL3) over

50 millimetres of hot mix asphaltic concrete, Superpave 19.0 (HL8) over

150 millimetres of OPSS Granular A base over

OPSS Granular B, Type II subbase (50 or 100 millimetre minus crushed stone)
(if required to raise subgrade to base) over

6 oz/sq-yd Non-woven geotextile cloth such as US Fabrics US 160NW, Layfield LP6 or Terrafix 360R or approved alternative, over

Existing imported granular and crushed concrete fill materials

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 any roadway fill has been adequately compacted. If the roadway 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 pavement granular materials to ensure the materials meet the specifications from a compaction point of view.

The native silt and clay deposits within glacial till 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
June 26,2017
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

Attachments: Record of Boreholes and Test Pits
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 (WEST)

PROJECT: Proposed Light Industrial Building
CLIENT: CAMM Warehousing Rentals Inc.
LOCATION: 6622 Bank Street, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 170035
DATE OF BORING: April 20, 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	Grey brown silty sand, some gravel, cobbles and boulders, trace to some concrete and wood debris (FILL)		0.00	1	SS	11				
1				2	SS	28				
	Yellow brown to grey brown silty sand, some gravel, cobbles and boulders, trace of clay (GLACIAL TILL)		1.37	3	SS	44				Borehole dry, April 20, 2017.
2				4	SS	83				
3					RC					
				5	SS	50				
4	Refusal on BEDROCK (Advanced by coring)		4.06		RC					
	End of borehole in BEDROCK		4.77							
5										
6										
7										
8										
9										

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2 (EAST)

PROJECT: Proposed Light Industrial Building
CLIENT: CAMM Warehousing Rentals Inc.
LOCATION: 6622 Bank Street, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 170035
DATE OF BORING: April 20, 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	Grey brown silty sand, some gravel, cobbles and boulders, trace to some concrete and wood debris (FILL)		0.00	1	SS	19				
1				2	SS	35				Borehole dry, April 20, 2017.
	Yellow brown to grey brown silty sand, some gravel, cobbles and boulders, trace of clay (GLACIAL TILL)		1.22	3	SS	24				
2	Practical refusal on large boulder or bedrock (Advanced by coring)		2.13		RC					
3	End of Borehole in BEDROCK		3.35							
4										
5										
6										

DEPTH SCALE: 1 to 35
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

TABLE I

RECORD OF TEST PITS
GEOTECHNICAL INVESTIGATION
PROPOSED HEAVY MACHINERY MOVERS WAREHOUSING FACILITY
6622 BANK STREET
CITY OF OTTAWA, ONTARIO

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP1(west)	0.00 – 1.10	Compact crushed concrete, brick, rebar and asphalt (FILL)
	1.10 – 1.60	Grey brown silty sand, some gravel, cobbles and larger boulders, trace clay (GLACIAL TILL)
	1.60	End of test pit

Test pit dry, June 8, 2017.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP2(southwest)	0.00 – 0.80	Compact crushed concrete, brick, rebar and asphalt (FILL)
	0.80 – 1.20	Grey brown silty sand, some gravel, cobbles and larger boulders, trace clay (GLACIAL TILL)
	1.20	End of test pit

Test pit dry, June 8, 2017.

TABLE I(continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP3(south)	0.00 – 0.80	Compact crushed concrete, brick, rebar and asphalt (FILL)
	0.80 – 1.10	TOPSOIL
	1.10 - 1.15	Grey brown silty sand, some gravel, cobbles and larger boulders, trace clay (GLACIAL TILL)
	1.15	End of test pit

Test pit dry, June 8, 2017.

KEY PLAN

FIGURE 1

NOT TO SCALE

Kollaard Associates
Engineers

Project No. 170035

Date June 2017

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

TP1 APPROXIMATE TEST PIT 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:
CAMM WAREHOUSING AND RENTALS INC.

PROJECT:
GEOTECHNICAL INVESTIGATION FOR LIGHT INDUSTRIAL BUILDING

LOCATION:
6622 BANK STREET
CITY OF OTTAWA, ONTARIO

DESIGNED BY: --	DATE: MAY 12, 2017
DRAWN BY: DT	SCALE: N.T.S

KOLLAARD FILE NUMBER:
170035

Laboratory Test Results for Sulphate, Resistivity and pH

Certificate of Analysis

Eurofins Environment Testing

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1706862
		Date Submitted:	2017-05-05
		Date Reported:	2017-05-12
		Project:	170035
		COC #:	190406
Attention:	Mr. Dean Tataryn		
PO#:	170035		
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:	1706862
Attention:	Mr. Dean Tataryn	Date Submitted:	2017-05-05
PO#:	170035	Date Reported:	2017-05-12
Invoice to:	Kollaard Associates Inc.	Project:	170035
		COC #:	190406

Group	Analyte	MRL	Units	Guideline	
				Lab I.D.	1291532
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2017-04-20
				Sample I.D.	BH1 SS3
Agri. - Soil	pH	2.0			8.0
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.21
	Resistivity	1	ohm-cm		4760
	SO4	0.01	%		<0.01
Subcontract	Cl	5	ug/g		13

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

eurofins | Environment Testing

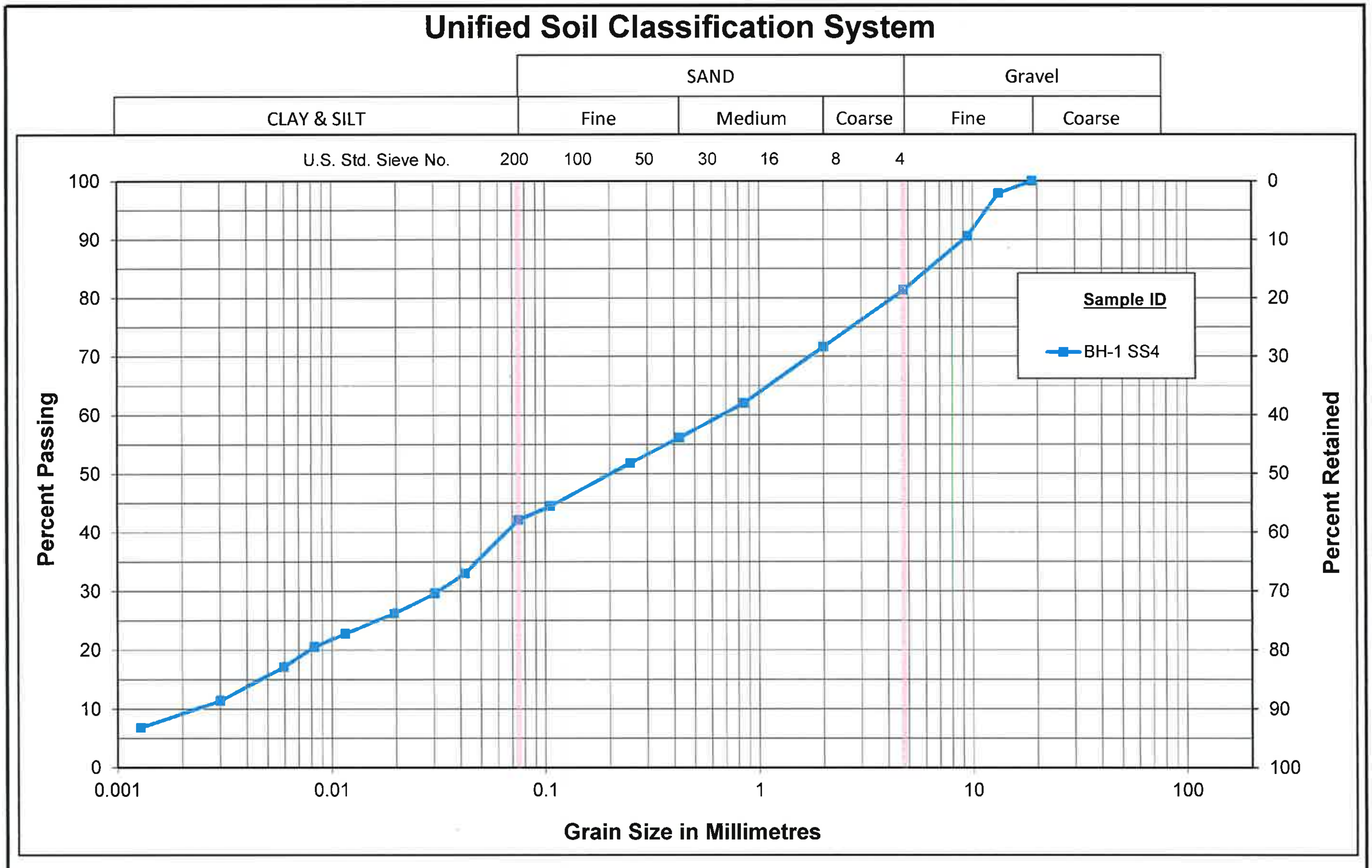
Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0
Attention:	Mr. Dean Tataryn
PO#:	170035
Invoice to:	Kollaard Associates Inc.

Report Number:	1706862
Date Submitted:	2017-05-05
Date Reported:	2017-05-12
Project:	170035
COC #:	190406

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 325967 **Analysis/Extraction Date** 2017-05-06 **Analyst** C F			
Method C SM4500-SO4--D			
SO4	<0.01 %	90	70-130
Run No 325987 **Analysis/Extraction Date** 2017-05-09 **Analyst** AJS			
Method Ag Soil			
pH			90-110
Method Cond-Soil			
Electrical Conductivity			85-115
Method Resistivity - soil			
Resistivity			
Run No 326289 **Analysis/Extraction Date** 2017-05-10 **Analyst** SDC			
Method SUBCONTRACT P			
Chloride	<5.0 ug/g	100	

Laboratory Test Results for Physical Properties



GRAIN SIZE DISTRIBUTION

Kollaard Associates Engineers, File #170035

6622 Bank Street, Ottawa

Figure No.

Project No. 122410003

Stantec

2781 Lancaster Road, Suite 101
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #170035	Project No.:	122410003
Project:	6622 Bank Street, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH-1	Date Sampled:	April 20, 2017
Sample No.:	SS4	Tested By:	Daniel Boateng
Sample Depth:	7'6"-9'6"	Date Tested:	May 9, 2017

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	61.54
Sample Weight after Hydrometer and Wash (g)	25.46
Percent Passing No. 200 Sieve (%)	58.6
Percent Passing Corrected (%)	41.99

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	838.40
Sample Weight After Sieve (g)	834.30
Percent Loss in Sieve (%)	0.49

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)	55.57
Air Dried Mass (W_a), (g)	55.73
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9971
Air Dried Mass in Analysis (M_a), (g)	61.72
Oven Dried Mass in Analysis (M_o), (g)	61.54
Percent Passing 2.0 mm Sieve (P_{10}), (%)	71.62
Sample Represented (W), (g)	85.92

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	0.0	100.0
13.2	18.1	97.8
9.5	78.7	90.6
4.75	156.5	81.3
2.00	237.9	71.6
Total (C + F)[1]	834.30	
0.850	8.22	62.06
0.425	13.31	56.13
0.250	17.01	51.83
0.106	23.34	44.46
0.075	25.37	42.10
PAN	25.42	

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)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 6:27 AM

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
09-May-17	6:28 AM	1	35.0	6.0	23.0	29.0	33.02	10.78691	9.3925	0.0128	0.0421
09-May-17	6:29 AM	2	32.0	6.0	23.0	26.0	29.60	11.25191	9.3925	0.0128	0.0304
09-May-17	6:32 AM	5	29.0	6.0	23.0	23.0	26.19	11.71691	9.3925	0.0128	0.0196
09-May-17	6:42 AM	15	26.0	6.0	23.0	20.0	22.77	12.18191	9.3925	0.0128	0.0116
09-May-17	6:57 AM	30	24.0	6.0	23.0	18.0	20.50	12.49191	9.3925	0.0128	0.0083
09-May-17	7:27 AM	60	21.0	6.0	23.0	15.0	17.08	12.95691	9.3925	0.0128	0.0060
09-May-17	10:37 AM	250	16.0	6.0	23.0	10.0	11.39	13.73191	9.3925	0.0128	0.0030
10-May-17	6:27 AM	1440	12.0	6.0	23.0	6.0	6.83	14.35191	9.3925	0.0128	0.0013

Remarks:

Reviewed By: Brian Prevost
Date: May/11/2017

V:\01216\active\laboratory_standing_offers\2017 Laboratory Standing Offers\122410003 Kollaard Associates Engineers\May 5, Hydrometer, Kollaard #170035\Hydrometer.xlsx

ATTACHMENT A

SEISMIC HAZARD CALCULATIONS

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

May 01, 2017

Site: 45.2566 N, 75.5493 W   User File Reference: Greely Hall Estates

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)**
0.508	0.588	**0.487**	0.367	**0.257**	**0.125**	**0.058**	**0.015**	**0.0055**	**0.312**	**0.212**

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)	0.044	0.160	0.274
Sa(0.1)	0.061	0.199	0.329
Sa(0.2)	0.055	0.170	0.277
Sa(0.3)	0.044	0.131	0.210
Sa(0.5)	0.031	0.092	0.147
Sa(1.0)	0.015	0.046	0.072
Sa(2.0)	0.0061	0.021	0.034
Sa(5.0)	0.0012	0.0048	0.0083
Sa(10.0)	0.0006	0.0019	0.0032
PGA	0.033	0.108	0.178
PGV	0.021	0.071	0.117

References

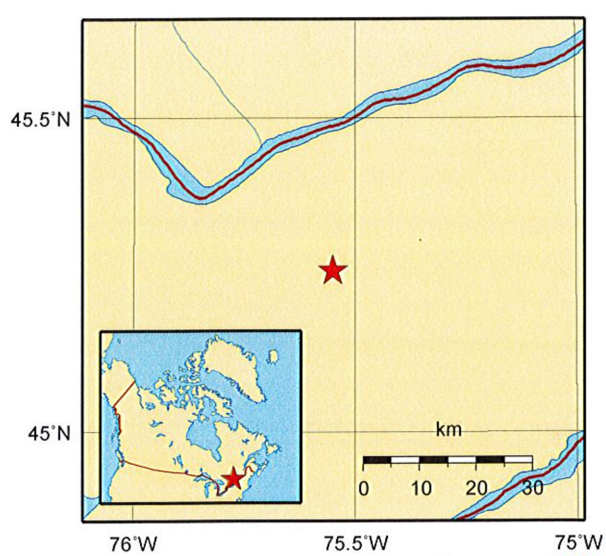
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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

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

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

Aussi disponible en français



Natural Resources Canada   Ressources naturelles Canada

Canada