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Civil • Geotechnical •
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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED 4-STOREY RESIDENTIAL APARTMENT BUILDING
370 CAMBRIDGE STREET NORTH
CITY OF OTTAWA, ONTARIO

Project # 220214

Submitted to:

2250276 Ontario Inc
7 Charnwood Court
Ottawa, Ontario
K2E 7C9

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April 18, 2022 220214

2250276 Ontario Inc
7 Charnwood Court
Ottawa, Ontario
K2E 7C9

RE: GEOTECHNICAL INVESTIGATION
PROPOSED 4-STOREY RESIDENTIAL APARTMENT
370 CAMBRIDGE STREET NORTH
CITY OF OTTAWA, ONTARIO

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the above noted proposed residential development at 370 Cambridge Street North, City of Ottawa, Ontario (See Key Plan, Figure 1).

The purpose of the investigation was to:

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

2.0 BACKGROUND INFORMATION AND SITE GEOLOGY

2.1 Existing Conditions and Site Geology

The subject site for this assessment consists of about a 0.06 hectare (0.15 acres) rectangular shaped property located at 370 Cambridge Street North, City of Ottawa, Ontario (see Key Plan, Figure 1).

For the purposes of this assessment, project north lies in a direction parallel to Cambridge Street North, located immediately east of the site. The site is currently occupied by a multi-unit residential building, which is to be demolished prior to construction.

Surrounding land use is mostly residential development. The site is bordered to the north by residential development, to the east by Cambridge Street followed by residential development, to the west by Arthur Lane followed by a Highway 417.

The ground surface at the site is currently graded such that surface water drains from the front of the building to the east and the rear of the building to the west.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by glacial till and/or shallow bedrock. Bedrock geology maps indicate that the bedrock underlying the site consists of limestone with shaley partings of the Ottawa formation.

Based on a review of available borehole information, the overburden at and near the site likely consists of some 0 to 2 metres of glacial till followed by limestone bedrock.

2.2 Proposed Development

It is understood that preliminary plans are being prepared for the construction of a 4-storey, 20 unit residential apartment building. There is proposed parking at the rear of the site. It is understood that the building will be wood framed with some brick veneer and conventional concrete spread footing foundations, with a full basement containing 3 residential units and the mechanical and electrical rooms. The proposed building will be serviced by municipal water and sanitary services.

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

3.0 PROCEDURE

The field work for this investigation was carried out on April 6, 2022, at which time three boreholes, numbered BH1 to BH3 were put down at the site using a truck mounted drill rig equipped with a hollow stem auger owned and operated by CCC Geotechnical & Environmental Drilling of Ottawa, Ontario. Borehole BH1 was put down in front of the existing building and boreholes BH2 and BH3 were put down within the parking lot at the rear of the property.

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

Sampling of the overburden materials encountered at each 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. All of the boreholes were put down to bedrock at the site. The soils were classified using the Unified Soil Classification System.

As native soils were not encountered within the boreholes, laboratory testing of soil samples was not completed for this site.

The field work was supervised throughout by members 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 is given in the attached Record of Borehole Sheets. The approximate locations of the boreholes are shown on the attached Site Plan, Figure 2.

4.0 SUBSURFACE CONDITIONS

4.1 General

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

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

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

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

4.2 Fill

Fill materials consisting of asphaltic concrete and grey crushed granular stone were encountered from the surface in boreholes BH2 and BH3, and topsoil fill was encountered from the surface in borehole BH1. These materials were underlain by black silty sand fill materials in all boreholes. The fill materials were encountered to depths of about 0.9 to 1.1 metres. The fill materials had blow counts from 5 to 12 blows per 0.3 metres, indicating a loose to compact state of packing. The fill materials were fully penetrated at all borehole locations.

4.3 Bedrock

Beneath the topsoil and fill materials, boreholes BH1, BH2 and BH3 encountered limestone bedrock at depths of about 0.9, 1.1 and 0.9 metres, respectively, below the existing ground surface. Refusal at the bedrock surface was encountered in all boreholes.

4.4 Groundwater

Some groundwater was observed in borehole BH1 at a depth of about 0.9 metres below the existing ground surface. Boreholes BH2 and BH3 were dry at the time of drilling. It should be noted that the groundwater levels may be higher during wet periods of the year.

5.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

5.1 General

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

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

5.2 Foundation for Proposed Residential Building

With the exception of the fill materials, the subsurface conditions encountered within the test holes are suitable for the support of the proposed residential building on conventional spread footing foundations. Excavations for the proposed foundations should be taken through the fill materials to expose the bedrock subgrade.

5.3 Foundation Design and Bearing Capacity

It is suggested that the building be founded either directly on the underlying bedrock or on engineered fill placed on the underlying bedrock. The underside of footings can be stepped as necessary to facilitate placement on the bedrock.

The foundation of the proposed residential building may be placed on conventional pad and strip footings. A maximum allowable bearing pressure of 500 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 1000 kilopascals using ultimate limit states design may be used for the design of conventional strip or pad footings, a minimum of 0.6 metres in width, founded on relatively sound bedrock. Relatively sound bedrock consists of a hard relatively level bedrock surface free of loose material, rock shatter and loose fractured rock.

The foundation of the proposed residential building founded on engineered fill placed on the bedrock may be designed a maximum allowable bearing pressure of 400 kilopascals for serviceability limit states design and a factored ultimate bearing resistance of 800 kilopascals for ultimate limit states design for the design of convention strip or pad footings, a minimum of 0.6 metres in width.

There is no limit to the allowable landscape grade raise adjacent to the proposed building foundation is required. Total and differential settlement of the footings for the residential building designed and founded based on the above guidelines should be less than 15 millimetres and 10 millimetres, respectively.

The subgrade surfaces should be inspected and approved by geotechnical personnel prior to placement of any engineered fill or concrete.

5.4 Engineered Fill

It is recommended that the building be founded either on relatively sound bedrock or on engineered fill placed on relatively sound bedrock. It is not recommended that the footings be placed on both bedrock and engineered fill at different locations in the building.

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 100 percent of the standard Proctor maximum dry density. It is considered that the engineered fill should be compacted using dynamic compaction with a large diameter vibratory steel drum roller or diesel plate compactor. If a diesel plate compactor is used, the lift thickness may need to be restricted to less than 300 mm to achieve proper compaction. Compaction should be verified by a suitable field compaction test method.

To allow the spread of load beneath the footings, the engineered fill should extend out 0.5 metres horizontally from the edges of the footing then down and out at 1 horizontal to 1 vertical, or flatter. The excavations for the proposed residential building should be sized to accommodate this fill placement.

5.5 Foundation Excavation

Any excavation for the proposed structure will likely be carried out through fill materials (topsoil, asphalt, crushed stone, silty sand) to bear upon the limestone bedrock. The sides of the excavations should be sloped in accordance with the requirements of Ontario Regulation 213/91, s. 226 under the Occupational Health and Safety Act. According to the Act, the fill materials at the site can be classified as Type 3 soil above bedrock and Type 1 below the bedrock surface, however this classification should be confirmed by qualified individuals as the site is excavated and if necessary, adjusted.

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

5.5.1 Bedrock Removal

Small amounts of bedrock removal can most likely be carried out by hoe ramming and heavy excavating equipment. To limit the effects of the bedrock on the adjacent structures and municipal services it is recommended that line drilling be used in conjunction with hoe ramming to reduce the effort required to fracture and remove bedrock. Due to the proximity of adjacent structures and municipal services, blasting is not recommended. It is considered that where large amounts of bedrock are removed by hoe ramming, the hoe ramming could also introduce significant vibrations through the bedrock. As such it is considered that pre-excavation surveys of nearby structures and existing utilities should also be completed before extensive hoe ramming.

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

It is expected that bedrock will be encountered during excavating for the building foundation and site services. It is considered that where large amounts of bedrock are removed by hoe ramming, the hoe ramming could also introduce significant vibrations through the bedrock. The ground induced vibration can be mitigated by the use of line drilling in combination with the hoe ramming and by operating only one piece of equipment at a time during the removal of the bedrock. Line drilling will reduce the effort required to fracture the rock reducing the size of the equipment and hoe ram needed to remove the rock and the duration of the hoe ramming. If the size of the equipment used is limited to less than 30 tonnes, the level of ground induced vibration is expected to be less than that which would cause damage to the adjacent structures or city municipal services.

5.5.3 Ground Water in Excavation and Construction Dewatering

Some groundwater was encountered in borehole BH1 at a depth of about 0.9 metres below the existing ground surface at the time of drilling on April 6, 2022. Boreholes BH2 and BH3 were dry at the time of drilling on April 6, 2022. As the building will be founded on shallow bedrock, water intrusion into the excavation is not a concern and dewatering will not be required. As such a permit to take water will not be required prior to excavation. If groundwater is encountered, at minimum registration on the Environmental Activity Sector Registry as per O. Reg 63/15 is expected to be required.

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

Since the building is to be founded on shallow bedrock and all adjacent buildings are also founded on shallow bedrock, dewatering of the foundation will not remove water from any historically saturated soils that are important for the support of any building. As such dewatering of the foundation or site services excavations, if required, will not have a detrimental impact on the adjacent structures.

5.6 Frost Protection Requirements for Spread Footing Foundations

In general, 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.

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.

Where the proposed building foundations are placed on sound bedrock or on engineered fill over bedrock, the subgrade materials would be considered to be non susceptible to frost action and no frost protection for the foundations is required.

5.6 Foundation Wall Backfill

To prevent possible foundation frost jacking, the backfill against any unheated or insulated walls or isolated walls or piers should consist of free draining, non-frost susceptible material. If imported material is required, it should consist of sand or sand and gravel meeting OPSS Granular B Type I grading requirements.

Alternatively, foundations could be backfilled on the exterior with native material in conjunction with the use of an approved proprietary drainage layer system (such as Platon System Membrane) against the foundation wall. There is potential for possible frost jacking of the upper portion of some types of these drainage layer systems if frost susceptible material is used as backfill. To mitigate this potential, the upper approximately 0.6 metres of the foundation should be backfilled with non-frost susceptible granular material.

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

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

$$P = k_0 (\gamma h + q)$$

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

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

5.6.1 Foundation and Under Slab Drainage

A conventional, perforated perimeter drain, with a 150 millimetre surround of 20 millimetre minus crushed stone, should be provided at the founding level for the cast-in-place concrete basement floor slab and should lead by gravity flow to the City Storm Sewer or to a sump. Under floor slab drains should be placed beginning at the inside edge of the foundation wall and should be spaced a maximum of 5 metres apart. The under floor slab drains should also be directed to the City Storm Sewer or sump. The sump discharge should be equipped with a backup flow protector. If the perimeter drain tile is discharged by gravity to the Storm Sewer a backup flow valve must be used. If a sump is used, the sump should be equipped with a backup pump and generator. The sump discharge should be equipped with a backup flow protector

5.7 Basement Floor Slab

As stated above, it is expected that the proposed building will be founded on bedrock or on an engineered pad placed on bedrock. For predictable performance of the proposed concrete basement floor slab all existing fill material and any otherwise deleterious material should be removed from below the proposed floor slab areas. The exposed bedrock surface should then be inspected and approved by geotechnical personnel.

In order to accommodate the under slab drains, the exposed subgrade should be built up from the underside of footing level with a minimum thickness of 0.3 metres of 20 or 25 mm clear stone. Provided the basement floor slab is not relied on to support any structural elements within the building, the clear stone could be extended to the underside of the basement floor. The clear stone should be well consolidated with a minimum of 3 passes per 0.3 metres of thickness using a large diesel plate compactor.

If the basement floor slab is to be relied on to support sensitive equipment or structural building elements, the 0.3 metres of clearstone should be well consolidated (as described above) then covered with a minimum 6 oz/yd^2 non woven geotextile followed by crushed stone meeting OPSS Granular A. The granular A 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. The slab should be cut as soon as it is possible to work on the slab without damaging the surface of the slab.

5.8 Seismic Design for the Proposed Residential Building

5.8.1 Seismic Site Classification

Based on the limited information from the boreholes, for seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response for the bedrock is Site Class C.

5.8.2 National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.280 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The results of the test are attached following the text of this report.

5.8.3 Potential for Soil Liquefaction

As indicated above, the results of the boreholes indicate that the subsurface conditions consist of a thin layer of fill materials followed by bedrock. The proposed building will be founded on the bedrock or on engineered fill placed on the bedrock.

The bedrock is not considered to be liquefiable under seismic conditions.

Therefore, it is considered that no damage to the proposed residential building will occur due to liquefaction of the native subgrade under seismic conditions.

6.0 SITE SERVICES

6.1 Excavation

The excavations for the site services will be carried out through fill materials (topsoil, asphalt, crushed stone and silty sand) and potentially bedrock. For the purposes of Ontario Regulation 213/91 the soils at the site can be considered to be Type 3 soil above bedrock, and Type 1 below the bedrock surface. Work within an excavation in the bedrock should follow the requirements of Ontario Regulation 213/91 in particular O.Reg 213/91 S230 – S233. Excavation walls within bedrock may be made near vertical. 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.

It is expected that bedrock will be encountered during excavating for site services. It is considered that were large amounts of bedrock are removed by hoe ramming, the hoe ramming could also introduce significant vibrations through the bedrock. It is recommended that where large amounts of bedrock are to be removed by hoe ramming, line drilling techniques be combined with the hoe ramming. As such it is considered that pre-excavation surveys of nearby structures and existing utilities should also be completed before extensive hoe ramming.

Groundwater was encountered at a depth of about 0.9 metres, or at the surface of bedrock, in borehole BH1. Boreholes BH2 and BH3 were dry at the time of excavation, however were not advanced into the bedrock to the expected depth of the services. As such it is uncertain where the groundwater elevation is with respect to the service elevations. Based on available information it is unlikely that a permit to take water will be required to dewater the service trench. It is considered however that an ESR may be required.

6.2 Pipe Bedding and Cover Materials

It is suggested that the service pipe bedding material consist of at least 150 millimetres of granular material meeting OPSS requirements for Granular A. A provisional allowance should, however, be made for sub-excavation of any existing fill or disturbed material encountered at sub-grade level. Granular material meeting OPSS specifications for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as bedding or sub-bedding material should not be permitted.

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

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

6.3 Trench Backfill

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

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, granular fill material should be used as backfill between the roadway sub-grade level and the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway.

As there is limited native material expected to exist onsite, imported granular material will likely have to be used. Where imported granular materials are used, suitable frost tapers should be used OPSD 802.013. Frost tapers are not required below the bedrock surface.

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

7.0 TREES

The site is underlain by a thin layer of fill materials over bedrock, which is not considered to be susceptible to shrinkage caused by changes to moisture content. As such, it is considered that there are not any increased separation distances or limitations to the type of trees planted onsite.

The effects of existing and future trees on the adjacent buildings, services and other ground supported structures should be considered in the landscaping design.

8.0 CONSTRUCTION CONSIDERATIONS

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

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

The subgrade for the site services 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.

Any native topsoil and fill materials 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
April 18, 2022
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: 2250276 Ontario Inc
LOCATION: 370 Cambridge Street North
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 220214
DATE OF BORING: 22-4-6
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o (0 20 40 60 80 100)	DYNAMIC CONE PENETRATION TEST blows/300 mm (0 20 40 60 80 100)	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	Topsoil, black silty sand, some gravel (FILL)	0.00			1	SS	5				Some groundwater observed at about 0.9 metres below existing ground surface, April 6, 2022.
0.5					2	SS	100				
	Refusal on BEDROCK	0.91									

DEPTH SCALE: 1 to 10
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: CI
CHECKED: SD

BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: 2250276 Ontario Inc
LOCATION: 370 Cambridge Street North
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 220214
DATE OF BORING: 22-4-6
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o (0 20 40 60 80 100)	DYNAMIC CONE PENETRATION TEST blows/300 mm (0 20 40 60 80 100)	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	ASPHALTIC CONCRETE	0.00									
	Grey crushed granular stone (FILL)	0.03									
	Black silty sand, some gravel, glass (FILL)	0.10			1	SS	12				Borehole dry, April 6, 2022.
0.5											
					2	SS	100				
1.0											
	Refusal on BEDROCK	1.02									

DEPTH SCALE: 1 to 10
LOGGED: CI
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
CHECKED: SD

BOREHOLE BH3

PROJECT: Proposed Residential Development
CLIENT: 2250276 Ontario Inc
LOCATION: 370 Cambridge Street North
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 220214
DATE OF BORING: 22-4-6
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o (0 20 40 60 80 100)	DYNAMIC CONE PENETRATION TEST blows/300 mm (0 20 40 60 80 100)	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	ASPHALTIC CONCRETE	0.00									
	Grey crushed granular stone (FILL)	0.03									
	Black silty sand, some gravel (FILL)	0.10									
					1	SS	10				Borehole dry, April 6, 2022.
0.5											
					2	SS	100				
	Refusal on BEDROCK	0.91									

DEPTH SCALE: 1 to 10
LOGGED: CI
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
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 drill 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

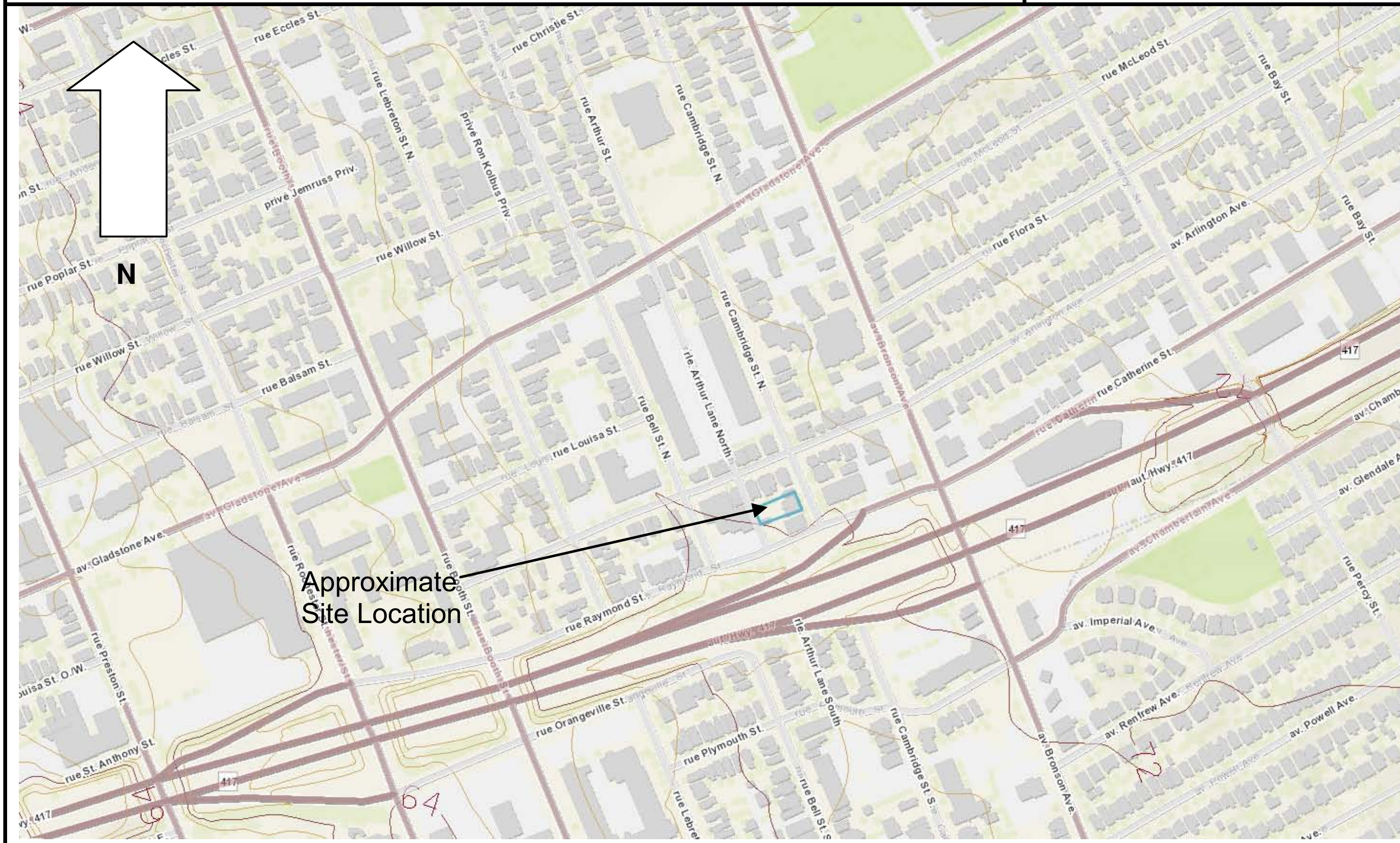
Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	over 100

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
$' effective angle of friction
r unit weight of soil
y' unit weight of submerged soil
cr normal stress

KEY PLAN	FIGURE 1

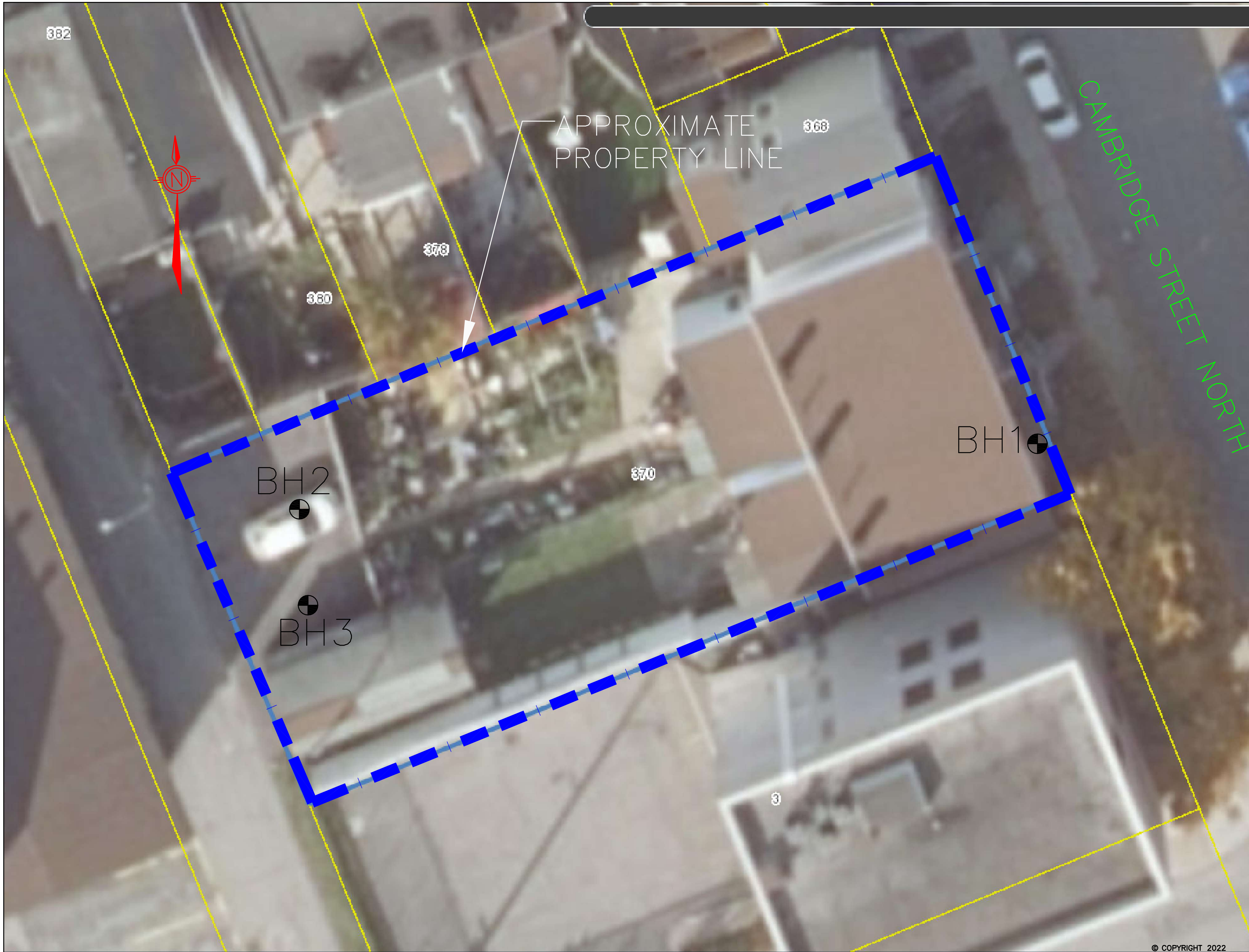


NOT TO SCALE

Kollaard Associates
Engineers

Project No. 220214

Date April 2022



DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

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

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CLIENT:
2250276 ONTARIO INC.

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION:
370 CAMBRIDGE STREET NORTH
CITY OF OTTAWA, ON

DESIGNED BY: --	DATE: APRIL 18, 2022
DRAWN BY: DT	SCALE: N.T.S
KOLLAARD FILE NUMBER: 220214	

National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

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

Site: 45.405N 75.704W **User File Reference:** 370 Cambridge Street N 2022-04-08 18:23 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.446	0.247	0.148	0.044
Sa (0.1)	0.522	0.299	0.186	0.061
Sa (0.2)	0.438	0.254	0.161	0.055
Sa (0.3)	0.333	0.195	0.124	0.043
Sa (0.5)	0.237	0.138	0.088	0.031
Sa (1.0)	0.118	0.069	0.044	0.015
Sa (2.0)	0.056	0.033	0.020	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.280	0.163	0.101	0.033
PGV (m/s)	0.196	0.110	0.068	0.021

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

References

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

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

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

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