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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
971 MONTREAL ROAD
CITY OF OTTAWA, ONTARIO

Project # 200543

Submitted to:

Développements Proximi-T Inc.
3500 Atwater, Suite 6
Montreal, Quebec
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DISTRIBUTION

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July 31, 2020 (Revised February 8, 2022)

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Développements Proximi-T Inc.
3500 Atwater, Suite 6
Montreal, Quebec
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RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
971 MONTREAL ROAD
CITY OF OTTAWA, ONTARIO

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed residential development to be located at 971 Montreal Road, in the City of Ottawa, Ontario. The proposed development will consist of a nine storey residential apartment building having some 82 units. The proposed building will be provided with one storey of underground parking.

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 0.18 hectare (0.44 acres) rectangular shaped property located on the north side of Montreal Road, about 932 metres west of the intersection of Montreal Road and Blair Road, in the City of Ottawa, Ontario (see Key Plan, Figure 1).

For the purposes of this assessment, project north lies in a direction perpendicular to Montreal Road which is located immediately south of the subject site.

The subject site is bordered on the east and west by commercial development, on the south by Montreal Road followed by high density residential development and on the north by light industrial development. The site is currently occupied by a commercial development consisting of a Chinese Restaurant – (Dragon Restaurant) with a second floor apartment.

The ground surface at the site is graded such that surface water drains from the front of the building towards Montreal Road and then to the west and from the west and north side of the building toward the northwest corner of the subject site.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by a thin veneer of glacial till followed by bedrock. Bedrock geology maps indicated that the bedrock underlying the site consists of limestone with some shaly partings of the Ottawa Formation.

The ground surface elevations at the borehole locations were extrapolated from a survey plan prepared by Farley, Smith & Denis Surveying Ltd., File No:628-20, dated January 12, 2021.

2.2 Proposed Development

It is understood that plans are being prepared for the construction of an 82 unit, nine storey residential apartment building with a proposed building footprint of approximately 720 square metres and one storey of underground parking with approximately 42 parking spaces. The apartment building will be serviced by municipal water and sanitary site services. It is understood that the apartment building will be of concrete construction with conventional concrete spread footing foundations and concrete floor slab.

The proposed apartment building will be provided with an asphaltic concrete surfaced access roadway and a ramp to the underground parking.

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

3.0 PROCEDURE

The field work for this investigation was carried out on July 10, 2020, at which time three boreholes/coreholes, numbered BH1, BH2 and BH3 were put down at the site. The three boreholes were put down within the building footprint. The boreholes were put down using a truck mounted drill rig equipped with a hollow stem auger owned and operated by George Downing Estate Drilling of Hawkesbury, Ontario.

The subsurface soil conditions encountered at the boreholes were classified based on visual and tactile examination of the samples recovered (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) and the results of the standard penetration tests. The soils were classified using the Unified Soil Classification System. Groundwater conditions at the test holes were noted at the time of the field work. A standpipe was installed at each of the borehole locations for subsequent ground water level monitoring and for the Phase II Environmental Site Assessment being carried out by CM3 Environmental.

No samples were submitted for physical or chemical laboratory testing as only small amounts of fill samples were recovered from each of the boreholes overlying shallow bedrock.

Based on known shallow bedrock conditions at the site, it was expected that some bedrock hoe-ramming will be required in order to achieve the proposed underside of footing elevation for the underground parking for the proposed apartment building and for the installation of the site services. Accordingly, the bedrock was cored at all three boreholes using diamond drilling procedures.

Any soil samples from the boreholes, where possible, were recovered from cuttings of the boreholes. The soil samples were classified on site, placed in a sealed plastic bag and transported to our laboratory. Rock samples from all three boreholes BH1, BH2 and BH3 were recovered using a core barrel. The rock samples were classified on site, placed in wooden and hard cardboard core boxes and transported to our laboratory. The rock cores are shown as RC on the Record of Borehole sheets.

Diamond drilling was carried out in all of the boreholes to determine the nature and quality of the bedrock. The recovery value and the rock quality designation value (RQD) were calculated for the drilled section (core run) of bedrock. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of sound rock pieces longer than 100 millimetre in one core run over the length of the core run. Both values are indicative of the quality of the bedrock.

The field work was supervised throughout by a member of our engineering staff who located the boreholes in the field, logged the subsurface conditions encountered 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 following this report. The approximate locations of the boreholes are shown on the attached Site Plan, Figure 2.

The location of the boreholes were identified in the field by paint marks and the ground surface elevations were provided by CM3 Environmental.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, a description of the subsurface conditions encountered at the test holes are provided in the attached Record of Borehole Sheets. The test hole logs indicate the subsurface conditions at the specific test hole 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 test hole locations may vary from the conditions encountered at the test holes.

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). No soil samples were submitted to a laboratory as only limited amounts of fill materials and/or a thin veneer of glacial till (BH3) were recovered from the boreholes followed by shallow bedrock at all of the test hole locations. The soils were classified in the field based on visual and tactile inspection (ASTM D2488).

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 test hole 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 site during the site visit.

4.2 Fill

Fill materials were encountered from the surface at all three boreholes and ranged in thickness from about 0.2 to 3.05 metres. The fill materials were observed to consist of asphaltic concrete followed by grey crushed stone, then by grey brown sand, some gravel, topsoil/organics and a trace of clay and brick. The fill material was fully penetrated at all three borehole locations.

4.3 Topsoil

At borehole BH3, about a 0.5 metre thickness of black topsoil with a trace of sand and gravel was encountered below the fill materials. The material was classified as topsoil based on colour and the presence of organic materials and is intended as identification for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.4 Glacial Till

A thin deposit of glacial till was encountered beneath the fill materials and topsoil at borehole BH3. The glacial till consisted of gravel, cobbles and boulders, in a matrix of grey silty sand/sandy silt with a trace of silty clay. The results of standard penetration testing carried out in the glacial till material, which range from 37 to 50 blows per 0.3 metres, indicating a dense to very dense state of packing.

Practical refusal on the surface of a large boulder was encountered at about 0.7 metres below the existing ground surface. Recovery within the spoon was poor and most of the sample was lost due to the boulders encountered within the borehole. Practical refusal for advancement on the surface of bedrock was encountered at about 2.72, 3.05, and 1.37 metres, respectively, below the existing ground surface for boreholes BH1, BH2 and BH3.

4.5 Bedrock

As indicated above, bedrock was encountered at all three of the boreholes at about 2.72, 3.05, and 1.37 metres, respectively. All of the boreholes were extended by coring to verify the quality of the upper bedrock.

The boreholes were continued into the bedrock using diamond coring to depths of about 7.16, 7.19 and 6.13 metres below the existing ground surface. A visual assessment of the bedrock indicated that the bedrock is grey limestone. The total core run length in each borehole was 4.44, 4.14 and 4.76 metres, respectively for boreholes, BH1, BH2 and BH3. Fracturing of the core samples is mostly along near horizontal bedding planes.

A measure of the condition of the bedrock core obtained from the boreholes can be represented as a percentage of Total Core Recovery (T.C.R.), Solid Core Recovery (S.C.R.) and Rock Quality Designation (R.Q.D.). There was no measurable amount of core lost during recovery of the bedrock giving a T.C.R. value of 100 percent.

The S.C.R. average value for the cores is about 96.9 percent.
From the bedrock surface to about 1.5 metres below the bedrock surface the S.C.R. = 95 percent.
Between 1.5 and 3.0 metres below the bedrock surface the S.C.R. = 95.9 percent.
Between 3.0 and 4.5 metres below the bedrock surface the S.C.R. = 100 percent
The R.Q.D. values for the cores vary as follows:
From the bedrock surface to 1.5 metres below the bedrock surface the R.Q.D = 90 to 92.5 percent.
Between 1.5 and 3.0 metres below the bedrock surface the R.Q.D = 73 to 90 percent.
Between 3.0 and 6.1 metres below the bedrock surface the R.Q.D = 93 to 100 percent.

Using the classification table, the R.Q.D. index for the rock mass can be classified as fair to excellent (R.Q.D. = 73 to 100%).

4.6 Groundwater

On July 16, 2020, groundwater was measured in standpipes installed in all three boreholes below the existing ground surface as follows (elevations are referenced to a local datum):

Borehole	Ground Surface Elevation (m)	Ground Water Elevation (m)	Depth to Groundwater (m)
BH1	105.92	101.60	4.32
BH2	105.04	101.67	3.37
BH3	104.96	100.91	4.05

It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

5.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

5.1 General

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

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

5.2 Foundations for Proposed Residential Apartment Building

With the exception of the topsoil and fill materials**,** the subsurface conditions encountered within the test holes are suitable for the support of the proposed apartment building with underground parking on conventional spread footing foundations. Excavations for the proposed foundations should be taken through the topsoil, fill materials and glacial till to expose the bedrock subgrade.

5.2.1 Proposed Apartment Building

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 footing can be stepped as necessary to facilitate placement on the bedrock.

The foundation of the proposed apartment building with a parking structure foundation may be placed on conventional pad and strip footings. A maximum allowable bearing pressure of 2000 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 4000 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 sound bedrock. Sound bedrock consists of a hard relatively level bedrock surface free of loose material, rock shatter and fractured rock.

No maximum allowable landscape grade raise adjacent to the proposed building foundation is required. Total and differential settlement of the footings for the apartment 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.

5.3 Engineered Fill below Building Foundation

It is not recommended that the footings be placed on both bedrock and engineered fill or native glacial till at different locations in the building. As such engineered fill below the footing is not recommended. Should the bedrock surface be below the proposed underside of footing elevation, it is recommended that the bedrock subgrade be raised to the proposed underside of footing using a concrete sub-footing or that the foundation walls be extended.

5.4 Frost Protection Requirements for Spread Footing Foundations

Part 4 of the Ontario Building code indicates that the depth of foundation shall be below the level of potential damage including damage from frost action with that provision that the bearing surface need not be below the level of potential frost (Part 4.2.4.4 (2)) where the foundation overlies material not susceptible to frost action.

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.

5.5 Foundation Wall Backfill and Drainage

To prevent possible foundation frost jacking, the backfill against unheated walls or isolated walls or piers should consist of the 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 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.

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 basement floor parking area and should lead by gravity flow to a sump to reduce the potential for buildup of hydrostatic pressure below the parking garage floor. The sump should be equipped with a backup pump and generator.

The under floor 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 drain should also be directed to the sump. The sump discharge should be equipped with a backup flow protector.

It is considered that in view of the groundwater conditions observed at the boreholes, the above perimeter drainage system should adequately handle any groundwater seepage to the basement or elevator pit.

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

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

Where:

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

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

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

5.6 Building Structure Floor Slab

For predictable performance of the proposed concrete floor slab any existing topsoil, fill materials, soft or loose and any deleterious material should be removed from below the proposed floor slab area. The exposed native sub-grade surface should then be inspected and approved by geotechnical personnel. Should complete removal of all deleterious material result in a subgrade below the concrete floor structure, the subgrade can be built up using engineered fill.

The engineered 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 98 percent of the standard Proctor maximum dry density. Alternatively clear crushed 20 mm minus stone could be used immediately below the concrete floor slab provided the clear stone is well compacted prior to concrete placement.

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.

5.7 Seismic Design for the Proposed Apartment 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 B Rock. The subsurface conditions below the proposed footing design level consists of a thin veneer of glacial till over bedrock at a depth of about 1.4 to 2.7 metres. As indicated above, the bedrock is sound at a depth of 1.5 metres below the bedrock surface with an RQD of 73 to 90 percent. The bedrock consists of limestone.

5.8 National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.287 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The results of the test are attached following the text of this report.

5.9 Potential for Soil Liquefaction

As indicated above, the results of the boreholes indicate that the site is underlain by a thin veneer of glacial till overlying shallow bedrock and/or fill materials overlying shallow bedrock. As such, it is considered that no damage to the proposed residential building should occur due to liquefaction of the bedrock under seismic conditions.

5.9.1 Dewatering of Foundation Excavation

Bedrock was encountered at about 1.4 to 2.7 metres below the existing ground surface. On July 16, 2020, groundwater was measured in the standpipes placed within the boreholes by CM3 Environmental professional staff at about 4.3, 3.4 and 4.1 metres in boreholes BH1, BH2 and BH3, respectively, below the existing ground surface on July 16, 2020. The ground water level encountered corresponds to about 1.6, 0.3 and 2.7 metres, respectively, below the surface of the bedrock.

The excavation for the proposed building will be extended one storey below the existing ground surface and into the bedrock subgrade. Adjacent buildings will be either founded either on bedrock or on a relatively thin overburden layer above the bedrock above the ground water level.

Since the groundwater level is below the surface of the bedrock, lowering the groundwater level will not result in settlement as bedrock is not susceptible to shrinking and settling due to groundwater lowering.

Any groundwater inflow from the overburden deposits into the excavations should be controlled by pumping from filtered sumps within the excavations. There are no settlement concerns to the adjacent dwellings and other buildings due to groundwater removal from the foundation excavation at this site.

Based on the results of the boreholes, we do not expect significant groundwater inflow into the excavation for the proposed development. However, if groundwater is encountered during excavation for the proposed services or building foundation, a Permit to Take Water (PTTW) may be required for pumping rates exceeding 400,000 Litres/day. If groundwater is encountered, at minimum, registration on the Environmental Activity Sector Registry (EASR) as per O.Reg. 63/16 is expected to be required.

6.0 SITE SERVICES

6.1 Excavation

The excavations for the site services will be carried out through fill materials, topsoil, a thin layer of glacial till and/or bedrock. 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.

For the purposes of Ontario Regulation 213/91, the subsurface conditions at the site can be considered to be Soil Type 1. 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.

It is expected that bedrock will be encountered during excavating for site services. Small amounts of bedrock removal, can most likely be carried out by hoe ramming and heavy excavating equipment. Where larger amounts of bedrock removal are required it may be more economically feasible to use drill and blasting techniques which should be carried out under the supervision of a blasting specialist engineer. Monitoring of the blasting should be carried out throughout the blasting period to ensure that the blasting meets the limiting vibration criteria established by the specialist engineer. Pre-blast condition surveys of nearby structures and existing utilities are essential. It is also 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.

6.2 Pipe Bedding and Cover Materials

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

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

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

6.3 Trench Backfill

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

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, acceptable native materials should be used as backfill between the roadway sub-grade level and the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway. Where native material consists of bedrock, Granular A or Granular B Type 2 may be used for backfill.

Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future roadway areas. Any boulders larger than 300 millimetres in size should not be used as service trench backfill. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I. If the native material is not suitable for backfill, imported granular material may have to be used. If imported granular materials are used, suitable frost tapers should be used OPSD 802.013.

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

7.0 ACCESS ROADWAY AND PARKING LOT PAVEMENTS

7.1 Subgrade Preparation

In preparation for pavement construction at this site any fill and topsoil and any soft, wet or deleterious materials should be removed from the proposed access roadway and parking lot area. The exposed subgrade surface should then be proof inspected and approved by geotechnical personnel. Any soft or unacceptable areas evident should be subexcavated and replaced with suitable earth borrow material. The subgrade should be shaped and crowned to promote drainage of the roadway and parking area granulars. Following approval of the preparation of the subgrade, the pavement granulars may be placed.

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

7.2 Access Roadway Pavements

In preparation for pavement construction at this site the topsoil and any soft, wet or deleterious materials should be removed from the proposed access roadway area. 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 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 granular may be placed.

For any areas of the site that require the sub-grade to be raised to proposed roadway area sub-grade level, the material used should consist of OPSS select sub-grade material or OPSS Granular B Type I or Type II. 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 or hot mix asphalt concrete (HL3) over
> 150 millimetres of OPSS Granular A base over
> 300 millimetres of OPSS Granular B, Type II subbase
> (50 or 100 millimetre minus crushed stone)

Performance grade PG 58-34 asphaltic concrete should be specified.

Compaction of the granular pavement materials should be carried out in maximum 300 millimetre thick loose lifts to 100 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

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

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 apartment 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 to support the concrete floor slab and within the access roadway pavement structure should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The sub-grade 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 access roadway granular materials to ensure the materials meet the specifications from a compaction point of view.

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
5.Aug.2020
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: Developpments Proximi-T Inc.
LOCATION: 971 Montreal Road, Ottawa, ON
PENETRATION TEST HAMMER: N/A

PROJECT NUMBER: 200543
DATE OF BORING: July 10, 2020
SHEET 1 of 1
DATUM: Geodetic

DEPTH SCALE (meters)	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (M)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m
	Ground Surface	105.92			
0	ASPHALTIC CONCRETE	0.00			
	Grey crushed stone (FILL)				
	Sand, gravel, organics, trace clay (FILL)		1	SS	8
1			2	SS	6
			3	SS	3
2			4	SS	3
			5	SS	50
	Advanced corehole through Limestone BEDROCK	103.20 2.72			
3			1	RC	MCR=92.5% RQD=Excellent
4–5			2	RC	MRC=79.2% RQD=Good
6			3	RC	MCR=93% RQD=Excellent
7	End of corehole in Limestone BEDROCK	98.76 7.16			
8					

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 measured in borehole by CM3 Environmental at approximately 4.3 metres below existing ground surface on July16, 2020.

DEPTH SCALE: 1 to 50
BORING METHOD: Coring
AUGER TYPE: NQ Core Barrel
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: Developpments Proximi-T Inc.
LOCATION: 971 Montreal Road, Ottawa, ON
PENETRATION TEST HAMMER: N/A

PROJECT NUMBER: 200543
DATE OF BORING: July 10, 2020
SHEET 1 of 1
DATUM: Local

DEPTH SCALE (meters)	DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m
	Ground Surface	105.04			
0	ASPHALTIC CONCRETE	0.00			
	Grey crushed stone (FILL)		1	SS	26
	Sand, gravel, topsoil, organics, trace brick, clay (FILL)		2	SS	11
			3	SS	9
			4	SS	9
			5	SS	50
3	Advanced corehole through Limestone BEDROCK	101.99 3.05	1	RC	MCR=92%
			2	RC	MCR=73% RQD=Fair
			3	RC	MCR=92% RQD=Excellent
7	End of corehole in Limestone BEDROCK	97.85 7.19			

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 measuredin borehole by CM3 Environmental at approximately 3.4 metres below existing ground surface on July16, 2020.

DEPTH SCALE: 1 to 50
BORING METHOD: Coring
AUGER TYPE: NQ Core Barrel
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH3

PROJECT: Proposed Residential Development
CLIENT: Developpments Proximi-T Inc.
LOCATION: 971 Montreal Road, Ottawa, ON
PENETRATION TEST HAMMER: N/A

PROJECT NUMBER: 200543
DATE OF BORING: July 10, 2020
SHEET 1 of 1
DATUM: Local

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH Cu, kPa / REM. SHEAR STRENGTH Cu, kPa	DYNAMIC CONE PENETRATION TEST blows/300 mm	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface	104.96							
0	ASPHALTIC CONCRETE	0.00							
	Grey crushed stone (FILL)		1	SS	10				
	TOPSOIL, trace sand and gravel, practical refusal on large boulder	104.30 0.66							
1	Grey brown silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)		2	SS	37				
		103.59 1.37	3	SS	50				
2	Advanced corehole through Limestone BEDROCK		1	RC	MCR=48% RQD=Poor				
3			2	RC	MCR=100% RQD=Excellent				
5			3	RC	MCR=97% RQD=Excellent				
6	End of corehole in Limestone BEDROCK	98.83 6.13							Water measured in borehole by CM3 Environmental at approximately 4.1 metres below existing ground surface on July16, 2020.

DEPTH SCALE: 1 to 50
BORING METHOD: Coring
AUGER TYPE: NQ Core Barrel
LOGGED: DT
CHECKED: SD

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

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

PENETRATION RESISTANCE

Standard Penetration Resistance, N

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

Dynamic Penetration Resistance

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

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drih rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

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

SOIL DESCRIPTIONS

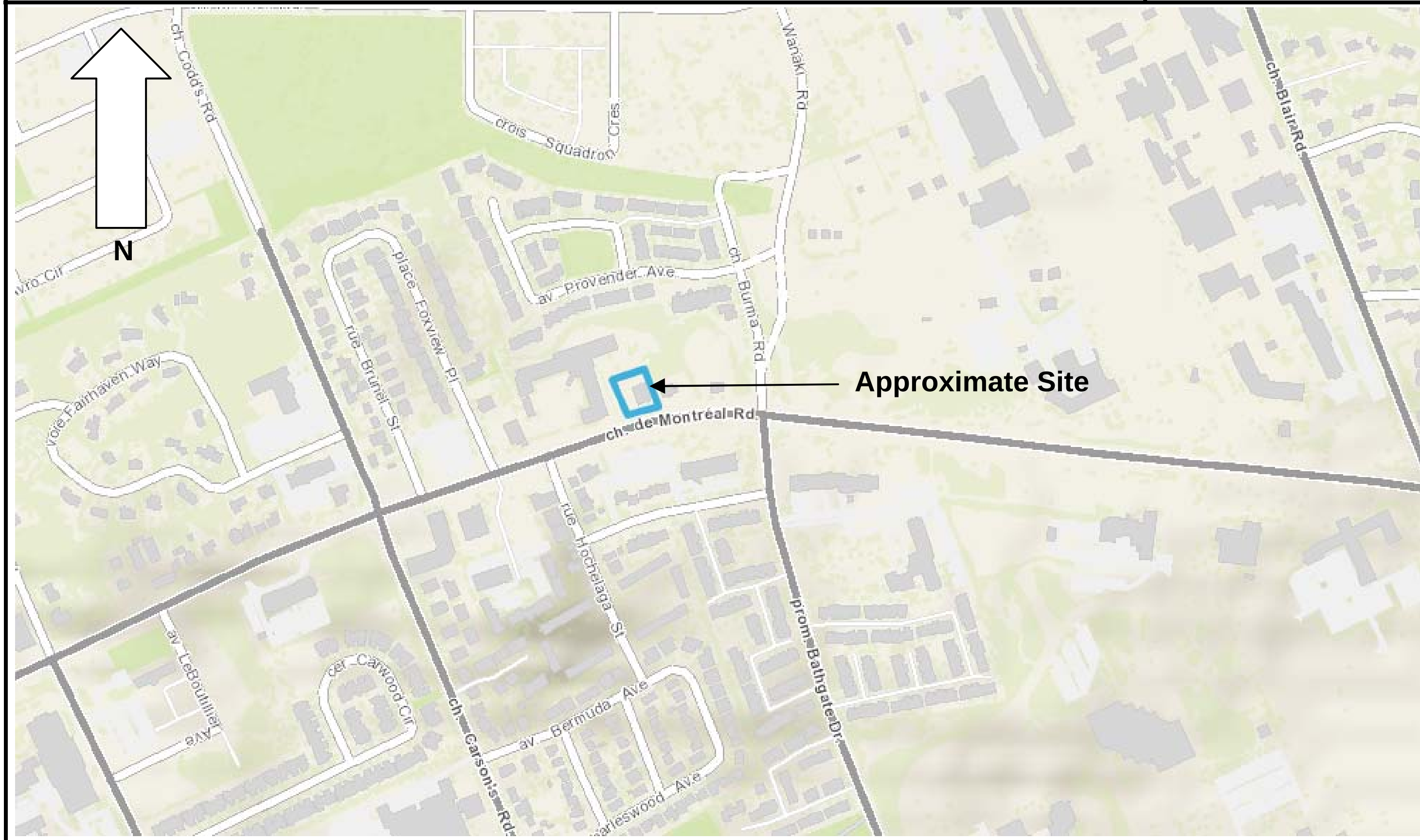
Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

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

LIST OF COMMON SYMBOLS

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

KEY PLAN	FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 200543

Date July 2020

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

REFERENCE: PLAN SUPPLIED BY CITY OF OTTAWA EMAPS.

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

REV.	NAME	DATE	DESCRIPTION
1.	DT	Jan 14/21	CORRECTED BOREHOLES #S

Kollaard Associates
Engineers
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KEMPTVILLE ONTARIO
K0G 1J0 FAX (613) 258-0475
http://www.kollaard.ca
(613) 860-0923
info@kollaard.ca

CLIENT:
DEVELOPMENTS PROXIMI-T INC.

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION:
971 MONTREAL ROAD
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
--	JULY 10, 2020
DRAWN BY:	SCALE:
DT	N.T.S

KOLLAARD FILE NUMBER:
200543

ATTACHMENT A

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: 45.448N 75.627W **User File Reference:** 971 Montreal Road, Ottawa 2020-07-21 18:22 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)	0.459	0.254	0.153	0.045
Sa (0.1)	0.535	0.307	0.191	0.062
Sa (0.2)	0.448	0.260	0.164	0.056
Sa (0.3)	0.340	0.199	0.126	0.044
Sa (0.5)	0.241	0.141	0.089	0.031
Sa (1.0)	0.119	0.070	0.045	0.015
Sa (2.0)	0.057	0.033	0.021	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.287	0.167	0.104	0.033
PGV (m/s)	0.200	0.112	0.069	0.022

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

References

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

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

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

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