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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
42 NORTHSIDE ROAD, BELLS CORNERS
CITY OF OTTAWA, ONTARIO

Project #211099

Submitted to:
Rohit Communities Ontario Inc.
550 91 Street SW, Suite 101
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T6X 0V1

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October 7, 2021 211099

Rohit Communities Ontario Inc.
550 91 Street SW, Suite 101
Edmonton, Alberta
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RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
42 NORTHSIDE ROAD
CITY OF OTTAWA, ONTARIO

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the above noted proposed residential building located immediately southeast of the intersection of Northside Road and Thorncliffe Place in the City of Ottawa, Ontario.

The purpose of the investigation was to identify the subsurface conditions at the site based on a limited number of boreholes. Based on the factual information obtained, Kollaard Associates Inc. was to provide guidelines on the geotechnical engineering aspects of the project design, including construction considerations, which could influence design decisions.

2.0 BACKGROUND INFORMATION AND SITE GEOLOGY

2.1 Existing Conditions and Site Geology

The subject property for this assessment consists of an area of about 0.12 hectares (0.30 acres). The site is currently occupied by a single storey commercial building used as a restaurant scheduled for demolition and removal for the proposed residential building (see Key Plan, Figure 1).

The site is bordered on the north by Northside Road, on the east by institutional development, on the south by an animal hospital and on the west by Thorncliff Place.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by deposits of clay and silt. Bedrock geology maps indicate that the bedrock underlying the site consists of dolomite and limestone of the Oxford formation.

Based on a review of available borehole information, the overburden at and near the site likely consists of some 12 metres of silty clay and silty sand, followed by limestone bedrock.

2.2 Proposed Development

It is understood that plans are being prepared for the construction of a five storey residential building with two levels of underground parking, access roadway and site services. The residential building has a typical floor plan area of approximately 827 m^2. The building is likely to be of wood frame construction with conventional spread footing foundations. 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 February 28 and 29, 2012 at which time three boreholes, numbered BH1, BH2 and BH3 were put down at the site using a truck mounted drill rig equipped with a hollow stem auger owned and operated by OGS Inc. of Almonte, Ontario.

Sampling of the overburden materials encountered at the boreholes 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 of about 9.2, 8.6 and 11.9 metres below the existing ground surface in BH1, BH2 and BH3, respectively. In situ vane shear testing was carried out in the cohesive materials encountered at all three boreholes. BH3 was continued below 11.9 metres with coring to confirm the presence of bedrock.

The subsurface soil conditions at BH1, BH2 and BH3 were identified based on visual examination of the samples recovered and the results of the in situ vane shear testing and standard penetration tests. Groundwater conditions at the boreholes were noted at the time of drilling. A standpipe was installed at BH3 for subsequent ground water level monitoring. The boreholes were loosely backfilled with the auger cuttings upon completion of drilling.

The field work was supervised throughout by a member of our engineering staff who located the boreholes in the field, logged the boreholes and cared for the samples obtained. A description of the subsurface conditions encountered at BH1, BH2 and BH3 are 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. The borehole logs indicate the subsurface conditions at the specific borehole locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than borehole locations may vary from the conditions encountered at the boreholes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification was in general completed by visual-manual procedures in accordance with ASTM 2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). 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 soils were classified using the Unified Soil Classification System.

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 material was encountered from the surface at all three boreholes extending to depths of about 0.8 to 0.9 metres. The fill was observed to consist of asphaltic concrete followed by grey crushed stone then grey brown silty clay with a trace of gravel. The fill material was fully penetrated at all three borehole locations.

4.3 Silty Clay

A deposit of grey brown to grey silty clay was encountered below the fill at the three borehole locations. The silty clay layers were encountered at all three boreholes at depths ranging from about 0.8 to 0.9 metres and fully penetrated at about 6.1, 6.5 and 7.1 metres, respectively below existing ground surface.

The results of the in situ vane shear testing gave undrained shear strength values ranging from about 37 to 142 kilopascals with an average value of 89.5 kilopascals. The results of the in situ vane shear testing and tactile examination carried out for the silty clay material indicate that the silty clay is very stiff to firm in consistency.

The upper about 0.8 to 3.0 metre portion of the silty clay has been weathered to a stiff to very stiff grey brown crust. Beneath the grey brown crust the silty clay becomes grey and decreases to firm in consistency. The results of in situ vane shear testing carried out in the softer grey silty clay gave undrained shear strength values ranging from about 37 to 68 kilopascals in the grey silty clay.

4.4 Glacial Till

A deposit of glacial till was encountered beneath the silty clay layer at all of the borehole locations. The glacial till consists of gravel and possible cobbles and boulders in a matrix of silty sand with some clay. The results of standard penetration testing carried out in the glacial till material, which range from 4 to 102 blows per 0.3 metres with an average value of 53 blows per 0.3 metres, indicate a loose to compact to very dense state of packing.

4.5 Bedrock

Bedrock was encountered at borehole BH3 about 9.26 metres below the existing ground surface. The bedrock was cored to verify the quality of the upper bedrock. The total core run length was 2.4 m. The bedrock consists of dolomite and limestone of the Oxford. Fracturing of the core samples is mostly along near horizontal bedding planes.

A measure of the condition of the bedrock core obtained from the borehole 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.). The amount of core lost during recovery of the bedrock was low, giving a T.C.R. value of 89 percent. It is noted that the majority of the core loss occurred in the upper portion of the core run.

The S.C.R. values vary from about 16 to 96 percent as follows:
From the bedrock surface to 0.5 meters below the bedrock surface the S.C.R. = 16 percent.
Between 0.5 and 2.4 m below the bedrock surface the S.C.R. = 98 percent.

The R.Q.D. values vary from 0 to 98 percent as follows:
From the bedrock surface to 0.5 meters below the bedrock surface the R.Q.D = 0 percent.
Between 0.5 and 1.4 m below the bedrock surface the R.Q.D = 95 percent.
Between 1.4 m and 2.4 m below the bedrock surface the R.Q.D = 98 percent.

4.6 Groundwater

Groundwater seepage was observed in BH1, BH2 and BH3 at the time of drilling at depths of about 3.0, 4.4 and 4.0 metres, respectively, below the existing ground surface. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

4.7 Sulphate, Resistivity and pH

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
pH	5.0 < pH	7.8 to 8.2	Basic

			Negligible concern
Resistivity	R < 20,000 ohm-cm	2220 to 3330	Corrosive to highly corrosive
Sulphates (SO_4)	SO_4 > 0.1%	0.04 to 0.07	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 range from 7.8 to 8.2, 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.

Corrosivity Rating for soils ranges from extremely corrosive to non-corrosive as follows:

Soil Resistivity (ohm-cm)	Corrosivity Rating
> 20,000	non- corrosive
10,000 to 20,000	mildly corrosive
5,000 to 10,000	moderately corrosive
3,000 to 5,000	corrosive
1,000 to 3,000	highly corrosive
< 1,000	extremely corrosive

The Soil resistivity was found to be 2220 to 3330 ohm-cm for the samples analyzed making the soil corrosive to highly for buried steel. Consideration to increasing the specified strength and adding air entrainment into any reinforced concrete in contact with the soil should be given. Consideration should also be given to increasing the minimum concrete cover over reinforcing steel.

Based on the chemical test results, Type GU General use Hydraulic Cement may be used for this proposed development. Special protection is required for reinforcement steel within the concrete walls.

5.0 GEOTECHNICAL GUIDELINES AND RECCOMENDATIONS

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 boreholes 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 presented in this report include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or 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

Based on the proposed two levels of underground parking for the residential building it is considered that the finished floor for the lowest parking level will be about 6 metres below the existing ground surface. It is therefore expected that the underside of footing level would be about 7 metres below the existing ground surface.

The site is underlain by a layer of fill materials overlying firm to stiff silty clay, followed by compact to dense glacial till and limestone bedrock. Based on the results of the geotechnical assessment, the subsurface conditions at the estimated underside of footing level have very limited capacity to support additional loading and will have insufficient capacity to support the proposed building. As such it is considered that the foundation for the proposed building will have to be supported below the weak soils on the underlying bedrock. It is considered that large boulders could be encountered during the excavation to expose the bedrock.

The bedrock is considered suitable for the support of the proposed building on conventional spread footing foundations placed directly on the bedrock surface or on engineered fill placed on the native subgrade.

The allowable bearing pressure for any footings depends on the depth of the footings below original ground surface, the width of the footings, the height above the original ground surface of any landscape grade raise adjacent to the foundations and the thickness of the soils deposit beneath the footings.

5.3 Excavations

Based on the limited capacity of the subsurface soils below the proposed underground parkign level it is expected that the excavation for the proposed building will be extended to the bedrock surface about 9 to 9.5 metres below the existing ground surface. It is further understood that the excavation will extended close to the property lines at all sides of the site.

There is an existing building located south of the site. The footprint of the existing building will be separated from the proposed building basement / underground parking area by about 4.4 metres. It is estimated that the underside of footing elevation of the adjacent building will be at about 8 metres above the base of the excavation for the proposed residential development.

Where, due to space constraints, one horizontal to one vertical side slopes for the excavation cannot be provided, the excavation walls should be adequately shored. The shoring should be designed by a geotechnical engineer with experience in shoring design (shoring specialist) to support the lateral earth pressure ‘p’ plus the additional surcharge load of the adjacent building. The lateral earth pressure 'p' can be calculated using the following equation:

$$p = k(\gamma h + q) + \gamma_w H$$

Where p = the lateral earth pressure, at any depth, h, below the ground surface
k = earth pressure coefficient of 0.35
γ = unit weight of soil to be retained, estimated at 22 kN/m^3
h = the depth, in metres, at which pressure, p, is being computed
γ_w = unit weight of water (9.81 kN/m^3)

H = height of water level, in metres, from bottom of the excavation

q = the equivalent surcharge acting on the ground surface adjacent to the shoring including expected vehicular loads

The hydrostatic pressure, γ_w H, may be neglected where soldier piles and timber lagging are used as drainage is expected to occur between the lagging and thus no build-up of hydrostatic pressure is likely.

The surcharge load on the shoring from the adjacent building can be calculated by considering the footing of the building to be placing a vertical line load of 25 kN at 8 metres above the bottom of the excavation spaced horizontally from the location of the shoring. Alternatively the load of from the adjacent building can be equated to a equivalent surcharge q of 16 kPa at the ground surface adjacent the shoring.

5.4 Below Grade Basement and Parking Structure Foundation

As previously indicated, based on the expected loading from the proposed building and on the limited bearing capacity of the material immediately below the proposed foundation it is suggested that the building be founded either directly on the underlying bedrock, on sub-footings bearing on the bedrock.

Since the use of engineered fill would require that the excavation be sized to allow the spread of load beneath the footings, the use of engineered fill to raise the bedrock elevation to underside of footing level is not recommended. The engineered fill should extend horizontally from the edges of the footing a minimum distance of 0.3 m and then down and out at 1 horizontal to 1 vertical, or flatter. This would require an additional excavation width of about 2 to 3 metres.

For the proposed below grade basement and parking structure foundation, a maximum allowable bearing pressure of 1000 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 2500 kilopascals using ultimate limit states design, may be used for the design of conventional strip footings, a minimum of 0.6 metres in width, or pad footings founded directly on the underlying sound bedrock or on sub-footings founded directly on the underlying bedrock.
There are no grade raise restrictions associated with the above allowable bearing pressures

5.4.1 Sub-footings

It is considered that sub-footings could be constructed by excavating below the proposed footing level to the underlying bedrock in the areas of the proposed strip and pad footings only. The excavation for each sub-footing should be 0.3 metres wider on each side then the proposed strip or pad footing to be supported by the sub-footing. Upon approval of the subgrade, a low strength concrete (5 MPa) could be placed directly in the excavation to raise the bedrock surface to the underside of footing level.

5.4.2 Engineered Fill

Any fill required to raise the footings for the proposed residential 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 200 millimetre thick loose lifts to 100 percent of the standard Proctor maximum dry density. To allow the spread of load beneath the footings, the engineered fill should extend horizontally from the edges of the footing a minimum distance of 0.3 m and 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. Currently, OPSS documents allow recycled asphaltic concrete to be used in Granular A and Granular B Type II materials. Since the source of recycled material cannot be determined, it is suggested that any granular materials used below the founding level be composed of virgin materials only.

5.5 Groundwater

5.5.1 Excavation and Construction Dewatering

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

Groundwater was observed in boreholes BH1, BH2 and BH3 at about 3.0, 4.4 and 4.0 metres, respectively, below the ground surface at time of drilling on February 28, 2012.

Based on the results of the boreholes, it is expected that there will be groundwater inflow into the excavation for the proposed development. Since it is likely that groundwater will be encountered, it is expected that registration on the Environmental Activity Sector Registry (EASR) as per O.Reg. 63/16 will be required. Since the majority of the surficial soil thickness is comprised of silty clay it is expected that the pumping rates will be less than 400,000 L/day. However, if groundwater is encountered during excavation for the proposed services or building foundation requiring a pumping rate exceeding 400,000 Litres/day, a Permit to Take Water (PTTW) will be required.

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

Since the subsurface soil conditions below the level at which groundwater was encountered consist of stiff to very stiff silty clay overlying glacial till, it is expected that dewatering will not have a significant effect on the capacity of the subsurface soils to continue to support the adjacent structures. The silty clay will have sufficient residual strength to support the adjacent lightly loaded building in close proximity to the proposed development. The glacial till is not particularly sensitive to decreases in moisture content so a lowered groundwater will not decrease its capacity.

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

As previously indicated, the proposed foundation excavations will be carried out through fill, native silty clay and glacial till. There will be no bedrock excavation or removal. As such, there will be no excavation processes which could contribute to vibration which could potentially damage adjacent City of Ottawa Services.

5.6 Frost Protection Requirements

In general, all exterior foundation elements and those in any unheated parts of the proposed buildings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated foundation elements adjacent to surfaces, which are cleared of snow cover during winter months should be provided with a minimum 1.8 metres of earth cover for frost protection purposes.

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.

5.7 Below Grade and Parking Floor Slab

As stated above, it is expected that the native subsurface soils at the proposed second parking floor level will consist of glacial till with limited bearing capacity. For predictable performance of the proposed concrete floor slab all soft or loose and any deleterious material should be removed within the proposed building area. The exposed native sub-grade surface should then be inspected and approved by geotechnical personnel.

Engineered fill materials provided to support the concrete floor slab should consist of sand, or sand and gravel meeting the Ontario Provincial Standards Specifications (OPSS) for Granular B Type I or crushed stone meeting OPSS grading requirements for Granular B Type II. A minimum 150 millimetre thickness of crushed stone meeting OPSS Granular A should be provided immediately beneath the concrete floor slab. The engineered fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 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.

5.8 Building Basement and Below Grade Parking Structure Foundation Walls

The native soils at the site are considered to be frost susceptible. As such, to prevent possible foundation frost jacking, the backfill against unheated 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 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.

The parking structure and 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.

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.

Groundwater inflow from the native soils into the below grade parking structure or basement excavation during construction, if any should be handled by pumping from sumps within the excavations.

If the building basement and/or the parking structure will be unheated, the footings/grade beams, foundation walls and floor slabs will require protection from frost effects. Should the building basement and/or the parking structure not be heated we will be pleased to provide guidelines for suitable frost protection.

5.9 Seismic Design for the Proposed Residential Building

5.9.1 Seismic Site Classification

Based on the limited information from the boreholes, for seismic design purposes, in accordance with the 2006 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class D above the bedrock surface, and Site Class B when bearing on the bedrock surface.

Sentence 4.1.8.16.(4) of the 2006 OBC requires that the basement walls be designed to resist earthquake pressure from backfill or natural ground. The seismic pressure can be modelled by a static load distribution with a maximum at the ground surface level and a minimum at the base of the foundation. Using the Mononobe-Okabe method, the load distribution can be considered linear. The lateral seismic soil pressure, P_s, acting against the walls at any depth, h, calculated using the following equation.

$$\mathbf{P_s = 0.5\ \gamma\ h\ (1-k_v)\ K_{ae}}$$

Where: k_v = vertical acceleration coefficient and can be set at 0

γ = unit weight of soil to be retained, is estimated to be 22 kN/m^3

h = the height of the foundation wall above the founding level at the depth p_{ae} is being calculated.

Kae = 0.44 for any ground surface level or sloping away from the foundation

The total lateral seismic pressure P_{ae} acting against the foundation wall is equal to

$$\mathbf{P_{ae} = 0.5\ \gamma\ H^2\ (1-k_v)\ K_{ae}}$$

Where: H = Height of wall.

The lateral seismic soil pressure at the ground surface level for the foundation walls can be obtained from the following formula. $\mathbf{P_s = 0.5\ \gamma\ H\ (1-k_v)\ K_{ae}}$ The minimum lateral seismic soil pressure at the base is 0 kPa.

5.9.2 National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.266 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.

5.9.3 Potential for Soil Liquefaction

As indicated above the results of the boreholes indicate that the native deposits underlying the site consist of very stiff to firm silty clay and compact to dense glacial till underlain by bedrock. As these materials are not prone to liquefaction, it is considered that no damage to the proposed residential building should 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, silty clay and glacial till. The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupational Health and Safety Act. That is, open cut excavations with overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box. Groundwater seepage into the excavations, if any, should be handled by pumping from sumps in the excavation.

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 subexcavation of any existing fill or disturbed material encountered at subgrade level. Granular material meeting OPSS specifications for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as a 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.

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 subgrade 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 backfill is used, it should match the native materials exposed on the trench walls. Some of the native materials from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or recompaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future roadway areas. 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.

To minimize future settlement of the backfill and achieve an acceptable subgrade 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.

6.4 Access Roadway Pavements

In preparation for pavement construction at this site the surficial fill and any soft, wet or deleterious materials should be removed from the proposed access roadway area. The exposed subgrade should be inspected and approved by geotechnical personnel and any soft areas evident should be subexcavated and replaced with suitable earth borrow approved by the geotechnical engineer. The subgrade should be shaped and crowned to promote drainage of the roadway 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 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 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.

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

50 millimetres of hot mix asphaltic 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)

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

40 millimetres of hot mix asphaltic concrete (HL3) over
40 millimetres of hot mix asphaltic concrete (HL8) over
150 millimetres of OPSS Granular A base over
350 millimetres of OPSS Granular B, Type II subbase
(50 or 100 millimetre minus crushed stone)

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 and service trench backfill 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 and/or incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material.

7.0 CONSTRUCTION CONSIDERATIONS

It is suggested that the final design drawings for the project, including the proposed site grading plan, be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended and to re-evaluate the guidelines provided in the report with respect to the actual project plans. Items such as actual foundation wall/column loads, whether or not the basement or below grade parking structure is heated, etc could have significant impacts on foundation type, frost protection requirements, etc.

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

All foundation areas and any engineered fill areas for the proposed residential 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 foundation should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the site services and access roadways 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 roadway granular materials to ensure the materials meet the specifications from a compaction point of view.

The native sand and silty clay 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
Oct 15, 2021
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Building
CLIENT: Rohit Communities Ontario Inc.
LOCATION: 42 Northside Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 211099
DATE OF BORING: February 28, 2012
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH (×) / REM. SHEAR STRENGTH (○) Cu, kPa	DYNAMIC CONE PENETRATION TEST blows/300 mm	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface								
0	Asphaltic Concrete	0.00							
	Grey crushed stone (FILL)								
	Grey brown silty clay (FILL), trace of gravel	0.30							
1	Stiff to very stiff grey brown SILTY CLAY	0.84	1	SS	18				
2			2	SS	9				
			3	SS	4				
3	Firm to stiff grey SILTY CLAY	3.00	4	SS	WH				Water observed at approximately 3.0 metres below the existing ground surface on February 28,
4						142 kPa			
5			5	SS	WH				
						102 kPa			
6	Grey silty clay, some gravel and silt (GLACIAL TILL)	6.10	6	SS	4				
7			7	SS	7				
8			8	SS	7				
9			9	SS	102				
			10	SS	53				
	End of borehole	9.27							

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

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Building
CLIENT: Rohit Communities Ontario Inc.
LOCATION: 42 Northside Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 211099
DATE OF BORING: February 28, 2012
SHEET 1 of 1
DATUM:

DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	NOTES
Ground Surface					
Asphaltic Concrete	0.00				
Grey crushed stone (FILL)					
Grey brown silty clay (FILL), trace of gravel	0.30				
Stiff to very stiff grey brown SILTY CLAY, trace of silt layers	0.90	1	SS	18	
		2	SS	9	
		3	SS	4	
Firm to stiff grey SILTY CLAY	3.00	4	SS	WH	
		5	SS	WH	
					Water observed at approximately 4.4 metres below the existing ground surface on February 28, 2012.
		6	SS	4	
					142 kPa
Grey silty clay, some gravel and silt (GLACIAL TILL)	6.55	7	SS	7	
		8	SS	7	
		9	SS	102	
End of borehole	8.58				

Chart columns: DEPTH SCALE (meters) 0–9; STRATA PLOT; 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

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

RECORD OF BOREHOLE BH3

PROJECT: Proposed Residential Building
CLIENT: Rohit Communities Ontario Inc.
LOCATION: 42 Northside Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 211099
DATE OF BORING: February 28, 2012
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	UNDIST./REM. SHEAR STRENGTH Cu, kPa	DYNAMIC CONE PENETRATION TEST blows/300 mm	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface								
0	Asphaltic Concrete	0.00							Cuttings
	Grey crushed stone (FILL)								
	Grey brown silty clay (FILL), trace of gravel								
1	Stiff to very stiff grey brown SILTY CLAY, trace of silt layers	0.81	1	SS	15				
2			2	SS	11				
			3	SS	6				
3			4	SS	12				
4	Firm to stiff grey SILTY CLAY	4.01	5	SS	2				Water level
						46 kPa			
5						68 kPa			
			6	SS	WH				
6						108 kPa			
						91 kPa			
7	Grey silty clay, some gravel and silt (GLACIAL TILL)	7.09	7	SS	3				Seal
8			8	SS	9				Sand
			9	SS	29				
9	Grey silty sand, some gravel, cobbles, trace of clay (GLACIAL TILL)	9.24	10	SS	53				
10	REFUSAL ON BEDROCK								
	Limestone or dolomite BEDROCK (Advanced by coring)								
12	End of Borehole	11.89							

Water observed at approximately 4.0 metres below the existing ground surface on February 28,

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

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

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

PENETRATION RESISTANCE

Standard Penetration Resistance, N

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

Dynamic Penetration Resistance

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

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from 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

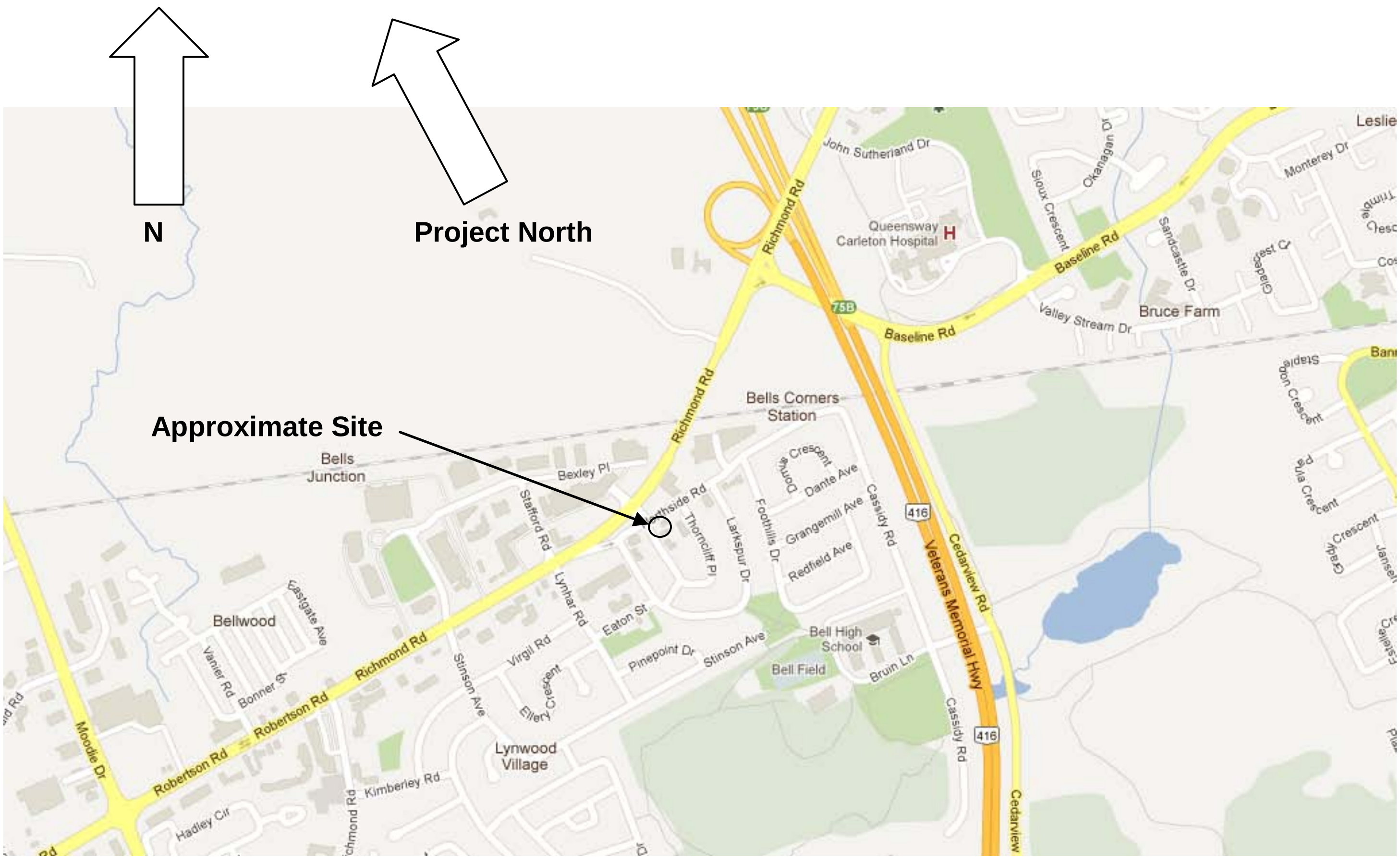
Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	over 100

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
ϕ' effective angle of friction
γ unit weight of soil
γ' unit weight of submerged soil
σ normal stress

KEY PLAN

FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 211099

Date October 2021

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

REFERENCE: PLAN SUPPLIED BY CITY OF OTTAWA EMAPS

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

REV.	NAME	DATE	DESCRIPTION

Kollaard Associates
Engineers
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KEMPTVILLE ONTARIO
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http://www.kollaard.ca
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CLIENT:
ROHIT COMMUNITIES ONTARIO INC.

PROJECT:
PROPOSED RESIDENTIAL
DEVELOPMENT
GEOTECHNICAL INVESTIGATION

LOCATION:
42 NORTHSIDE ROAD
BELL'S CORNER'S
OTTAWA, ONTARIO

DESIGNED BY: --	*DATE:* OCT. 15, 2021
DRAWN BY: DT	*SCALE:* N.T.S

KOLLAARD FILE NUMBER:
211099

Laboratory Test Results for Chemical Properties

Certificate of Analysis

Client: Kollaard Associates Inc.
210 Prescott St., Box 189
Kemptville, ON
K0G 1J0
Attention: Mr. Dean Tataryn
PO#:
COC# 145342

Report Number: 1204235
Date Submitted: 2012-03-12
Date Reported: 2012-03-19
Project: 120068

Group	Analyte	MRL	Units	Guideline	945321	945322
				Lab I.D.	945321	945322
				Sample Matrix	Soil	Soil
				Sampling Date	2012-02-28	2012-02-28
				Sample I.D.	BH2-SS3-7'6"-9'-6"	BH2-SS9-26'6"-28'-6"
Agri. - Soil	Electrical Conductivity	0.05	mS/cm		0.45	0.30
	pH	2.0			7.8	8.2
General Chemistry	Resistivity	1	ohm-cm		2220	3330
	SO4	0.01	%		0.07	0.04

Guideline = *** = Guideline Exceedence**

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.

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: 45.329N 75.816W **User File Reference:** 42 Northside Road 2021-10-12 18:15 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)	0.421	0.229	0.136	0.041
Sa (0.1)	0.495	0.280	0.173	0.056
Sa (0.2)	0.416	0.239	0.151	0.052
Sa (0.3)	0.317	0.184	0.117	0.041
Sa (0.5)	0.225	0.132	0.084	0.030
Sa (1.0)	0.113	0.067	0.043	0.015
Sa (2.0)	0.054	0.032	0.020	0.006
Sa (5.0)	0.014	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.266	0.153	0.094	0.030
PGV (m/s)	0.187	0.105	0.065	0.020

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