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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
65 ACACIA AVENUE
CITY OF OTTAWA, ONTARIO

Project # 170717

Submitted to:

Simon Saab and Jeffrey Abboud
1294 Kilborn Avenue
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January 12, 2018

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January 12, 2018 170717

Simon Saab and Jeffrey Abboud
1294 Kilborn Avenue
Ottawa, Ontario
K1H 6L3

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
65 ACACIA AVENUE
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed residential development to be located at 65 Acacia Avenue, City of Ottawa, Ontario.

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

BACKGROUND INFORMATION AND SITE GEOLOGY

Preliminary plans are being prepared to construct a residential development consisting of about a 72 square metre four storey building with a single storey of "below grade" parking at 65 Acacia Avenue in the City of Ottawa, Ontario (see Key Plan, Figure 1). The site has about 27 metres of frontage onto Acacia Avenue and the total site area is approximately 0.12 acres (0.05 hectares). For the purposes of this report, Acacia Avenue is considered to be oriented along a north south axis with the site located on the west side of the Avenue.

The legal description of the site was provided and is described as Lots 10 and 11, Plan 189537, City of Ottawa, Ontario (PIN 042250273 and 042250274). Currently, part of the site is occupied by a single family dwelling and the other part is vacant.

Preliminary plans indicate that the proposed building will likely be of ordinary wood frame construction with brick exterior gladding. Where not gladded with brick, the exterior walls will be covered with corrugated metal siding or fibre cement paneling. The building will be constructed with conventional concrete spread footing foundations and concrete slab-on-grade construction. The proposed building will have one level of below grade parking accessed from the southeast corner of the site. The existing ground surface slopes from an elevation of about 67.85 m at the southeast corner of the site to 71.30 m at the northwest corner of the site. As such the proposed "below grade" parking will have surface access at the southeast corner and be below grade along the north and west sides.

The proposed building will be serviced by municipal water and sanitary services. Surface drainage for the proposed building will be by means of swales and storm sewers. The site is located within an area of residential development.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by a thin layer of sand, glacial till and/or shallow bedrock. Bedrock geology maps indicate that the bedrock underlying the site may consist of limestone with possible shaly partings of the Ottawa Formation or dark grey almost black limestone of the Eastview Formation.

PROCEDURE

The field work for this investigation was carried out on October 3, 2017 at which time two test pits, numbered TP1 and TP2 were put down at the site using a rubber tire mounted backhoe supplied and operated by a local excavating contractor. The location of the proposed development was indicated to us on a site plan provided by the owners of the site.

The test pits were advanced to depths ranging from about 3.5 to 4.0 metres below the existing ground surface. The soil conditions observed in the test pits were classified based on visual and tactile examination of the materials in the walls and bottom of the test pits (ASTM D2488 - Standard

Practice for Description and Identification of Soils (Visual-Manual Procedure) and an assessment of the difficulty of excavation. The soils were classified using the Unified Soil Classification System. The groundwater conditions were observed in the open test pits at the time of the field work. The test pits were loosely backfilled with the excavated materials upon completion of the fieldwork.

One soil sample (TP1) was submitted for sieve analysis (ASTM C136). One sample of soil from TP2 was also delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack and soil corrosion on buried concrete and steel.

The field work was supervised throughout by a member of our engineering staff who located the test pits in the field, logged the test pits and cared for the samples obtained. A description of the subsurface conditions encountered at the test pits are given in the attached Table I, Record of Test Pits Sheet. The approximate locations of the test pits are shown on the attached Site Plan, Figure 2.

SUBSURFACE CONDITIONS

General

As previously indicated, a description of the subsurface conditions encountered at the test pits is provided in the attached Record of Test Pits following the text of this report. The test pit logs indicate the subsurface conditions at the specific test 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). 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 pit 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 test pits.

Fill

Fill materials consisting of a mixture of topsoil, grey brown sand and gravel with a trace of silt, cobbles and boulders and some glass debris were encountered from the ground surface at both of the test pits. The fill thickness ranged from about 0.7 to 0.8 metres below the existing ground surface. The fill materials were fully penetrated at the test pit locations.

Topsoil

Beneath the fill materials at TP2, a layer of topsoil ranging in thickness from about 0.15 to 0.8 metres in thickness was encountered at all of the test pits. The material was classified as topsoil based on the colour and the presence of organic materials. The identification of the topsoil layer is for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustainable plant growth.

Sand and Gravel

A deposit of red brown sand and gravel with a trace of cobbles was encountered beneath the fill materials and topsoil at both test pits. The deposit of sand and gravel extends to about 1.2 to 1.9 metres below the existing ground surface. The sand and gravel layer was fully penetrated at the test pit locations. Based on the difficulty of digging, the sand is considered to be compact.

One soil sample of the sand and gravel (TP1–1.2 to 1.9m) was submitted to Stantec for sieve analysis (ASTM C136). The results of the sieve analysis testing indicated that the sample consists of about 4.1 percent gravel with 74.1 percent sand and 21.7 percent silt and clay size particles. The moisture content of the soil sample was 3.8. The results are located in Attachment A.

Silty Sand

A deposit of grey silty sand with a trace of gravel, cobbles and boulders was encountered beneath the red brown sand layer. Test pit 1 was terminated within the silty sand deposit at a depth of about 3.5 metres below the existing ground surface. Test pit 2 was terminated on the surface of a large boulder or bedrock at a depth of about 4.0 metres below the existing ground surface. Based on the difficulty of digging, the silty sand is considered to be in compact state.

Bedrock

Test pit 2 was terminated on the surface of a large boulder or bedrock with practical refusal for advancement at a depth of about 4.0 metres below the existing ground surface. This corresponds to an elevation of about 66.3 metres. The surface of the bedrock was scraped using the bucket of the backhoe to determine the quality of the upper bedrock. The surface of the bedrock was observed to be smooth and un-fractured within the test pit location. The bedrock was observed to consist of limestone.

A review of borehole information provided on engineering plans obtained from the City of Ottawa for Acacia *Acacia Avenue Sewer, Road & Watermain Construction Beachwood Avenue to Maple Lane Rideau Ward Plan & Profile 2 Sta. 20+100 to Sta. 20+230 Dwg No. 5009-011 as built 01/17/05,* indicates that bedrock is encountered below Acacia Avenue at an elevation ranging from about 64 metres at the south end of the site to about 66 metres at the north end of the site.

Groundwater

Both of the test pits were dry at the time of the field work. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

Corrosivity on Reinforcement and Sulphate Attack on Portland Cement

One sample of soil from TP2 was delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack and soil corrosion on buried concrete and steel.

The results of the laboratory testing of a soil sample for submitted for chemistry testing related to corrosivity is summarized in the following table.

Item	Threshold of Concern	Test Result	Comment
Chlorides (Cl)	Cl > 0.04 %	< 0.002	Negligible concern
pH	5.0 < pH	8.9	Basic Negligible concern
Resistivity	R < 1500 ohm-cm	20000	Mildly to Non-Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	< 0.01	Negligible concern

Based on the chemical test results, Type GU General use Hydraulic Cement may be used for this proposed development. No special protection is required for reinforcement steel within the concrete walls. The laboratory results are presented at the end of this report.

No special protection is expected to be required for reinforcement steel within the concrete walls.

PROPOSED RESIDENTIAL DEVELOPMENT BUILDING FOUNDATIONS

General

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

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

Foundation Excavation

The excavations for the building foundation will be carried out through fill, sand and gravel, silty sand and possibly 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.

In accordance with O.Reg 213/91, s. 226, the upper soils at this site can be considered to be Type 2 soil. As such, open cut excavations which result in confined spaces within the upper soil deposits at this site above the ground water level should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter to within 1.2 metres of the bottom of the excavation. There should be no excavated material stockpiled with a distance from the excavation equal to the depth of excavation. Alternatively a written opinion from a professional engineer can be made with respect to the stability of the excavation side slopes during excavation prior to the excavation being entered by any persons.

Foundations for Proposed Residential Building

With the exception of the fill materials and topsoil, the subsurface conditions encountered at the test pits advanced during the investigation are suitable for the support of the proposed residential building on conventional spread footing foundations.

The allowable bearing pressure for any footings depends on the depth of the footings below original ground surface, the width of the footings, and the height above the original ground surface of any landscape grade raise adjacent to the dwelling foundation.

As previously indicated, the subsurface conditions at the site encountered at the test pits advanced during the investigation consisted of fill materials and topsoil followed by sand and gravel and/or silty sand then bedrock or large boulders. With the exception of the fill materials and topsoil, the subsurface conditions are suitable for the support of the proposed residential building on conventional spread footing foundations placed on a native subgrade or engineered fill placed on the native sand and gravel or silty sand.

Conventional Spread Footing Foundations

For the proposed residential development, a maximum allowable bearing pressure of 100 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 150 kilopascals using ultimate limit states design, may be used for the design of conventional strip footings or pad footings, a minimum of 0.6 metres in width, founded on the compact sand and gravel and/or silty sand or on a suitably constructed engineered pad placed on the sand and gravel and/or silty sand.

The above allowable bearing pressure is subject to a maximum grade raise of 2.0 metres above the original ground surface within 5 metres of the proposed building footprint and to maximum strip and pad footing widths of 1.5 metres.

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

For predictable performance of the proposed foundation, all existing fill, topsoil, and/or any debris or deleterious materials and any disturbed subgrade material should be removed from within the proposed foundation areas. The exposed subgrade should consist of sand and gravel or silty sand. The subgrade surface should then be inspected and approved by geotechnical personnel.

Should the complete removal of all fill materials and topsoil and any otherwise deleterious material result in a subgrade below the proposed founding level, 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 300 millimetre thick loose lifts to 98 percent of the standard Proctor maximum dry density. It is considered that the engineered fill should be compacted using dynamic compaction with a large diameter vibratory steel drum roller or diesel plate compactor. If a diesel plate compactor is used, the lift thickness may need to be restricted to less than 300 mm to achieve proper compaction. Compaction should be verified by a suitable field compaction test method.

To allow the spread of load beneath the footings, the engineered fill should extend horizontally from the edges of the footing a minimum distance of 1.0 metre 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.

The first lift of engineered fill material should have a thickness of 300 millimetres in order to protect the subgrade during compaction. It is considered that the placement of a geotextile fabric between the engineered fill and the subgrade is not necessary where granular materials meeting the grading requirements for OPSS Granular A or Granular B Type II are placed on the subgrade above the normal ground water level. If trucks are used to place the engineered fill on the subgrade, a thickened path of 0.6 metres should be used to protect the subgrade from the truck traffic.

The native silty sand deposits at this site will be sensitive to disturbance from construction operations and from rainwater or snowmelt, and frost. In order to minimize disturbance, construction traffic operating directly on the subgrade should be kept to an absolute minimum and the subgrade should be protected from below freezing temperatures.

Frost Protection Requirements for Spread Footing Foundations

In general, all exterior foundation elements and those in any unheated parts of the proposed residential 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. 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, if required. Alternatively, the foundation footing can be stepped downward at the southeast corner or entrance to the below grade parking to ensure adequate frost cover over the footing. The footing should be stepped downward at maximum 0.6 metre increments separated by 1.2 metres.

All shallow exterior footings if present and those in any unheated parts of the structures should be provided with at least 1.5 metres of earth cover for frost protection purposes. A conventional, perforated perimeter drain, with a 150 millimetre surround of 20 millimetre minus crushed stone, should be provided at founding level and should lead by gravity flow to a sump and/or to a storm sewer. The proposed basement should also be provided with under floor drains consisting of perforated pipe with a surround of 20 millimetre minus crushed stone to reduce the potential for buildup of hydrostatic pressure below the basement floor.

Building Basement Foundation Walls and Drainage

A conventional, perforated perimeter drain should be provided at founding level, leading by gravity flow to a sump or storm sewer. The drain should be installed at footing level and provided with a 150 millimetre thick surround of 20 millimetre minus crushed stone. The drain should be provided with a backflow preventer.

It is considered that in view of the limited groundwater conditions observed at the test holes, the above perimeter drainage system should adequately handle any groundwater seepage to the basements.

To prevent possible foundation frost jacking due to frost adhesion, any 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. The native soils at this site are considered to be slightly frost susceptible, as such, it is recommended that imported granular material be used to backfill the foundation. Alternatively, foundations could be backfilled on the exterior 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. 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.

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 at least 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 basement excavations during construction, if any should be handled by pumping from sumps within the excavations.

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.

Building Structure Floor Slab

As stated above, it is expected that the proposed building will be founded on native sand and gravel or silty sand or on an engineered pad placed on the sand and gravel/silty sand. For predictable performance of the proposed concrete floor slab all existing fill material, topsoil and any otherwise deleterious material should be removed from below the proposed floor slab areas. The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any soft or disturbed areas evident should be subexcavated and replaced with suitable engineered fill.

The fill materials beneath the proposed concrete floor slab should consist of a minimum of 150 millimetre thickness of crushed stone meeting OPSS Granular A immediately beneath the concrete floor slab followed by sand, or sand and gravel meeting the OPSS for Granular B Type I, or crushed stone meeting OPSS grading requirements for Granular B Type II, or other material approved by the Geotechnical Engineer. The fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density.

The 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 twenty-five times the slab thickness to a maximum of 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.

Seismic Design for the Proposed Residential Buildings

Based on the limited information from the test pits, 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 D.

National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.282g with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The results of the test are attached following the text of this report.

Potential for Soil Liquefaction

As previously indicated, it is expected that bedrock will be encountered at an elevation of about 64 metres to 66 metres or about 0.5 to 2.5 metres below the proposed underside of footing elevation. No groundwater was encountered within the test pits put down at the site. The subsurface conditions between the proposed underside of footing level and the underlying bedrock consist of sand and gravel or silty sand in a compact state of packing. Given the relatively thin layer of sand and gravel or silty between the underside of footing and the bedrock, the compact state of packing and the absence of groundwater it is considered that the underlying subgrade conditions are not liquefiable. It is considered that no damage to the proposed building should occur due to liquefaction of the native subgrade under seismic conditions.

EXISTING RETAINING WALL

There is an existing retaining wall which begins on the adjacent property at about the north east corner of the site, extends across the north west corner of the site and ends about 4.5 metres west of the site. The retaining wall varies in height from about 0.6 metres to 1.5 metres with the retained soil being on the subject site. The existing retaining wall has been constructed with segmental blocks is located about 2 metres from the proposed building.

Foundation Excavation

Excavation for the proposed foundation will compromise the lateral support for the retaining wall unless the full length of the retaining wall adjacent the excavation is shored. The shoring must be designed by an engineering firm with experience in shoring design.

The shoring should be designed to support the lateral earth pressure 'p' 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, $\gamma_w H$, 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.

Alternatively, the existing retaining wall and retained soil can be removed during the initial stages of the excavation. This will lower the height of the excavation by 1.5 metres and should allow adequate horizontal distance from the proposed building to slope the excavation side slopes in order to maintain the stability of the side slopes without shoring.

SITE SERVICES

Excavation

The excavations for the site services will be carried out through fill, sand and gravel, silty sand and possibly 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.

In accordance with O.Reg 213/91, s. 226, the upper soils at this site can be considered to be Type 2 soil. As such, open cut excavations within the upper soil deposits at this site above the ground water level should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter to within 1.2 metres of the bottom of the excavation. 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. No material should be stored adjacent the top of excavation.

It is expected that boulders may be encountered during excavating for site services. It is considered that the boulder removal, if required, can most likely be carried out by hoe ramming and heavy excavating equipment. As extensive hoe ramming can also result in ground vibrations, it is considered a pre-construction survey of nearby structures be completed if hoe ramming is used as well.

Pipe Bedding and Cover Materials

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

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

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

Trench Backfill

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

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

Where native backfill is used, it should match the native materials exposed on the trench walls. Some of the native materials from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or recompaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future roadway areas. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I. If the native material is not suitable for backfill, imported granular material may have to be used. If imported granular materials are used, suitable frost tapers should be used OPSD 802.013.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the parking areas, sidewalks, etc., the trench should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced to 90 percent where the trench backfill is not located or in close proximity to existing or future roadways, driveways, sidewalks, or any other type of permanent structure.

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 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)

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 sub-base material. The adequacy of the design of the pavement thickness should be assessed by the geotechnical personnel at the time of construction.

CONSTRUCTION CONSIDERATIONS

It is suggested that the final design drawings for the project, including the proposed site grading plan, be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

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 footing 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 foundations should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the site services and access roadway should be inspected and approved by geotechnical personnel. In situ density testing should be carried out on the service pipe bedding and backfill and the pavement granular materials to ensure the materials meet the specifications from a compaction point of view.

The native silty sand deposits at this site will be sensitive to disturbance from construction operations, from rainwater or snow melt, and frost. In order to minimize disturbance, construction traffic operating directly on the subgrade should be kept to an absolute minimum and the subgrade should be protected from below freezing temperatures.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact our office.

Regards,
Kollaard Associates Inc.

LICENSED PROFESSIONAL ENGINEER
Jan.12, 2018
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve deWit, P.Eng.

Attachments: Table I - Record of Test Pits
List of Abbreviations
Key Plan - Figure 1
Site Plan - Figure 2
Laboratory Test Results for Chemical
Laboratory Test Results for Physical Properties - Stantec Laboratory Test Results for Soils
Attachment A - National Building Code Seismic Hazard Calculation

TABLE I

RECORD OF TEST PITS
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
65 ACACIA AVENUE
CITY OF OTTAWA, ONTARIO

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP1 Elevation	0.00 - 0.30	Topsoil (FILL)
~ 69.78	0.30 – 0.70	Grey brown sand and gravel, trace of cobbles and boulders, and glass (FILL)
	0.70 - 1.30	Compact red brown SAND and GRAVEL
	1.30 - 3.50	Compact grey SILTY SAND, trace gravel, cobbles and boulders
	3.50	End of test pit

Test pit dry, October 3, 2017.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP2 Elevation	0.00 - 0.15	Topsoil (FILL)
~ 71.54	0.15 – 0.80	Grey brown sand and gravel, trace silt (FILL)
	0.80 - 0.90	TOPSOIL
	0.90 - 1.90	Compact red brown SAND and GRAVEL, trace cobbles
	1.90 - 4.00	Compact grey SILTY SAND, trace gravel
	4.00	Practical refusal on large boulder or bedrock

Test pit dry, October 3, 2017.

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

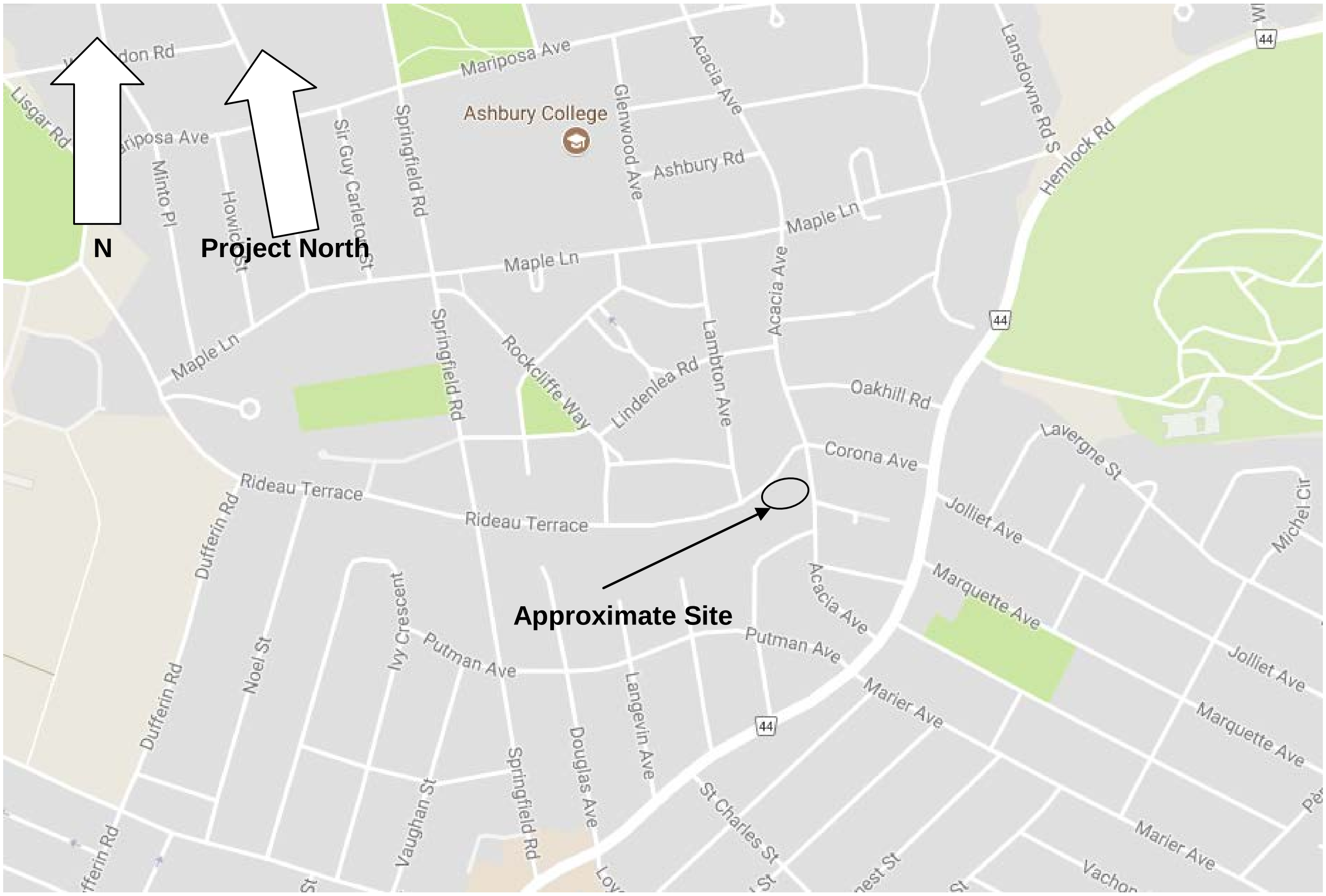
Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

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. 170717

Date January 2018

DRAWING NUMBER:

SITE PLAN, FIGURE 2

LEGEND:

TP1 APPROXIMATE TEST PIT LOCATION

REFERENCE: PLAN SUPPLIED BY CITY OF OTTAWA EMAPS.

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

REV.	NAME	DATE	DESCRIPTION

Kollaard Associates
Engineers

PO, BOX 189, 210 PRESCOTT ST
KEMPTVILLE ONTARIO
K0G 1J0 FAX (613) 258–0475
http://www.kollaard.ca

(613) 860–0923
info@kollaard.ca

CLIENT:
SIMON SAAB AND JEFFREY ABBOUD

PROJECT:
GEOTECHNICAL INVESTIGATION FOR RESIDENTIAL BUILDING

LOCATION:
65 ACACIA AVENUE
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	*DATE:*
––	OCT. 6, 2017
DRAWN BY: DT	*SCALE:* N.T.S

KOLLAARD FILE NUMBER:
170717

Laboratory Test Results for Chemical Properties

Certificate of Analysis

Environment Testing

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

Report Number:	1719269
Date Submitted:	2017-10-04
Date Reported:	2017-10-11
Project:	170717
COC #:	192260

Group	Analyte	MRL	Units	Guideline	1324639
				Lab I.D.	1324639
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2017-10-03
				Sample I.D.	TP2 GS3
Agri. - Soil	pH	2.0			8.9
	SO4	0.01	%		<0.01
General Chemistry	Cl	0.002	%		<0.002
	Electrical Conductivity	0.05	mS/cm		0.05
	Resistivity	1	ohm-cm		20000

Guideline = *** = Guideline Exceedence**

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

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

Laboratory Test Results for Physical Properties

Stantec 2781 Lancaster Road
Ottawa ON, K1B 1A7

Sieve Analysis
LS 602
ASTM C136

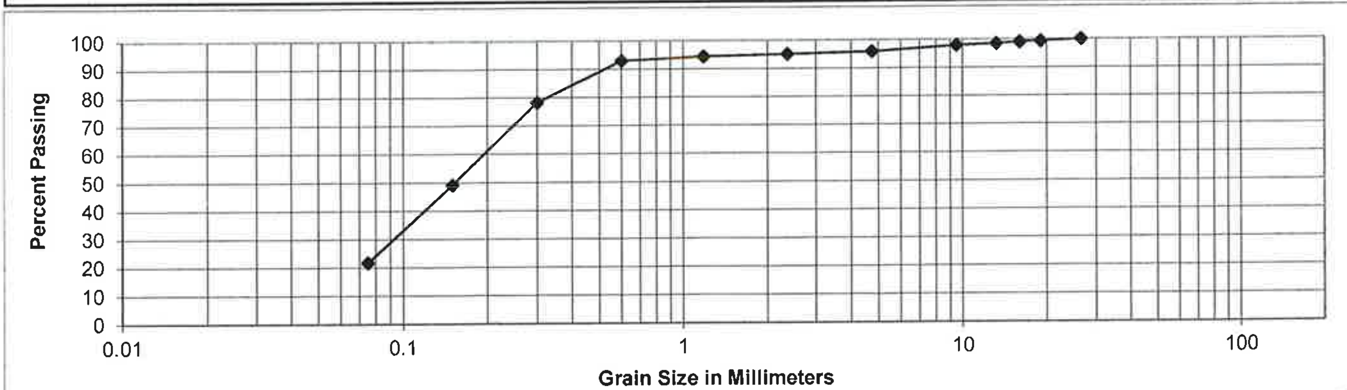
Client:	**Kollaard Associates Engineers File #170717**	Project Number:	**122410003**
Project:	**65 Acacia Avenue**		
Material Type:	**Soils / Aggregates:**		
Proposed Use:	**Fill/Granulars**		
Source:	**TP-1**		
Sample Number:	**GS-2**		
Sampled Depth:	**At Source**		
Sampled By:	**Kollaard Associates Engineers**	Tested By:	**Denis Rodriguez**
Date Sampled:	**October 3, 2017**	Date Tested:	**October 6, 2017**

Sieve Test Data		Wash Test Data		
Sample Weight Before Sieve, (g):	2010.2	Sample Weight Before Wash, (g):	255.0	Corrected
Sample Weight After Sieve, (g):	2009.7	Sample Weight After Wash, (g):	210.4	
Percent Loss In Sieve, (%):	0.02	Percent Passing No. 200, (%):	17.5	16.8

Sieve Analysis

Sieve No.	Size of Opening Inches	Size of Opening mm	Weight Retained g	Cumulative Weight Retained g	Percent Passing %	No Envelope Minimum	No Envelope Maximum
	6	150					
	4	106					
	3	76.2					
	2	53.0					
	1.5	37.5					
	1	26.5	0.0	0.0	100.0		
	3/4	19.0	14.7	14.7	99.3		
	5/8	16.0	5.6	20.3	99.0		
	1/2	13.2	10.3	30.6	98.5		
	3/8	9.5	11.8	42.4	97.9		
+4	0.187	4.75	40.5	82.9	95.9		
		- 4.75	1926.8	2009.7			
8	0.0937	2.36		2.0	95.1		
16	0.0469	1.18		4.0	94.4		
30	0.234	0.600		7.7	93.0		
50	0.0117	0.300		47.0	78.2		
100	0.0059	0.150		124.6	49.0		
200	0.0029	0.075		197.2	21.7		
		Pan		209.9			

Classification of Sample: % Gravel: 4.1 % Sand: 74.1 % Silt & Clay: 21.7



Remarks:

Reviewed By: Brian Prevost **Date:** october 10/2017

V:\01216\active\laboratory_standing_offers\2017 Laboratory Standing Offers\122410003 Kollaard Associates Engineers\October 3, Sieve, & MC, Kollaard #170717\Sieve Analysis.xlsx

Stantec

2781 Lancaster Rd. Suite 101
Ottawa ON, K1B 1A7

Determination of Moisture Content of Soil

LS 701 or CSA A23.2-11A
ASTM D2216

Project: Kollaard #170717
Project No.: 122410003

Date Tested: October 4, 2017
Tested By: Brian Prevost

Moisture Content Worksheet

Borehole / Test Pit No.	TP-1						
Sample	GS-2						
Tare No.							
Weight Sample (Wet+Tare) (g)	2388						
Weight Sample (Dry+Tare) (g)	2311.7						
Weight of Water (g)	76.3						
Tare Container	300.5						
Weight Dry Sample (g)	2011.2						
Moisture Content (%)	**3.8**						
Comments							
Borehole / Test Pit No.							
Sample							
Tare No.							
Weight Sample (Wet+Tare) (g)							
Weight Sample (Dry+Tare) (g)							
Weight of Water (g)							
Tare Container							
Weight Dry Sample (g)							
Moisture Content (%)							
Comments							
Borehole / Test Pit No.							
Sample							
Tare No.							
Weight Sample (Wet+Tare) (g)							
Weight Sample (Dry+Tare) (g)							
Weight of Water (g)							
Tare Container							
Weight Dry Sample (g)							
Moisture Content (%)							
Comments							
Borehole / Test Pit No.							
Sample							
Tare No.							
Weight Sample (Wet+Tare) (g)							
Weight Sample (Dry+Tare) (g)							
Weight of Water (g)							
Tare Container							
Weight Dry Sample (g)							
Moisture Content (%)							
Comments							

Reviewed By: ____________________ Date: ____________

ATTACHMENT A

NATIONAL BUILDING CODE
SEISMIC HAZARD CALCULATIONS

2015 National Building Code Seismic Hazard Calculation

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

October 05, 2017

Site: 45.444 N, 75.6724 W User File Reference: 65 Acacia Avenue

Requested by: , Kollaard Associates Inc.

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

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.449	0.525	**0.440**	0.335	**0.238**	**0.118**	**0.056**	**0.015**	**0.0054**	**0.282**	**0.197**

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

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.045	0.150	0.249
Sa(0.1)	0.061	0.188	0.301
Sa(0.2)	0.055	0.162	0.256
Sa(0.3)	0.044	0.125	0.196
Sa(0.5)	0.031	0.088	0.139
Sa(1.0)	0.015	0.045	0.070
Sa(2.0)	0.0061	0.021	0.033
Sa(5.0)	0.0012	0.0047	0.0081
Sa(10.0)	0.0006	0.0019	0.0032
PGA	0.033	0.102	0.164
PGV	0.021	0.068	0.111

References

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

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

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

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

Aussi disponible en français