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REPORT ON

GEOTECHNICAL INVESTIGATION PROPOSED RESIDENTIAL APARTMENT BUILDING 841, 845 AND 855(A) GRENON AVENUE CITY OF OTTAWA, ONTARIO

Kollaard Associates Project # 180966

Submitted to:

Building Investments and Developments
205 - 1320 Carling Avenue
Ottawa, Ontario
K1Z 7K8

City of Ottawa SPC Application File # D07-12-19-0018

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August 7, 2019 180966

Building Investments and Developments
205 - 1320 Carling Avenue
Ottawa, Ontario
K1Z 7K8

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL APARTMENT BUILDING
841, 845 AND 855(A) GRENON AVENUE
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed residential building. 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.

BACKGROUND INFORMATION AND SITE GEOLOGY

Plans are being prepared to construct a multi-unit residential building at 841, 845 and 855(A) Grenon Avenue in the City of Ottawa, Ontario (see Key Plan, Figure 1). In total, the site consists of about 0.14 hectares (0.35 acres) of land located on the east side of Grenon Avenue, about 160 metres south of the intersection of Carling Avenue and Grenon Avenue, City of Ottawa, Ontario. The site is currently occupied by a single family dwelling and vacant lands. The remaining areas not occupied by the dwelling are mostly scattered trees and open grassed surfaced yard space.

Preliminary plans are being prepared to construct a residential development consisting of about a 660 square metre four storey building. The upper three floors will contain 27 residential units. The ground floor and basement will contain guest parking at the basement level and will utilize three level car elevators (car stackers) for tenant parking. It is understood that the proposed building will be of wood or steel framed construction with conventional spread footing foundations. The proposed building will be serviced by municipal water and sanitary services.

For the purposes of this report, Grenon Avenue is considered to be oriented along a north south axis with the site located on the east side of Grenon Avenue. Surrounding land use is residential development. The site is bordered on the north and south by residential development, on the east by parkland followed by residential development and on the west by Grenon Avenue followed by residential development.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by older alluvial materials or possibly glacial till. Bedrock geology maps indicate that the bedrock underlying the site may consist of limestone of the Ottawa Formation.

It is understood that a former sand pit was located east of the site. A 1958 air photograph indicates the sand pit was in operation during that time period.

Based on a review of overburden thickness mapping for the site area, the overburden thickness above bedrock is estimated to be about 8 metres.

PROCEDURE

The field work for this investigation was carried out on December 18, 2018 at which time two boreholes, numbered BH1 and BH2 were put down at the site using a track mounted drill rig equipped with a hollow stem auger owned and operated by Capital Cutting & Coring Ltd of Ottawa, Ontario.

Sampling of the overburden materials encountered at the borehole location was carried out at regular 0.75 metre depth intervals using a 50 millimetre diameter drive open conventional split spoon sampler in conjunction with standard penetration testing (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils). In situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil) was not carried out as no cohesive materials were encountered at any of the boreholes.

The subsurface soil conditions at the boreholes were identified based on visual examination of the samples recovered (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), and standard penetration tests (ASTM D-1586) as well as laboratory test results on select samples. Groundwater conditions at the borehole was noted at the time of drilling. One standpipe was installed at BH1 for subsequent ground water level monitoring. The other borehole was loosely backfilled with the auger cuttings upon completion of drilling.

Two soil samples (BH1 and BH2) were submitted for sieve analysis (ASTM C136) testing. The soils were classified using the Unified Soil Classification System. A sample of soil obtained from BH2 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 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 the boreholes are given in the attached Record of Borehole Sheets. The results of the laboratory testing of the soil samples are presented in the Laboratory Test Results section and Attachment A following the text in this report. The approximate location of the boreholes are shown on the attached Site Plan, Figure 2.

SUBSURFACE CONDITIONS

General

As previously indicated, a description of the subsurface conditions encountered at the boreholes is provided in the attached Record of Borehole Sheets following the text of this report. The borehole logs indicate the subsurface conditions at the specific drill location only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than borehole 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) with select samples being classified by laboratory testing in accordance with ASTM 2487. Classification and identification of soil involves judgement and Kollaard Associates Inc. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

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

The ground surface elevation at the borehole locations were determined, in the field, relative to a geodetic benchmark provided by Farley, Smith & Denis Surveying Ltd. The geodetic benchmark is described as two nails on utility pole located on the west side of Grenon Avenue, directly opposite of the northwest corner of the site. The elevation of the top of spindle is reference as 72.41 metres.

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

Topsoil

From the surface at borehole BH1, a thin layer of topsoil measuring about 0.2 metres was encountered. 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. The topsoil layer was fully penetrated.

Fill

From the surface at borehole BH2, fill materials were encountered. The fill materials consisted of about a 200 millimetre thickness of grey brown silty clay with a trace of gravel and organics followed by about 0.9 metres of yellow brown silty sand. The fill materials were fully penetrated at the borehole location.

Sand

A deposit of grey brown to grey fine to medium sand with a trace of gravel and silt becoming grey fine to coarse sand with a trace of silt and gravel was encountered beneath the topsoil and the fill materials. The Sand was encountered at an elevation of about 73.8 metres in BH1 and about 73.4 metres in BH2

The results of the standard penetration testing carried out in the sand material, ranged from 10 to 34 blows per 0.3 metres with an average value of 22 blows per 0.3 metres, indicating a compact to state of packing.

BH1 was terminated within the grey fine to coarse sand at a depth of about 8.2 metres (elevation of 65.8) below the existing ground surface. BH2 was advanced beyond a depth of 8.2 metres below the existing ground surface by means of dynamic cone penetration testing. The dynamic cone penetration test gave values of greater than 20, 71 per 0.3 metres and 120 blows per 0.2 metres before refusal to cone penetration was encountered with no advancement on the surface of a large boulder or bedrock at a depth of about 9.0 metres (elevation of 65.4 metres) below the existing ground surface.

Two soil samples of sand (BH1 - SS7 - 4.57 to 5.18 metres and BH2 - SS6 - 3.81 - 4.41) were submitted to Stantec for particle size analysis (ASTM D2216). The results of the particle size analysis testing indicated that the samples consists of about 0 to 2.9 percent gravel, 82.8 to 89.8 percent sand and about 10.2 to 14.3 percent silt and clay. The tested soil samples are classified as poorly-graded sands, gravelly sands, little or no fines (SP) in accordance with the Unified Soil Classification System.

The results are located in Attachment A.

Groundwater

Groundwater seepage was observed within boreholes BH1 and BH2 at the time of drilling on December 18, 2018 at about 6.3 metres below the existing ground surface (67.8 m and 68.2 m in BH1 and BH2 respectively). Groundwater was measured in a stand pipe installed within BH1 at a depth of about 6.9 metres below the existing ground surface (elevation of 67.2 m) on December 20, 2018.

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

Corrosivity on Reinforcement and Sulphate Attack on Portland Cement

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

Item	Threshold of Concern	Test Result	Comment
Chlorides (Cl)	Cl > 0.04 %	0.005	Negligible
pH	5.0 < pH	7.40	Basic Negligible concern
Resistivity	R < 20,000 ohm-cm	24400	Non-Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	0.002	Negligible concern

The results were compared with Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and posses a "negligible" risk for sulphate attack on concrete materials and accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements.

The pH value for the soil sample was reported to be at 7.40, indicating a durable condition against corrosion. This value was evaluated using Table 2 of Building Research Establishment (BRE) Digest 362 (July 1991).The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids.

The chloride content of the sample was also compared with the threshold level and present negligible concrete corrosion potential.

The results of the laboratory testing of a soil sample for resistivity and pH indicates the soil sample tested has an underground corrosion rate of about 0.2 loss-oz./ft^2/yr (24400 ohm-cm). Based on the findings of Fischer and Bue (1981) underground corrosion rates (loss-oz./ft^2/yr) of 0.30 and less are considered nonaggresive, from 0.30 to 0.75 the rate is considered slightly aggressive, from 0.75 to 2.0 the rate is considered aggressive and 2.0 and greater the rate is considered very aggressive. Accordingly, the above mentioned soil sample is considered to be non-aggressive to reinforcement steel within below grade concrete walls. Minimum cover requirements as specified by the American Concrete Institute for concrete construction should be followed.

GEOTECHNICAL RECOMMENDATIONS AND DESIGN GUIDELINES

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 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 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 finished floor level at the bottom of the car elevator pit will be at an elevation of about 67.0 metres. The expected underside of footing for the car elevator pit foundation is at about 66.2 metres. The finished floor level for the remainder of the basement will be at about 70.8 m with an expected underside of footing elevation of 70.0 metres. It is expected that the excavation will extend about 1.0 metres beyond the basement foot print on all sides of the proposed excavation.

There is an existing row house development located south of the site. The foot print of the existing building will be separated from the proposed building basement / underground parking area by about 7.2 metres along the south property line. Based on the row house construction, it is estimated that the underside of footing elevation of the row house buildings will be at about 74 metres or at about 7.8 metres above the base of the excavation for the proposed residential development.

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.

Since the excavation is to extend 1 metres beyond the foot print of the proposed foundation, there is likely not sufficient space between the property lines and the excavation to provide adequate side slopes. The excavation walls will likely require shoring on all sides. 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 row house 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 20 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.

The surcharge load on the shoring from the adjacent row house building can be calculated by considering the footing of the building to be placing a vertical line load of 15 kN at an elevation of 74 metres 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 8 kPa at the ground surface.

Ground Water in 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.

The proposed basement parking area floor will have a finished surface of 70.80 metres and will be at least 2 metres above the ground water level. The footings for the portion of the foundation adjacent the basement parking will also be above the ground water level. The floor slab for the car stacker pit will be at an elevation of 67.02 metres with the underside of footing extended as much as 1.0 metre below the floor. Based on these proposed elevations, the car stacker pit foundation will extend below the ground water level.

Since the subsurface conditions at the groundwater level consist of medium to course sand, it is expected that there could be significant inflow into the excavation during construction of the foundation for the car stacker pit. There is potential that a permit to take water PTTW may be required in accordance with MECP guidelines where construction dewatering may result in flows of more than 400,000 Litres/day. A minimum registration on the Environmental Activity Sector Registry (EASR) as per O.Reg. 63/16 will be required. It is recommended that the shoring techniques used are designed to minimize the groundwater flows into the lower portion of the excavation.

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

Soils encountered at the site consist of poorly graded sand which is not sensitive to changing moisture conditions. As such dewater of the foundation or site services excavations, if required, will not have a detrimental impact on the adjacent structures.

Below Grade Basement and Parking Structure Foundation

Based on the expected loading from the proposed building and on the potential bearing capacity of the sand it is suggested that the building be founded either directly on the underlying sand or on engineered fill placed on the underlying sand.

For the proposed below grade basement and parking structure foundation, a maximum allowable bearing pressure of 150 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 250 kilopascals using ultimate limit states design, may be used for the design of conventional strip footings with a maximum width of 2.0 metres. A maximum allowable bearing pressure of 200 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 300 kilopascals using ultimate limit states design, may be used for the design of conventional pad footings with a maximum length and width of 3.0 metres.

The above allowable bearing pressures assume the footings area founded on an approved subgrade surface consisting of undisturbed native sand or on a suitably constructed engineered pad placed on the native sand above the ground water level.

The above allowable bearing pressures assume a maximum finished grade elevation at the site of 76.2 metres.

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

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

Grade Raise Restrictions.

As indicated above, the allowable bearing pressures provided above assume a maximum finished grade elevation at the site of 76.2 metres. The lowest existing grade at the site is located at the northwest corner of the site and is 73.1 metres. The highest existing grade at the site is along the south side of the site and is about 76.15 metres. The proposed finished grades range from 76.2 metres to 73.2 metres. The expected grade raise will vary between 0.05 and 1.0 metres. The maximum allowable grade raise at the site is 2.0 metres.

Foundation Drainage

A conventional, perforated perimeter drain, with a 150 millimetre surround of 20 millimetre minus crushed stone, should be provided at the founding level for the basement floor parking area (approximate elevation of 70 metres) and should lead by gravity flow to a sump. The sump should be equipped with a backup pump and generator. 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. The under floor drains should be placed beginning at the inside edge of the foundation wall and should be spaced a maximum of 5 metres apart. The under floor drain should also be directed to the sump. The sump discharge should be equipped with a backup flow protector.

The bottom portion of the car stacker pit will be below the ground water level. Since the subsurface conditions at the founding level for the car stacker pit consist of medium to coarse sand, it is not reasonable to expect that a foundation drainage system will be able to lower the groundwater level sufficiently to keep the foundation dry. As such, the car stacker pit should be constructed in a waterproof manner and should be provided with an exterior waterproof linear for secondary protection. The car stacker pit should be structurally designed to with stand an upwards buoyancy force of 30 kPa. Design of the liner and waterproofing should be completed by a specialist in building membranes and related building waterproofing design.

Below Grade and Parking Floor Slab

As stated above, it is expected that the proposed building will be founded on engineered fill or on native sand. 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 98 percent of the standard Proctor maximum dry density. Alternatively clear crushed 20 mm minus stone could be used immediately below the concrete floor slab provided the clear stone is well compacted prior to concrete placement.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage and expansion 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..

Building Basement and Below Grade Parking Structure Foundation Walls

The native soils at the site are considered to be slightly 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.

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.

Frost Protection Requirements for Spread Footing Foundations

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

The depth of frost cover could be reduced for footings bearing on engineered fill over the native sand. In this case, the combined thickness of earth cover and the engineered fill should be at least 1.5 metres for frost protection purposes. Alternatively, the required frost protection could be provided using a combination of earth cover and extruded polystyrene insulation. Detailed guidelines for footing insulation frost protection could be provided upon request.

Where less than the required depth of soil cover can be provided, the foundation elements should be protected from frost by using a combination of earth cover and extruded polystyrene rigid insulation. A typical frost protection insulation detail could be provided upon request, if required.

Seismic Design for the Proposed Residential Building

For seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class C.

Seismic Site Response Site Class Calculation

Borehole 2					
Layer	Description	Depth (m)	d_i (m)	$N(60)_i$ (blows/0.3m)	d_i/N_i (blows/0.3m)
Underside of Footing Level – Depth 4 m					
1	Sand	4.0	5	29	0.172
3	Bedrock	9.0	25	100	0.25
$sum(d_i/N(60)_i)$					0.422
$d_c/(sum(d_i/N(60)_i)$					71

Since the N(60) = 71 > 50, and the underside of footing level will be only about 5 metres above the bedrock, seismic site response is Site Class C.

Potential for Soil Liquefaction

Consideration for the potential for soil liquefaction was determined by considering the ratio between the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR) for the soils between the proposed underside of footing level and the depth at which refusal to further advancement using standard penetration testing was attained. The CRR value was determined from a mathematical expression as determined by Rauch (1997) of the base curve obtained from Robertson and Fear (1996). The CSR was determined from Seed and Idriss (1971). It is considered that a soil with a normalized SPT of greater than 30 is non-liquefiable. It is also considered that a soil with a CRR/CSR ratio of greater than one is not liquefiable.

The average CRR / CSR ratio for the materials encountered to the depth explored excluding the normalized SPT values above 30 is 1.6. The proposed underside of footing level will be only 5 metres above the bedrock. As such the underlying soils below the proposed foundation are not considered to be liquefiable.

National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.267g 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.

TREE PLANTING RESTRICTIONS

The subsurface soils at the site consisted of fine to medium grained sand which increased in coarseness with depth. Groundwater was measured at a depth of 6.3 metres below the ground surface. From a tree planting perspective, the subsurface conditions would be considered to be well drained sand. These subsurface conditions are not sensitive to volumetric changes due to changing moisture content as a result of dewatering caused by tree roots.

Proposed tree species for planting should be selected based on their suitability for growth in well drained sand soil. There are no other tree planting restrictions from a geotechnical perspective.

RETAINING WALLS

There will be a retaining wall along the south side of the site to support the grade difference between the adjacent row house development and proposed residential development. There will also be retaining walls on either side of the exterior portion of the entrance ramp to the basement parking level.

The retaining wall along the south side of the site will have a height of between about 1.0 and 1.6 metres. The retaining walls at the entrance to the basement parking will range in height from less than 0.6 metres to 1.2 metres.

Since the retaining walls will be above 1.0 metres in height, a global stability assessment is required.
The following assumptions have been made based on the height and proposed location of the retaining wall as well as the information available on the grading plan.

- The wall will consist of a reinforced cast in place concrete wall;
- The wall along the south side of the site will be constructed adjacent to the access easement for the adjacent row house development.
- The wall will have a minimum thickness of 0.25 m;
- The wall will be placed on a concrete footing founded at 1.5 metres below the finished grade at the bottom of the wall for frost protection purposes, alternatively;
- The wall will be provided with rigid insulation below the footing for frost protection purposes and the wall will bear a minimum of 0.6 metres below the finished grade at the bottom of the retaining wall;;
- The retaining wall will be designed by a structural engineer;
- The retaining wall will be structurally separated from the proposed building;
- The retaining wall can be designed assuming that the subgrade soils at the expected founding level of the retaining wall have an allowable bearing capacity of 150 kPa for serviceability limit states design.

The retaining wall should be designed to resist the lateral earth pressure from the retained soils 'p' plus any additional surcharge load adjacent the top of the wall including expected vehicular loads.

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

Where p = the lateral earth pressure, at any depth, h, below the ground surface

k = earth pressure coefficient of 0.27

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 wall foundation

q = the equivalent surcharge acting on the ground surface adjacent to the retaining wall and can be taken as 5 kPa.

For the purposes of this letter, the heel of the retaining wall footing is considered to be the portion of the retaining wall that extends out from the back of the wall under the retained soil on the high side of the wall. The toe of the footing is considered to be the portion of the footing that extends out from the face of the retaining wall under the ground surface on the low side of the retaining wall. In order to resist the lateral earth pressure without rotational failure, it is expected that the heel of the footing for the retaining wall will extend 0.3 metres from the back of the retaining wall and the toe will extend some 0.60 metres from the face of the retaining wall. Based on these expectations the wall will have a factor of safety against overturning of 1.9 to 2.4.

Global Stability of the Retaining Wall

The global stability of the Retaining wall was analysed using GeoStudio: Slope/W (2012) slope stability software. The section of wall analysed represents the highest location of the retaining wall. Seismic Stability from a global perspective was determined using a pseudo static approach using the Seismic information from the Ontario Building Code MMAH Supplementary Standard SB-1 Table 1.2. The analysis was completed assuming the retained soil will be either compacted granular fill or native sand.

Fill – Compacted Granular Material

effective cohesion – 0 kPa,

angle of effective internal friction - 38 degrees,

unit weight - 21.0 KN,

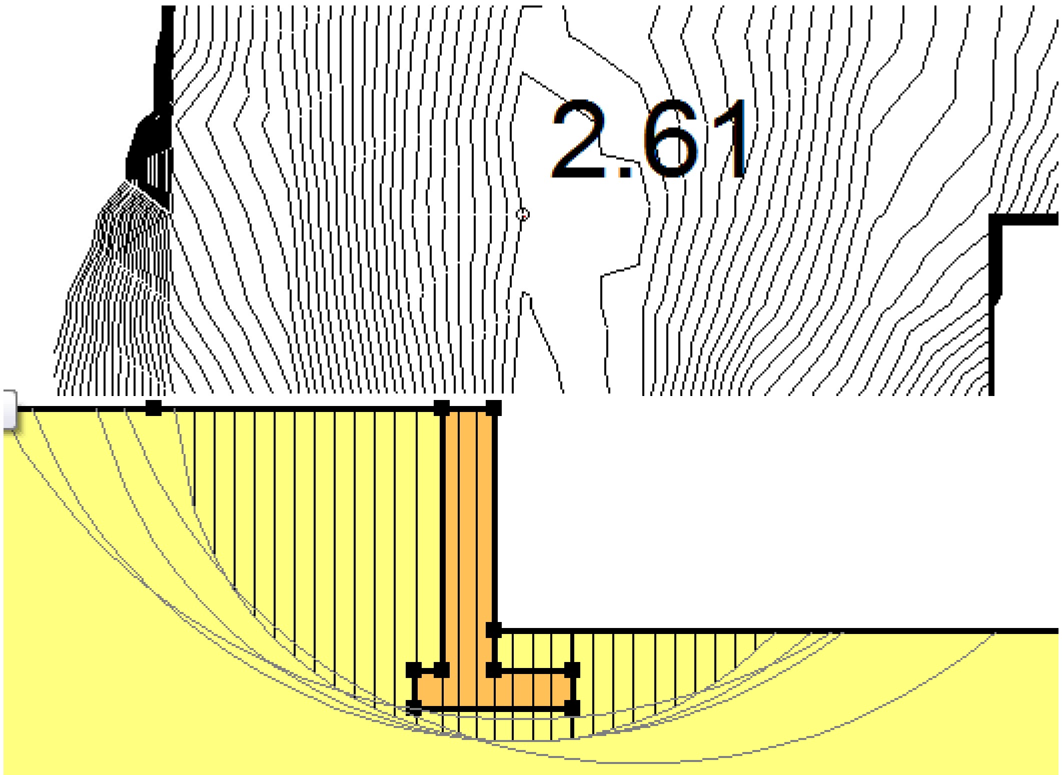
Native Sand

effective cohesion – 0 kPa,

angle of effective internal friction - 36 degrees,

unit weight - 18 KN,

Figure 1 - Slope Stability Assessment Section



From the Analysis, as summarized below, the minimum factor of safety from a Static Global Stability perspective is FS = 2.61 and from a Seismic Perspective FS = 1.61. The figures below show the minimum factors of safety for the above surfaces as well as the Slide Mass properties for the critical surface. The minimum factor of safety from a global stability perspective for a retaining wall under static conditions is FS = 1.5. The minimum factor of safety from a global stability perspective for a retaining wall under seismic conditions is FS = 1.1.

Based on the following analysis results, the proposed retaining wall will be stable from a global perspective during both static and seismic conditions.

Figure 2 – Slip Surfaces Factor of Safety

Slip #	F of S	Center X	Center Y	Radius
90	2.614	25.2136	68.2176	2.66588
321	2.686	25.2136	68.9036	3.35194
782	2.718	25.2136	70.2757	4.55407
574	2.735	26.1072	69.5897	4.20798
552	2.799	25.2136	69.5897	4.038
91	2.823	25.2136	68.2176	2.83587
805	2.845	26.1072	70.2757	4.89404
344	2.853	26.1072	68.9036	3.69191
1013	2.856	25.2136	70.9618	5.24013
113	2.861	26.1072	68.2176	3.00585

Figure 3 – Slide Mass – Critical Surface

Parameter	
Method	Morgenstern-Price
Factor of Safety	2.614
Total Volume	6.0254 m³
Total Weight	114.84 kN
Total Resisting Moment	239.01 kN-m
Total Activating Moment	91.446 kN-m
Total Resisting Force	77.428 kN
Total Activating Force	29.685 kN

Summary of Calculations – Critical Surface
Static Conditions:
Total Weight (kN) = 114.84
Total Resisting Moment (kN*m) = 239.01
Total Acting Moment (kN*m) = 91.446
2.61 FS

Total Resisting Force (kN) = 77.428
Total Activating Force (kN) = 29.685
2.61 FS

Seismic Conditions:
From OBC SB-1 table 1.2 the PGA for Ottawa = 0.32. Horizontal Seismic Coefficient k_h = 0.5 * PGA. k_h = 0.16
Seismic Horizontal Activating Force = k_h x Total Weight = 114.84 x 0.16 = 18.374 kN
Total Resisting Force (from above) = 77.43 kN
Total Seismic Activating Force = Total Activating Force (static conditions + Seismic horizontal Force) = 29.685 + 18.374 = 48.06 kN
Seismic Factor of safety FS = 77.43 / 48.06 = 1.61

SITE SERVICES

Excavation

The excavations for the site services will be carried out through fill materials and sand. 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 3 soil above the groundwater level. 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. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box.

Based on the depths at which groundwater was measured within the boreholes at the time of drilling significant groundwater flow into any excavation is unlikely. Any groundwater inflow into the service trenches should be handled by pumping from sumps from within the excavations.

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.

Pipe Bedding and Cover Materials

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

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

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

Trench Backfill

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

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

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

Underground Stormwater Storage Tanks

The subsurface conditions at the site are not particularly sensitive to changes in moisture content. Periodic saturation of the soils surrounding a stormwater storage tank will not have any significant effect on the subsurface conditions provided the stormwater storage tank was properly installed and backfilled.

The installation of the stormwater storage tank should be completed in accordance with manufacturers recommendations. The subgrade below the storage tanks should be free of loose, soft or otherwise deleterious material It is expected that the subgrade will consist of compacted native sand backfill adjacent the building foundation.

To reduce the potential demand on the perimeter drainage of the building during a storm event, the stormwater storage tank should be provided with an impermeable liner which extends across the bottom and up the sides of the tank to above the 100 year storage level.

Backfill adjacent and above the storage tank should be in keeping with manufactures recommendations and the future use of the area above the storage tanks. To minimize the potential for settlement above the storage tanks, the backfill should be completed in maximum 300 mm thick lifts to at least 95 percent of the standard proctor maximum dry density.

ACCESS ROADWAY

In preparation for pavement construction at this site any topsoil and any soft, wet or deleterious materials should be removed to the elevation of the proposed top of subgrade based on the proposed final grading plan and proposed pavement structure thickness. The exposed sub-grade should be inspected and approved by geotechnical personnel and any soft areas evident should be sub-excavated and replaced with approved engineered fill material. 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 pavement sub-grade level, the material used should consist of OPSS select sub-grade material or OPSS Granular B Type I or Type II. Recycled crushed concrete meeting the grading specifications for Granular B Type II could also be used. Materials used for raising the sub-grade to proposed roadway area sub-grade level should be placed in maximum 300 millimetre thick loose lifts and be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

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

50 millimetres of Superpave 12.5 asphaltic concrete over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase over
(50 or 100 millimetre minus crushed stone)
Non-woven geotextile fabric (4 oz/yd^2) such as Terrafix 270R or Thrace-Ling 130EX or approved alternative.

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. 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 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 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 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, access roadway and parking areas 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 soils 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.

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

Steve DeWit, P.Eng.

Attachments: Appendix A – Summary of Geotechnical Recommendations
List of Abbreviations
Record of Boreholes
Key Plan - Figure 1
Site Plan - Figure 2
Laboratory Test Results for Chemical Properties - Sulphate, Resistivity and pH
Laboratory Test Results for Physical Properties – Stantec Laboratory Test Results for Soils
National Building Code Seismic Hazard Calculation Results

APPENDIX A – SUMMARY OF GEOTECHNICAL RECOMMENDATIONS

This report provides geotechnical recommendations under the Headings: Proposed Residential Development Building Foundations; Retaining Walls; Site Services; Access Roadway Pavements; Construction Considerations:

These geotechnical recommendations include:

Corrosivity on Reinforcement and Sulphate Attack on Portland Cement
Foundation Excavation
Shoring Design
Ground Water in Excavation and Construction Dewatering
Effect of Dewatering of Foundation on Site Services Excavations on Adjacent Structures
Below Grade Basement and Parking Structure Foundation Design
Bearing Capacity
Foundation Drainage
Settlement
Subgrade preparation
Engineered Fill and Compaction
Frost Protection
Foundation Backfill
Floor Slab
Seismic Design
Tree planting Restrictions
Retaining wall Global Stability Analysis
Excavation for Services and Sewers
Bedding and Cover
Trench Backfill
Underground Stormwater Storage Tanks
Subgrade Preparation for Pavements
Pavement Structures
Pavement Placement and compaction
Inspection Requirements.

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 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	over 100

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
Φ'	effective angle of friction
ρ	unit weight of soil
ρ'	unit weight of submerged soil
σ	normal stress

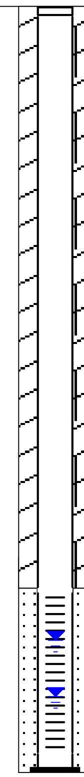
RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: Building Investments and Developments
LOCATION: 841, 845 and 855 Grenon Avenue, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180966
DATE OF BORING: December 18, 2018
SHEET 1 of 1
DATUM: Geodetic

DEPTH SCALE (meters)	DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m
	Ground Surface	74.06			
0	TOPSOIL	0.00			
	Grey brown fine to medium SAND, trace gravel		1	SS	8
1			2	SS	20
2			3	SS	18
			4	SS	23
3			5	SS	24
4			6	SS	25
5			7	SS	23
		68.15 5.91	8	SS	30
6	Grey fine to medium SAND, trace gravel	67.77 6.29			
	Grey fine to coarse SAND, trace silit		9	SS	34
7			10	SS	25
8		65.84 8.22	11	SS	17
	End of Borehole				
9					
10					
11					
12					
13					

UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80
REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80
DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90
ADDITIONAL LAB TESTING
PIEZOMETER OR STANDPIPE INSTALLATION



Water in borehole at about 6.3 metres, December 18, 2018. Water level measured in standpipe at about 6.9 metres below existing ground surface, December 20, 2018.

DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: Building Investments and Developments
LOCATION: 841, 845 and 855 Grenon Avenue, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180966
DATE OF BORING: December 18, 2018
SHEET 1 of 1
DATUM: Geodetic

DEPTH SCALE (meters)	DESCRIPTION	ELEV. / DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface	74.48				
0	Grey brown silty clay, trace gravel and organics (FILL)	0.00	1	SS	9	
	Yellow brown silty sand (FILL)	73.41				
1	Grey brown fine to medium SAND, trace gravel	1.07	2	SS	10	
			3	SS	17	
2			4	SS	18	
3			5	SS	17	
4			6	SS	29	
5			7	SS	28	
			8	SS	29	
6	Grey fine to coarse SAND, trace silt and gravel	68.22 / 6.26	9	SS	31	
7			10	SS	28	
			11	SS	22	
8	Borehole continued by Dynamic Cone Penetration Testing	66.26 / 8.22				Some water observed within borehole at about 6.26 metres below existing ground surface,December 18, 2018.
9	End of borehole. with Practical refusal on large boulder or BEDROCK	65.44 / 9.04				
10						
11						
12						
13						

SOIL PROFILE — STRATA PLOT; SAMPLES

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

DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

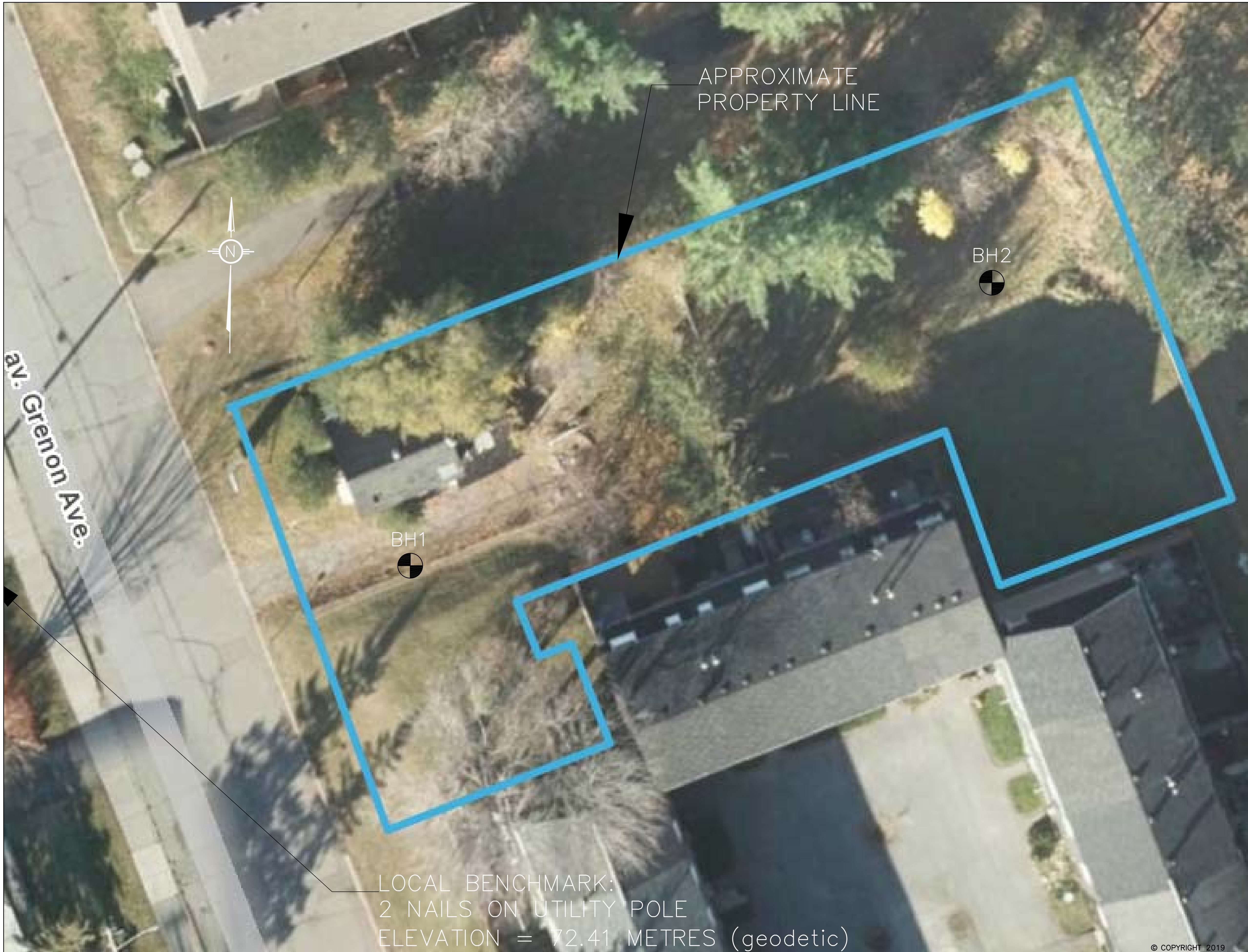
KEY PLAN	FIGURE 1

NOT TO SCALE

Kollaard Associates
Engineers

Project No. 180966

Date August 2019



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
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: BUILDING INVESTMENTS & DEVELOPMENTS

PROJECT: PROPOSED GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION: 841-855 GRENON AVENUE CITY OF OTTAWA, ONTARIO

DESIGNED BY: --	*DATE:* JAN 7, 2019
DRAWN BY: DT	*SCALE:* N.T.S

KOLLAARD FILE NUMBER: 1800966

Laboratory Test Results for Chemical Properties

Kollaard Associates (Kemptville)
ATTN: Dean Tataryn
210 Prescott Street Unit 1
P.O. Box 189
Kemptville ON K0G 1J0

Date Received: 21-DEC-18
Report Date: 28-DEC-18 13:16 (MT)
Version: FINAL

Client Phone: 613-860-0923

Certificate of Analysis

Lab Work Order #: L2214097

Project P.O. #: NOT SUBMITTED
Job Reference: 180966
C of C Numbers:
Legal Site Desc:

Melanie Moshi
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2214097-1 BH2 SS4 10'-12' Sampled By: CLIENT on 18-DEC-18 Matrix: SOIL							
Physical Tests							
Conductivity	0.0409		0.0040	mS/cm		24-DEC-18	R4413312
% Moisture	6.36		0.10	%	22-DEC-18	23-DEC-18	R4412470
pH	7.40		0.10	pH units		24-DEC-18	R4413270
Resistivity	24400		1.0	ohm*cm		24-DEC-18	
Leachable Anions & Nutrients							
Chloride	<5.0		5.0	ug/g	24-DEC-18	27-DEC-18	R4417016
Anions and Nutrients							
Sulphate	<0.002		0.0020	%	22-DEC-18	27-DEC-18	R4417016

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Quality Control Report

Client: Kollaard Associates (Kemptville)
210 Prescott Street Unit 1 P.O. Box 189
Kemptville ON K0G 1J0
Contact: Dean Tataryn

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
SO4-WT	**Soil**							
Batch R4417016								
WG2959762-3 DUP		**L2214097-1**						
Sulphate		N/A	<20	RPD-NA	mg/kg	N/A	30	27-DEC-18
WG2959762-2 LCS								
Sulphate			103.2		%		80-120	27-DEC-18
WG2959762-1 MB								
Sulphate			<20		mg/kg		20	27-DEC-18

Quality Control Report

Client: Kollaard Associates (Kemptville)
210 Prescott Street Unit 1 P.O. Box 189
Kemptville ON K0G 1J0
Contact: Dean Tataryn

Legend:

Limit	ALS Control Limit (Data Quality Objectives)
DUP	Duplicate
RPD	Relative Percent Difference
N/A	Not Available
LCS	Laboratory Control Sample
SRM	Standard Reference Material
MS	Matrix Spike
MSD	Matrix Spike Duplicate
ADE	Average Desorption Efficiency
MB	Method Blank
IRM	Internal Reference Material
CRM	Certified Reference Material
CCV	Continuing Calibration Verification
CVS	Calibration Verification Standard
LCSD	Laboratory Control Sample Duplicate

Sample Parameter Qualifier Definitions:

Qualifier	Description
J	Duplicate results and limits are expressed in terms of absolute difference.
RPD-NA	Relative Percent Difference Not Available due to result(s) being less than detection limit.

Hold Time Exceedances:

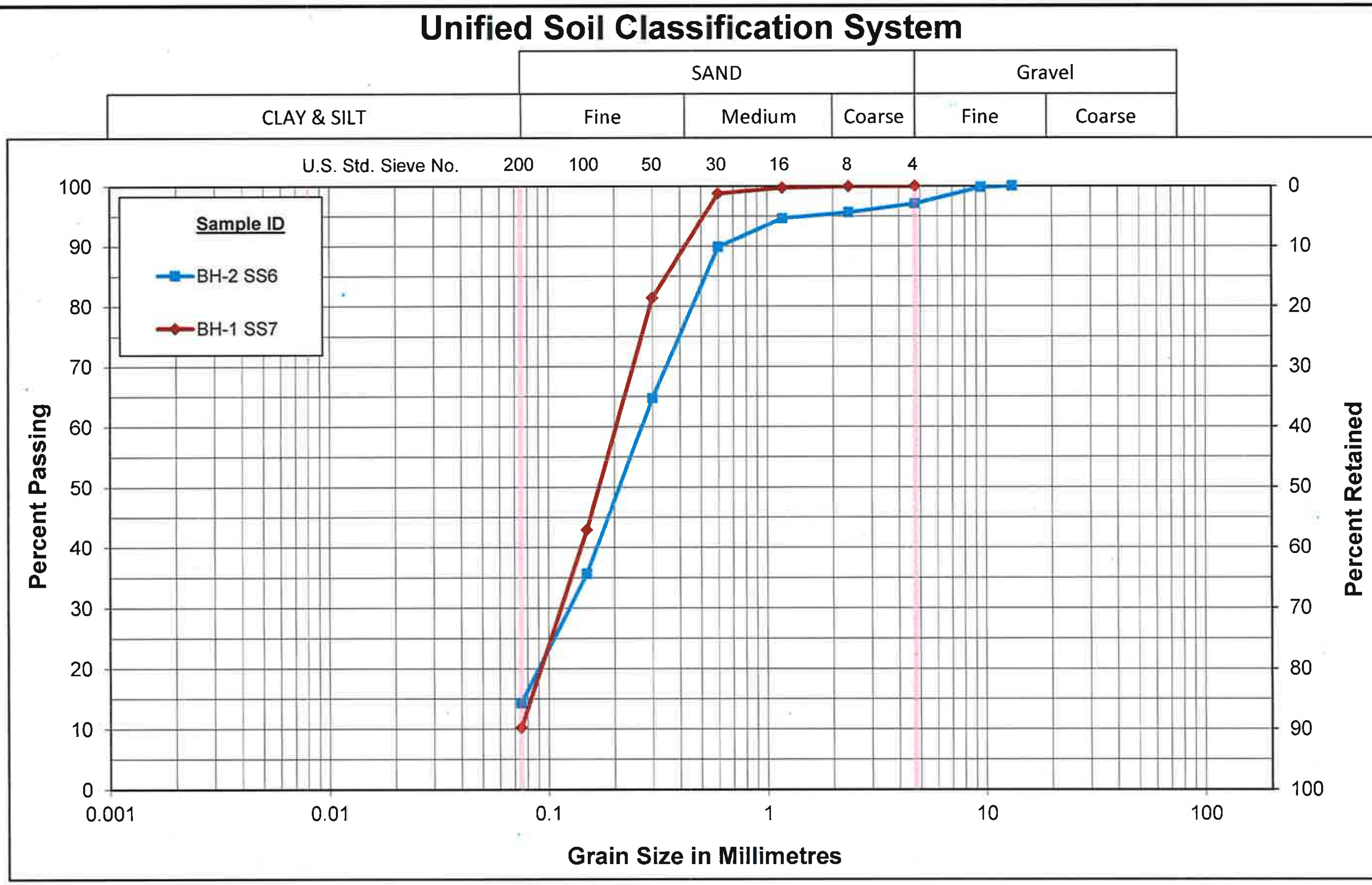
All test results reported with this submission were conducted within ALS recommended hold times.

ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where available). For more information, please contact ALS.

The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide confidence in the accuracy of associated test results.

Please note that this report may contain QC results from anonymous Sample Duplicates and Matrix Spikes that do not originate from this Work Order.

Laboratory Test Results for Physical Properties



GRAIN SIZE DISTRIBUTION

Kollaard Associate Engineers, File #180966

841 Grenon Avenue, Ottawa

Figure No.

Project No. 122410003

Stantec 2781 Lancaster Road
Ottawa ON, K1B 1A7

Sieve Analysis
LS 602
ASTM C136

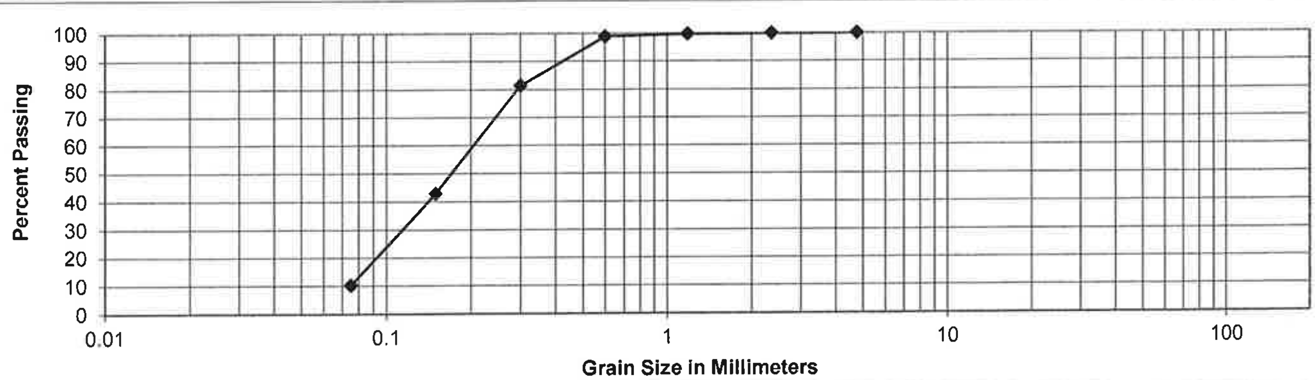
Client:	**Kollaard Associate Engineers, File #180966**	Project Number:	**122410003**
Project:	**841 Grenon Avenue, Ottawa**		
Material Type:	**Soils / Aggregates:**		
Proposed Use:	**Fill/Granulars**		
Source:	**BH-1**		
Sample Number:	**SS7**		
Sampled Depth:	**15'-17'**		
Sampled By:	**Kollaard Associate Engineers**	Tested By:	**Brian Prevost**
Date Sampled :	**December 18, 2018**	Date Tested:	**December 24, 2018**

Sieve Test Data		Wash Test Data		
Sample Weight Before Sieve, (g):	**681.0**	Sample Weight Before Wash, (g):	**262.8**	**Corrected**
Sample Weight After Sieve, (g):	**680.8**	Sample Weight After Wash, (g):	**241.5**	
Percent Loss In Sieve, (%):	**0.03**	Percent Passing No. 200, (%):	**8.1**	**8.1**

Sieve Analysis

Sieve No.	Size of Opening		Weight Retained	Cumulative Weight Retained	Percent Passing	No Envelope	
	Inches	mm	g	g	%	Minimum	Maximum
	6	150					
	4	106					
	3	76.2					
	2	53.0					
	1.5	37.5					
	1	26.5					
	3/4	19.0					
	5/8	16.0					
	1/2	13.2					
	3/8	9.5					
+4	0.187	4.75	0.0	0.0	100.0		
		- 4.75	680.8	680.8			
8	0.0937	2.36		0.2	99.9		
16	0.0469	1.18		0.7	99.7		
30	0.234	0.600		3.2	98.8		
50	0.0117	0.300		49.0	81.4		
100	0.0059	0.150		150.0	42.9		
200	0.0029	0.075		235.9	10.2		
		Pan		241.3			

Classification of Sample:	% Gravel: **0.0**	% Sand:	**89.8**	% Silt & Clay:	**10.2**



Remarks:

Reviewed By: Brian Prevost **Date:** December 28, 2018

V:\01216\active\laboratory_standing_offers\2018 Laboratory Standing Offers\122410003 Kollaard Associate Engineers\December 18, Two Sieves, Kollaard #180966\Sieve.xlsx

Stantec 2781 Lancaster Road
Ottawa ON, K1B 1A7

Sieve Analysis
LS 602
ASTM C136

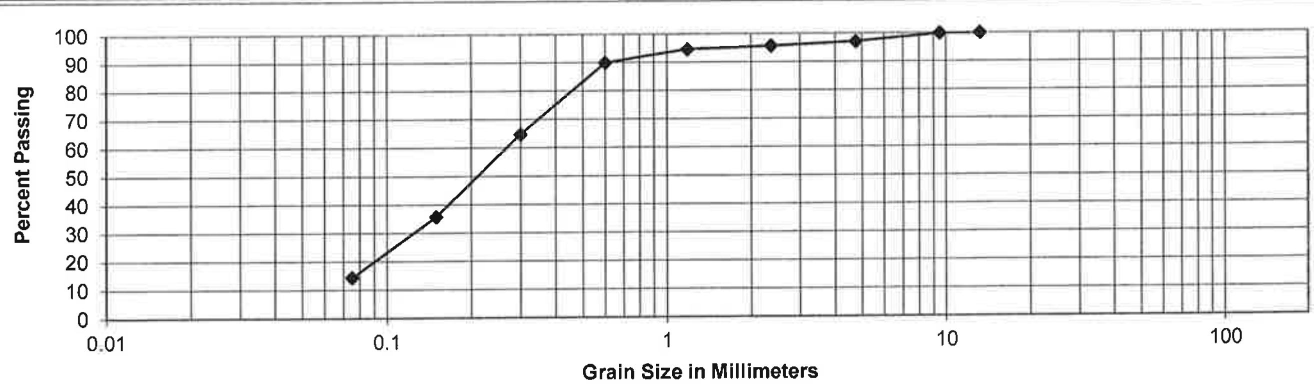
Client:	**Kollaard Associate Engineers, File #180966**	Project Number:	**122410003**
Project:	**841 Grenon Avenue, Ottawa**		
Material Type:	**Soils / Aggregates:**		
Proposed Use:	**Fill/Granulars**		
Source:	**BH-2**		
Sample Number:	**SS6**		
Sampled Depth:	**12'6"-14'6"**		
Sampled By:	**Kollaard Associate Engineers**	Tested By:	**Brian Prevost**
Date Sampled:	**December 18, 2018**	Date Tested:	**December 24, 2018**

Sieve Test Data		Wash Test Data		
Sample Weight Before Sieve, (g):	746.7	Sample Weight Before Wash, (g):	287.2	Corrected
Sample Weight After Sieve, (g):	746.3	Sample Weight After Wash, (g):	250.5	
Percent Loss In Sieve, (%):	0.05	Percent Passing No. 200, (%):	12.8	12.4

Sieve Analysis

Sieve No.	Size of Opening		Weight Retained	Cumulative Weight Retained	Percent Passing	No Envelope	
	Inches	mm	g	g	%	Minimum	Maximum
	6	150					
	4	106					
	3	76.2					
	2	53.0					
	1.5	37.5					
	1	26.5					
	3/4	19.0					
	5/8	16.0					
	1/2	13.2	0.0	0.0	100.0		
	3/8	9.5	1.4	1.4	99.8		
+4	0.187	4.75	20.5	21.9	97.1		
		- 4.75	724.4	746.3			
8	0.0937	2.36		4.3	95.6		
16	0.0469	1.18		7.4	94.6		
30	0.234	0.600		21.3	89.9		
50	0.0117	0.300		95.7	64.7		
100	0.0059	0.150		181.7	35.7		
200	0.0029	0.075		244.9	14.3		
		Pan		249.9			

Classification of Sample: % Gravel: **2.9** % Sand: **82.8** % Silt & Clay: **14.3**



Remarks:

Reviewed By: Brian Prevost **Date:** December 28, 2018

V:\01216\active\laboratory_standing_offers\2018 Laboratory Standing Offers\122410003 Kollaard Associate Engineers\December 18, Two Sieves, Kollaard #180966\Sieve.xlsx

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

January 07, 2019

Site: 45.3557 N, 75.7996 W User File Reference: 841 Grenon Avenue

Requested by: ,

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)**
0.423	0.497	**0.418**	0.318	**0.226**	**0.113**	**0.054**	**0.014**	**0.0053**	**0.267**	**0.188**

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)	0.041	0.137	0.230
Sa(0.1)	0.057	0.174	0.281
Sa(0.2)	0.052	0.152	0.241
Sa(0.3)	0.042	0.118	0.185
Sa(0.5)	0.030	0.084	0.132
Sa(1.0)	0.015	0.043	0.067
Sa(2.0)	0.0059	0.020	0.032
Sa(5.0)	0.0012	0.0046	0.0078
Sa(10.0)	0.0006	0.0018	0.0031
PGA	0.031	0.095	0.153
PGV	0.020	0.065	0.106

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