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REPORT ON

GEOTECHNICAL INVESTIGATION PROPOSED RESIDENTIAL DEVELOPMENT 16 & 20 CHAMPAGNE AVENUE SOUTH CITY OF OTTAWA, ONTARIO

Project # 180308

Submitted to:

JLG Developments Inc.
9 Calais Court
Nepean, Ontario
K2E 7E1

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July 27, 2018 180308

JLG Developments Inc.
9 Calais Court
Nepean, Ontario
K2E 7E1

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
16 & 20 CHAMPAGNE AVENUE SOUTH
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 16 & 20 Champagne Avenue South in the City of Ottawa, Ontario (see Key Plan, Figure 1). In total, the site consists of about 0.07 hectares (0.16 acres) of land located on the west side of Champagne Avenue South, about 150 metres north of the intersection of Champagne Avenue South and Beech Street, City of Ottawa, Ontario. The site is currently occupied by two residential dwellings along with two small storage sheds. The remaining areas not occupied by the dwellings or storage sheds are mostly grassed and asphaltic surfaced.

Preliminary plans indicate that the building will be three and a half storeys and will contain about 19 units. 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.

Surrounding land use is residential development. The site is bordered on the west, north and south by residential development, on the east by Champagne Street South 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 shallow bedrock or possibly glacial till. Bedrock geology maps indicate that the bedrock underlying the site consists of dark grey almost black limestone of the Eastview Formation or limestone with some shaly partings of the Ottawa Formation.

Based on a review of overburden thickness mapping for the site area, the overburden is estimated to be between about 1.5 to 3.0 metres in thickness above bedrock.

PROCEDURE

The field work for this investigation was carried out on August 9, 2017 at which time three boreholes, numbered BH1, BH2 and BH3 were put down at the site using a truck mounted drill rig equipped with a hollow stem auger owned and operated by Marathon Drilling Co. Ltd. of Greely, 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. The boreholes were loosely backfilled with the auger cuttings upon completion of drilling.

One soil sample (BH2) were submitted for Atterberg Limits (D4318), Moisture Content (ASTM D2216) and Hydrometer testing (ASTM D422). 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 following is a brief overview of the subsurface conditions encountered at the boreholes.

Fill

From the ground surface, fill materials were encountered at all three of the borehole locations. The fill materials consisted of about a 80 to 100 millimetre thickness of asphaltic concrete followed by about 120 to 210 millimetres of grey crushed stone. Beneath the grey crushed stone at BH2 and BH3, fill materials consisting of yellow brown sand and gravel with a trace of clay and topsoil was encountered at both boreholes. The thickness of the fill materials ranged from about 300 to 870 millimetres. The fill materials were fully penetrated at the borehole locations.

Silty Clay

A thin deposit of grey brown silty clay was encountered below the fill materials at BH1. The thickness of the silty clay was about 1.8 metres. The results of standard penetration testing carried out in the silty clay material, range from 8 to 11 blows per 0.3 metres with an average value of 9.5 blows per 0.3 metres, indicating a dense state of packing.

Glacial Till/Bedrock

Beneath the silty clay at BH1 and fill materials at BH2 and BH3, glacial till was encountered. The glacial till consisted of gravel, cobbles and boulders, in a matrix of grey brown to grey sand, with a trace to some clay. The results of standard penetration testing carried out in the glacial till material, range from 3 to 70 blows per 0.3 metres with an average value of 19 blows per 0.3 metres, indicating a compact to dense state of packing. A thin layer of grey silt was encountered within the glacial till in BH1 at about 5.3 metres below the existing ground surface. The silt layer measured about 0.6 metres in thickness. The results of standard penetration testing carried out in the silt material was 12 blows per 0.3 metres, indicating a compact state of packing.

All of the boreholes encountered refusal to further advancement of the standard penetration split spoon at depths of about 7.9, 7.8 and 6.5 metres, respectively, below the existing ground surface for boreholes BH1, BH2 and BH3 on the surface of large boulders or bedrock.

The results of Atterberg Limits and a moisture content test conducted on one soil sample of glacial till is presented in Table I and in Attachment A at the end of the report. The tested glacial till sample classifies as inorganic non-plastic silt (ML) in accordance with the Unified Soil Classification System.

Table I – Atterberg Limit and Water Content Results

Sample	Depth(metres)	LL (%)	PL (%)	PI (%)	W (%)
BH2-SS4	2.28 - 2.89	16.1	12.1	4.0	65.2

LL: Liquid Limit PL: Plastic Limit PI: Plasticity Index w: water content
ML: Non-Plastic Silt

One soil sample of glacial till (BH2 - SS4 - 2.28 to 2.89 metres) was submitted to Stantec for particle size analysis (ASTM D2216). The results of the particle size analysis testing indicated that the sample consists of about 12 percent clay, 38 percent fine to coarse sand and about 25 percent gravel. The sample is indicated to have between about 36 to 38 percent silt and clay size particles.

The results are located in Attachment A.

Groundwater

Groundwater was encountered at all of the boreholes at the time of drilling on July 10, 2018 at depths of about 5.3, 6.9 and 2.6 metres 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

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	8.48	Slightly Basic Negligible concern
Resistivity	R < 20,000 ohm-cm	5260	Moderately Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	<0.01	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 8.48, 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. Soil resistivity was found to be 5.26 ohm-m for the sample analyzed. Consideration to increasing the specified strength and/or adding air entrainment into any reinforced concrete in contact with the soil should be given. Special protection is required for reinforcement steel within the concrete walls.

GEOTECHNICAL DESIGN GUIDELINES

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 for Proposed Multi-Unit Residential Building

As previously indicated, the subsurface conditions at the site encountered at the boreholes advanced during the investigation consisted of asphaltic concrete, crushed stone and deleterious fill materials followed by silty clay and glacial till. With the exception of the asphaltic concrete, crushed stone and fill materials, 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. It is suggested that the building be founded either directly on the underlying glacial till or on engineered fill placed on the glacial till.

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 building foundation.

Conventional Spread Footing Foundations

For the proposed below grade basement foundation, a maximum allowable bearing pressure of 100 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 200 kilopascals using ultimate limit states design, may be used for the design of conventional strip footings or pad footings, founded on glacial till or on a suitably constructed engineered pad placed on the glacial till above the ground water level at a maximum depth of 2.5 metres. The above allowable bearing capacities are subject to a maximum strip footing width of 2.0 metres and maximum pad footings widths of 2.5 metres.

The above allowable bearing pressures are subject to a maximum grade raise above the existing ground surface of 2.0 metres. Provided that any loose and 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.

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

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

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.

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 glacial till. 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.

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

Basement Floor Slab

As stated above, it is expected that the proposed building will be founded on the native glacial till or on an engineered pad placed on the glacial till. 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 A or 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 followed by a minimum of 0.3 metres thickness of 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 engineered fill materials should be compacted in maximum 300 millimetre thick lifts to at least 98 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 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.

If any areas of the proposed building are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

Building Basement Structure Foundation Walls

The native soils at the site are considered to be frost susceptible. As such, to prevent possible foundation frost jacking, the backfill against unheated walls or isolated walls or piers should consist of free draining, non-frost susceptible material such as sand or sand and gravel meeting OPSS Granular B Type I grading requirements. Alternatively, foundations could be backfilled with native material in conjunction with the use of an approved proprietary drainage layer system against the foundation wall. It is pointed out that there is potential for possible frost jacking of the upper portion of some types of these drainage layer systems if frost susceptible material is used as backfill. This could be mitigated by backfilling the upper approximately 0.6 metres with non-frost susceptible granular material.

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

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

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

h = the depth, in metres, below the finished ground surface at which the pressure, P, is being computed

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

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

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

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

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 D.

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)
1	Glacial Till	2.5	5.3	13	0.396
2	Bedrock	7.80	24.7	100	0.247
$sum(d_i/N(60)_i)$					0.643
$d_c/(sum(d_i/N(60)_i)$					46.6

Since the 15 < N(60) = 46.6 < 50, the seismic site response is Site Class D.

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 1.1. 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 0.280g 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.

SITE SERVICES

Excavation

The excavations for the site services will be carried out through fill materials and glacial till. For the purposes of Ontario Regulation 213/91 the soils at the site can be considered to be Type 3 soil. The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupational Health and Safety Act. That is, open cut excavations with overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box.

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.

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.

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.

ACCESS ROADWAY AND PARKING AREA PAVEMENTS

In preparation for pavement construction at this site the existing asphalt and underlying fill material 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/sy) 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 silt and silty clay deposits within the fill materials and glacial till 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: Record of Boreholes
Figures 1 and 2
Laboratory Test Results for Chemical Properties
Laboratory Test Results for Physical Properties – Stantec Laboratory Test Results for Soils

RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: JLG Developments Inc.
LOCATION: 16 & 20 Champagne Avenue South, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180308
DATE OF BORING: July 10, 2018
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (M)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	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
	Ground Surface								
0	ASPHALTIC CONCRETE	0.00							
	Grey crushed stone (FILL)		1	SS	8				
	Stiff grey brown SILTY CLAY	0.30							
1			2	SS	11				
			3	SS	8				
2	Grey brown silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)	2.10	4	SS	14				
3			5	SS	8				
	Grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)	3.40							
4			6	SS	4				Some water observed within borehole at about 5.3 metres below existing ground surface, July
5			7	SS	22				
	Grey SILT	5.35	8	SS	12				
6	Grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)	5.94	9	SS	23				
7			10	SS	38				
			11	SS	70				
8	End of borehole, Refusal on bedrock or large boulder	7.94							

DEPTH SCALE: 1 to 50
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: JLG Developments Inc.
LOCATION: 16 & 20 Champagne Avenue South, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180308
DATE OF BORING: July 10, 2018
SHEET 1 of 1
DATUM:

DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m
Ground Surface				
ASPHALTIC CONCRETE	0.00			
Grey crushed stone (FILL)		1	SS	5
Yellow brown sand and gravel, trace clay and topsoil (FILL)	0.30	2	SS	3
Grey brown silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)	1.17	3	SS	6
		4	SS	26
		5	SS	5
Grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)	3.52	6	SS	5
		7	SS	4
		8	SS	23
		9	SS	32
		10	SS	41
		11	SS	50
End of borehole, Refusal on bedrock or large boulder	7.80			

Column headings: DEPTH SCALE (meters) 0–8; 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; PIEZOMETER OR STANDPIPE INSTALLATION

PIEZOMETER OR STANDPIPE INSTALLATION: Some water observed within borehole at about 6.9 metres below existing ground surface, July 10, 2018.

DEPTH SCALE: 1 to 50
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH3

PROJECT: Proposed Residential Development
CLIENT: JLG Developments Inc.
LOCATION: 16 & 20 Champagne Avenue South, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180308
DATE OF BORING: July 10, 2018
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m
	Ground Surface				
0	ASPHALTIC CONCRETE	0.00			
	Grey crushed stone (FILL)	0.20	1	SS	8
	Yellow brown sand and gravel, trace clay and topsoil (FILL)	0.50			
1	Grey brown silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)		2	SS	3
2			3	SS	10
	Grey silty sand, some gravel, cobbles and boulders, some clay (GLACIAL TILL)	2.58	4	SS	6
3			5	SS	6
4			6	SS	6
5			7	SS	5
			8	SS	16
6			9	SS	50
	End of borehole, Refusal on bedrock or large boulder	6.50			
7					
8					

UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80

REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80

DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90

ADDITIONAL LAB TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Some water observed within borehole at about 2.6 metres below existing ground surface, July 10, 2018.

DEPTH SCALE: 1 to 50
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

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

PENETRATION RESISTANCE

Standard Penetration Resistance, N

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

Dynamic Penetration Resistance

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

WH

_Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from 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

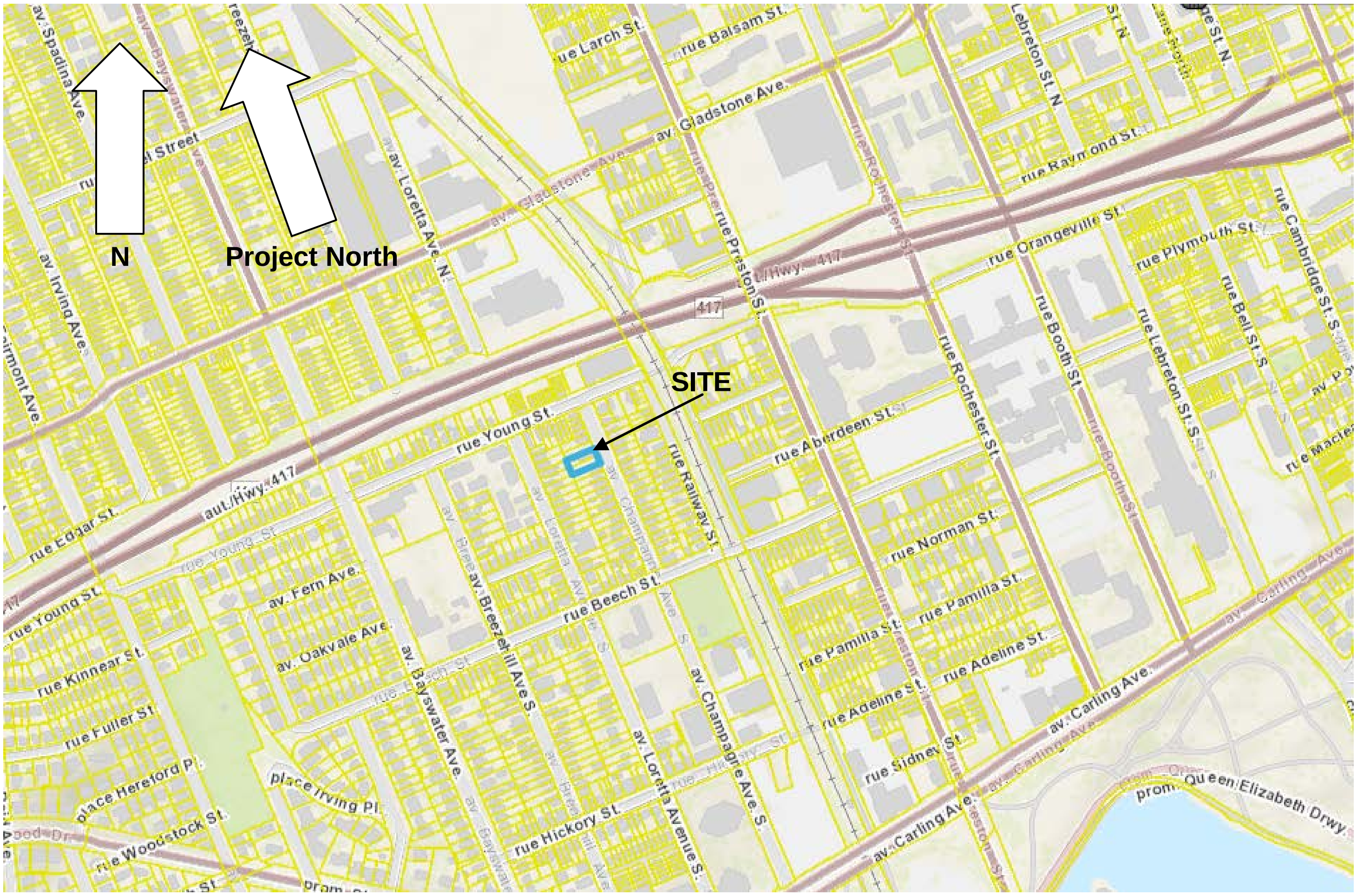
Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to100
Very Stiff	over100

LIST OF COMMON SYMBOLS

c_u undrained shear strength
e void ratio
C_c compression index
C_v coefficient of consolidation
k coefficient of permeability
I_p plasticity index
n porosity
u pore pressure
w moisture content
w_L liquid limit
w_p plastic limit
$\$^1$ effective angle of friction
r unit weight of soil
y^1 unit weight of submerged soil
cr normal stress

KEY PLAN

FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 180308

Date July 2018

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:

JLG DEVELOPMENTS INC.

PROJECT:

GEOTECHNICAL INVESTIGATION FOR RESIDENTIAL BUILDING

LOCATION:

16 & 20 CHAMPAGNE AVE SOUTH
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
--	JUNE 29, 2018
DRAWN BY: DT	SCALE: N.T.S

KOLLAARD FILE NUMBER:

180308

Laboratory Test Results for Chemical Properties

Certificate of Analysis

Environment Testing

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1812226
		Date Submitted:	2018-07-12
		Date Reported:	2018-07-18
		Project:	180308
		COC #:	195670
Attention:	Mr. Dean Tataryn		
PO#:	180140		
Invoice to:	Kollaard Associates Inc.		

Dear Dean Tataryn:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Sarah Horner, Inorganics Technician

All analysis is completed in Ottawa, Ontario (unless otherwise indicated).

Eurofins Ottawa is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on our CALA scope of accreditation. It can be found at http://www.cala.ca/scopes/2602.pdf.

Eurofins(Ottawa) is certified and accredited for specific parameters by OMAFRA, Ontario Ministry of Agriculture, Food and Rural Affairs (for farm soils). Licensed by Ontario MOE for specific tests in drinking water.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required.

Certificate of Analysis

eurofins | Environment Testing

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1812226
Attention:	Mr. Dean Tataryn	Date Submitted:	2018-07-12
PO#:	180140	Date Reported:	2018-07-18
Invoice to:	Kollaard Associates Inc.	Project:	180308
		COC #:	195670

Group	Analyte	MRL	Units	Guideline	Lab I.D. 1373829 Sample Matrix Soil Sample Type Sampling Date 2018-07-10 Sample I.D. BH2-SS3
Anions	Cl	0.002	%		0.005
	SO4	0.01	%		<0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.19
	pH	2.00			8.48
	Resistivity	1	ohm-cm		5260

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

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

Certificate of Analysis

Environment Testing

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0		Report Number:	1812226
Attention:	Mr. Dean Tataryn		Date Submitted:	2018-07-12
PO#:	180140		Date Reported:	2018-07-18
Invoice to:	Kollaard Associates Inc.		Project:	180308
			COC #:	195670

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 349180 **Analysis/Extraction Date** 2018-07-13 **Analyst** C F **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	85-115
pH	<2.00	99	90-110
Resistivity			
Run No 349426 **Analysis/Extraction Date** 2018-07-18 **Analyst** C F **Method** AG SOIL			
SO4	<0.01 %	94	70-130
Run No 349436 **Analysis/Extraction Date** 2018-07-18 **Analyst** C F **Method** C CSA A23.2-4B			
Chloride		102	90-110

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

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

Laboratory Test Results for Physical Properties

Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

Stantec

July 25, 2018
File: 122410003

Attention: **Dean Tataryn, Kollaard Associates Engineers**

Reference: **Kollaard File #180308**
ASTM D4318 Atterberg Limit & ASTM D2216 Moisture Content

The table below summarizes Atterberg Limit & Moisture Content results.

Source	Depth	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index
BH-2 SS4	7'6"-9'6"	65.2	16.1	12.1	4.0

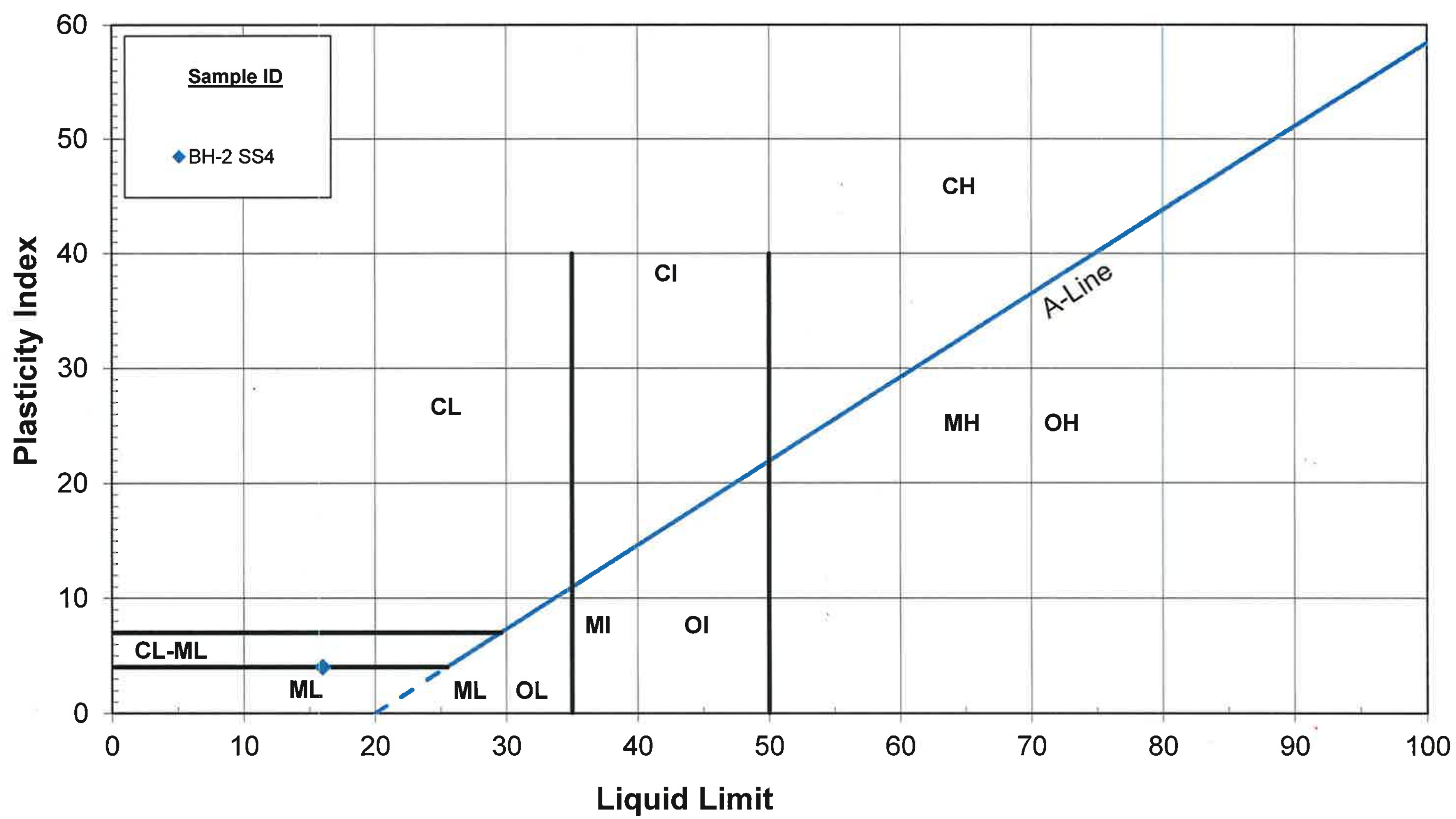
Sincerely,

Stantec Consulting Ltd

Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-722-2799
brian.prevost@stantec.com

Attachments: Atterberg Limit Plasticity Chart



Stantec

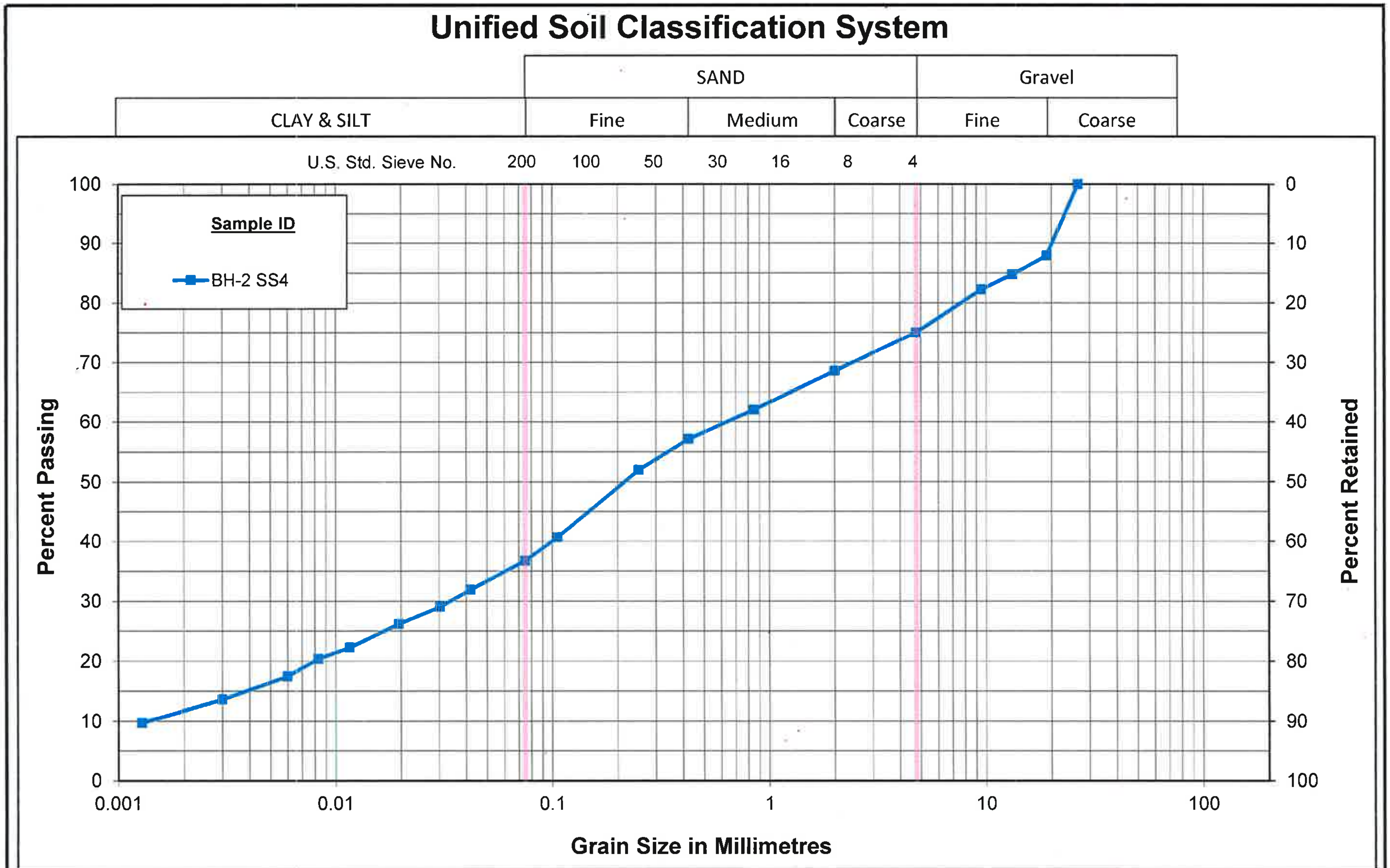
Kollaard Associates, File #180308

16/20 Champagne Street South

PLASTICITY CHART

Figure No.

Project No. 122410003



GRAIN SIZE DISTRIBUTION

Kollaard Associates , File #180308

16/20 Champagne Street South

Figure No.

Project No. 122410003

Stantec
2781 Lancaster Road
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates , File #180308	Project No.:	122410003
Project:	16/20 Champagne Street South	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates
Source:	BH-2	Date Sampled:	July 13, 2018
Sample No.:	SS4	Tested By:	Daniel Boateng
Sample Depth:	7'6"-9'6"	Date Tested:	July 19, 2018

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	69.41
Sample Weight after Hydrometer and Wash (g)	33.21
Percent Passing No. 200 Sieve (%)	52.2
Percent Passing Corrected (%)	35.77

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	552.70
Sample Weight After Sieve (g)	552.00
Percent Loss in Sieve (%)	0.13

SOIL INFORMATION

Liquid Limit (LL)	16.0	
Plasticity Index (PI)	4.0	
Soil Classification	ML	
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	24	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	71.87
Air Dried Mass (W_a), (g)	72.39
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9928
Air Dried Mass in Analysis (M_a), (g)	69.91
Oven Dried Mass in Analysis (M_o), (g)	69.41
Percent Passing 2.0 mm Sieve (P_{10}), (%)	68.59
Sample Represented (W), (g)	101.19

SIEVE ANALYSIS

Sieve Size mm	Cum. Wt. Retained	Percent Passing
75.0		100.0
63.0		100.0
53.0		100.0
37.5		100.0
26.5	0.0	100.0
19.0	66.8	87.9
13.2	84.4	84.7
9.5	98.3	82.2
4.75	138.1	75.0
2.00	173.6	68.6
Total (C + F)[1]	552.00	
0.850	6.64	62.03
0.425	11.62	57.11
0.250	16.82	51.97
0.106	28.29	40.63
0.075	32.27	36.70
PAN	33.18	

Note 1: (C + F) = Coarse + Fine

HYDROMETER DETAILS

Volume of Bulb (V_B), (cm^3)	63.0
Length of Bulb (L_2), (cm)	14.47
Length from '0' Reading to Top of Bulb (L_1), (cm)	10.29
Scale Dimension (h_s), (cm/Div)	0.155
Cross-Sectional Area of Cylinder (A), (cm^2)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 6:31 AM

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
19-Jul-18	6:32 AM	1	37.0	4.0	22.0	33.0	31.91	10.47691	9.6157	0.0130	0.0420
19-Jul-18	6:33 AM	2	34.0	4.0	22.0	30.0	29.01	10.94191	9.6157	0.0130	0.0303
19-Jul-18	6:36 AM	5	31.0	4.0	22.0	27.0	26.10	11.40691	9.6157	0.0130	0.0196
19-Jul-18	6:46 AM	15	27.0	4.0	22.0	23.0	22.24	12.02691	9.6157	0.0130	0.0116
19-Jul-18	7:01 AM	30	25.0	4.0	22.0	21.0	20.30	12.33691	9.6157	0.0130	0.0083
19-Jul-18	7:31 AM	60	22.0	4.0	22.0	18.0	17.40	12.80191	9.6157	0.0130	0.0060
19-Jul-18	10:41 AM	250	18.0	4.0	22.0	14.0	13.54	13.42191	9.6157	0.0130	0.0030
20-Jul-18	6:31 AM	1440	14.0	4.0	22.0	10.0	9.67	14.04191	9.6157	0.0130	0.0013

Remarks:

Reviewed By: Brian Prevost
Date: July 25/2018

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

July 24, 2018

Site: 45.4011 N, 75.7133 W User File Reference: 16

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)**
0.445	0.521	**0.438**	0.333	**0.236**	**0.118**	**0.056**	**0.015**	**0.0054**	**0.280**	**0.196**

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)	0.044	0.148	0.246
Sa(0.1)	0.061	0.186	0.298
Sa(0.2)	0.055	0.160	0.254
Sa(0.3)	0.043	0.124	0.194
Sa(0.5)	0.031	0.088	0.138
Sa(1.0)	0.015	0.044	0.069
Sa(2.0)	0.0061	0.020	0.033
Sa(5.0)	0.0012	0.0047	0.0081
Sa(10.0)	0.0006	0.0019	0.0032
PGA	0.032	0.101	0.162
PGV	0.021	0.068	0.110

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