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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
5536 MANOTICK MAIN STREET
CITY OF OTTAWA, ONTARIO

Project # 190117

Submitted to:

Royal Lepage Team Realty
101-555 Legget Drive
Kanata, Ontario
K2K 2X3

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June 11, 2019

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RE: GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
5536 MANOTICK MAIN STREET, MANOTICK
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed commercial 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 recommendations and guidelines on the geotechnical engineering aspects of the project design; including construction considerations, which could influence design decisions.

BACKGROUND INFORMATION AND SITE GEOLOGY

The site consists of rectangular shaped parcel of land some 0.16 hectares (0.40 acres) in plan area located on the west side of Manotick Main Street, about 160 metres south of the intersection of Bridge Street and Manotick Main Street, City of Ottawa, Ontario. The site is currently occupied by a an existing two storey commercial building with asphaltic surfaced access and a gravel surfaced parking lot (see Key Plan, Figure 1).

It is understood that plans are being prepared to re-develop the property and construct a new two storey commercial building at the site. This will include rear yard parking. It is understood that the proposed building will be of wood framed construction with a conventional cast in place concrete basement foundation. The proposed building will be serviced by municipal sewer and water supply. The proposed development will be accessed by local commercial roadways. Surface drainage for the proposed development will be by means of swales, catch basins and storm sewers.

Surrounding land use is mixed commercial development. The site is bordered on the east by Manotick Main Street, on the west by Ann Street, and on the north and south by commercial development.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by silty clay and/or silt. Bedrock geology maps indicate that the bedrock underlying the site consists of dolomite of the Oxford Formation. Based on a review of the topographical map for the site area, it is expected that the upper groundwater flow is north towards the Rideau River.

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. A review of a report by CM3 Environmental Inc. entitled Limited Phase II Subsurface Site Assessment, 5536 Manotick Main Street, Manotick, Ontario, Project Number MM2103, dated January 16, 2018 indicated that boreholes put down at the site encountered an overburden thickness above refusal on bedrock between about 1.4 to 3.1 metres below the existing ground surface.

PROCEDURE

The field work for this investigation was carried out on March 18, 2019 at which time three boreholes, numbered BH1, BH2 and BH3 were put down at the site using a track mounted drill rig equipped with a hollow stem auger owned and operated by OGS Drilling of Almonte, 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 soft cohesive materials were encountered at any of the boreholes.

Each of the boreholes were advanced to depths of about 2.0 to 2.9 metres below the existing ground surface. The soils were classified using the Unified Soil Classification System.

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 soil sample (BH1) was submitted for hydrometer and moisture content (ASTM D422 and ASTM D2216). A sample of soil obtained from BH1 was also delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack and soil corrosion on buried concrete and steel.

The field work was supervised throughout by a member of our engineering staff who located the 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 locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than borehole locations may vary from the conditions encountered at the boreholes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification was in general completed by visual-manual procedures in accordance with ASTM 2488 - Standard Practice for Description and

Identification of Soils (Visual-Manual Procedure) 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.

Asphaltic Concrete

Asphaltic concrete was encountered from the surface at borehole BH1. The asphaltic concrete measured between 50 millimetres in thickness.

Fill

Beneath the asphaltic concrete at borehole BH1 and from the surface at boreholes BH2 and BH3, a layer of grey crushed stone ranging in thickness from about 150 to 400 millimetres was encountered at the boreholes. Following the grey crushed stone layers, fill materials consisting of yellow brown sand and gravel and yellow brown to grey brown silty clay with a trace of sand and organics was encountered. The fill materials ranged in thickness from about 1.2 to 1.3 metres and were encountered at depths of about 0.3 to 1.7 metres below the existing ground surface. The results of standard penetration testing carried out in the fill materials for all of the boreholes ranged from 6 to 39 blows per 0.3 metres with an average value of 17.5 blows per 0.3 metres, indicating a compact state of packing. The fill materials were fully penetrated at all three borehole locations.

Silty Clay

A deposit of native grey brown silty clay was encountered, below the fill layers, at all of the borehole locations.

The results of the standard penetration testing carried out in the silty clay material at the boreholes ranged from 3 to 33 blows per 0.3 metres. The results of standard penetration testing and tactile examination carried out for the silty clay material indicate that the silty clay is stiff to very stiff in consistency. All of the boreholes were terminated with practical refusal at depths of about 2.94, 2.48 and 1.95 metres, respectively, below the existing ground surface.

One soil sample of silty clay (BH1 - SS4 - 2.28 to 2.89 metres) was submitted to Stantec for particle size analysis (ASTM D422 and D2216). The results of the hydrometer testing indicated that the sample consisted of about 96.7 percent silt and clay size particles. The grain size analysis of the borehole sample indicates 3.3 percent sand content. The sample has a clay content of 24 percent, where clay is defined as particle sizes of less than 2 µm diameter. The sample had a moisture content of 34 percent.

The results are located in Attachment A.

Bedrock

Beneath the native silty clay, refusal for advancement on the surface of un-fractured bedrock was encountered at depths of about 2.94, 2.48 and 1.95 metres, respectively, below the existing ground surface.

A detailed account of the subsurface conditions encountered in each of the boreholes is provided in the attached Table I, Record of Boreholes.

Groundwater

No groundwater seepage was observed within the boreholes at the time of drilling on March 18, 2019. 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 %	0.0241	Negligible
pH	5.0 < pH	7.35	Slightly Basic Negligible concern
Resistivity	R < 20,000 ohm-cm	1270	Highly Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	<0.01	Negligible concern

The results of the laboratory testing of a soil sample for sulphate gave a percent sulphate of less than 0.01. The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete based on percent sulphate in soil. From 0 to 0.10 percent the potential is negligible, from 0.10 to 0.20 percent the potential is mild but positive, from 0.20 to 0.50 percent the potential is considerable and 0.50 percent and greater the potential is severe. Based on the above, the soils are considered to have a negligible potential for sulphate attack on buried 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 7.35, 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.

Corrosivity Rating for soils ranges from extremely corrosive with a resistivity rating <1000 ohm-cm to moderately corrosive with a resistivity of 5000 to 10,000 ohm-cm to non-corrosive with a resistivity of >20,000 ohm-cm. The Soil resistivity was found to be 1270 ohm-cm for the sample analyzed making the soil corrosive for buried steel. Consideration to increasing the specified strength and/or adding air entrainment into any reinforced concrete in contact with the soil should be given. Protection is required for reinforcement steel within the concrete foundation walls.

GEOTECHNICAL DESIGN GUIDELINES AND RECOMMENDATIONS

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 Design for Proposed Commercial 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 silty clay overlying bedrock.

With the exception of the asphaltic concrete, crushed stone and fill materials, the subsurface conditions are suitable for the support of the proposed commercial building on conventional spread footing foundations placed on a native, undisturbed silty clay subgrade or engineered fill used to raise the silty clay subgrade to the proposed underside of footing elevation. For predictable performance of the proposed foundations, the excavations for the foundations should be taken through any fill materials, topsoil or otherwise deleterious material to expose the native, undisturbed silty clay.

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 120 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 silty clay or on a suitably constructed engineered pad placed on the silty clay. 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 subgrade surface should be inspected and approved by geotechnical personnel. It is expected that the subgrade, beneath the fill materials consists of native undisturbed silty clay.

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.

Excavations

Any excavation for the proposed structures will likely be carried out through fill material to bear on the native silty clay subgrade. The sides of the excavation should be sloped in accordance with the requirements of Ontario Regulation 213/91, s. 226 under the Occupational Health and Safety Act. According to the Act, the native soils at the site can be classified as Type 2 soil, however this classification should be confirmed by qualified individuals as the site is excavated and if necessary, adjusted.

It is expected that the side slopes of the excavation will be stable in the short term provided the walls are sloped at 1H:1V through the fill materials to 1.2 metres or less from the bottom of the excavation and provided no excavated materials are stockpiled within 3 metres of the top of the excavation.

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

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

The proposed building will not have a full basement. The depth of excavation will be limited to the lesser of the depth required to provide frost protection or to completely remove all fill materials. The proposed building is set back from the side property lines by 1.2 metres. Typically excavations are advanced with 1 metre offsets from the proposed foundation wall so to allow forming. As such the base of the excavation is expected to extent to about 0.2 metres from the property lines.

The closest projections of the adjacent building are located about 1.2 metres south and 2.3 metres north of the property lines. Based on the available information, the underside of footing elevation for the building to the south of the site is at about 87.3 metres and the underside of footing elevation for the building to the north of the site at about 86.3 metres. It is expected that the proposed underside of footing elevation will be at about 87.0 metres. Based on the difference in underside of footing elevation, the proposed excavation will not compromise the lateral support for the adjacent foundations nor will the adjacent foundations be within the zone of lateral support for the proposed foundation.

It is considered however that the side slopes of the excavation will not be achievable without encroaching on to the adjacent properties. Permission for encroachment should be obtained.

Where permission is not given or encroachment is not possible, the side slope of the excavation will have to be shored.

The shoring should be designed by a 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 potential surcharge acting on the ground surface adjacent to the shoring including expected vehicular loads

The hydrostatic pressure, γ_w H, may be neglected where soldier piles and timber lagging are used as drainage is expected to occur between the lagging and thus no build-up of hydrostatic pressure is likely. A surcharge load of 5 kPa at the ground surface can be used.

Engineered Fill

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 commercial 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 foundations, the engineered fill should extend out from the outside edges of the footings for a horizontal distance of 0.5 metres and then down and out at a slope of 1 horizontal to 1 vertical, or flatter. The excavations for the structure should be sized to accommodate this fill placement.

The excavations within the silty clay above any groundwater level should not present any serious constraints.

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.

The subgrade surface should be inspected by geotechnical personnel prior to the placement of engineered fill material and concrete. Field density testing should be carried out on the engineered fill during 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.

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

Frost Protection Requirements for Spread Footing Foundations

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

The depth of frost cover could be reduced for footings bearing on engineered fill over silty clay. 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.

Slab on Grade Support

For predictable performance of the proposed concrete floor slab all existing fill material, topsoil and any otherwise deleterious material should be removed from below the proposed floor slab area. The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any soft areas evident should be subexcavated and replaced with suitable engineered fill. Any fill materials consisting of granular material, removed from the proposed concrete floor slab area, could be stockpiled for possible reuse with approval from the geotechnical engineer.

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

The slab should be structurally independent from walls and columns, which are supported by the foundations. This is to reduce any structural distress that may occur as a result of differential soil movement. If it is intended to place any internal non-load bearing partitions directly on the slab-on-grade, such walls should also be structurally independent from other elements of the building founded on the conventional foundation system so that some relative vertical movement between the floor slab and foundation can occur freely.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage of the concrete. The saw cut depth should be about one quarter of the thickness of the slab. The crack control cuts should be placed at a grid spacing not exceeding the lesser of 25 times the slab thickness or 4.5 metres. The slab should be cut as soon as it is possible to work on the slab without damaging the surface of the slab.

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.

Foundation Wall Backfill and Drainage

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

Where the granular backfill will ultimately support a pavement structure or walkway, it is suggested that the wall backfill material be compacted in 250 millimetre thick lifts to 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.

Provided everywhere the proposed finished floor surfaces are everywhere above the exterior finished grade and provided the exterior grade is adequately sloped away from the proposed building, no perimeter foundation drainage system is required.

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

Seismic Design for the Proposed Commercial Building

For seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class B Rock. The subsurface conditions below the proposed footing design level consists of a thin veneer of stiff to very stiff silty clay over bedrock at a depth of about 1.9 to 2.9 metres below the ground surface. There will be between 0.5 and 1.5 metres of overburden between the underside of footing and the bedrock. Based on the results of the bore hole investigation, the bedrock is sound below the bedrock surface. The bedrock is expected to consist of either sandstone, limestone or dolostone.

National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.279 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The results of the test are attached following the text of this report.

Potential for Soil Liquefaction

The proposed building will be founded on a relatively thin layer of stiff silty clay or on engineered fill placed on the native stiff to very stiff silty clay overlying bedrock. Since the silty clay has a clay content of 24 percent which is much greater than 15 percent, it is considered to be not susceptible to liquefaction. The underlying bedrock is also not susceptible. Therefore, it is considered that no damage to the proposed commercial building should occur due to liquefaction of the native subgrade under seismic conditions.

Dewatering of Foundation Excavation

Bedrock was encountered at about 1.9 to 2.9 metres below the existing ground surface. Groundwater was not encountered within the boreholes at the time of the geotechnical investigation.

It is expected that the excavation for the proposed building will be extended to within the native silty clay above any suspected ground water level. Adjacent buildings will be founded either on bedrock, engineered fill or on a relatively thin overburden layer above the bedrock above the ground water level.

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

Since the groundwater level is presently lower than the underside of the surface soils, no additional dewatering of the surface soils will occur. If the groundwater level rises into the surface soils, due to significant rainfall or snow melt, dewatering will just lower the water level to pre-existing conditions. There are no settlement concerns to the adjacent dwellings and other buildings due to groundwater removal from the foundation excavation or excavation for services at this site.

SITE SERVICES

Excavation

The excavations for the site services will be carried out through a combination of fill materials, silty clay and/or bedrock. The sides of the excavation should be sloped in accordance with the requirements of Ontario Regulation 213/91, s. 226 under the Occupational Health and Safety Act. According to the Act, the native soils at the site can be classified as Type 2 soil, however this classification should be confirmed by qualified individuals as the site is excavated and if necessary, adjusted.

It is expected that the side slopes of the excavation will be stable in the short term provided the walls are sloped at 1H:1V through the fill materials to 1.2 metres or less from the bottom of the excavation and provided no excavated materials are stockpiled within 3 metres of the top of the excavation.

Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box. Excavation walls within bedrock may be made near vertical.

Any groundwater inflow into the service trenches should be handled by pumping from sumps from within the excavations.

It is expected that bedrock may be encountered within the excavations for the site services. Small amounts of bedrock removal, can most likely be carried out by hoe ramming and heavy excavating equipment. It is considered that where large amounts of bedrock are removed by hoe ramming, the hoe ramming could also introduce significant vibrations through the bedrock. A such it is considered that pre-excavation surveys of nearby structures and existing utilities should be completed before hoe ramming.

Pipe Bedding and Cover Materials

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

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

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

Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches. It should be possible to re-use the native silty clay obtained from above the groundwater level as trench backfill above the cover material.

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

Where native backfill is used, it should match the 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 driveway areas. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I. If the native material is not suitable for backfill, imported granular material may have to be used. If imported granular materials are used, suitable frost tapers should be used OPSD 802.013.

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

ACCESS ROADWAY AND PARKING LOT PAVEMENTS

In preparation for pavement construction at this site the existing asphaltic concrete, crushed stone, fill materials and any soft, wet or deleterious materials should be removed from the existing access roadway and parking lot to the elevation of the proposed top of subgrade based on the proposed final grading plan and proposed pavement structure thickness.

It is understood that this may result in the proposed pavement structure being placed on existing fill materials. It is considered that the existing fill materials, when free of significant organic, soft or deleterious materials, are adequate to support the proposed pavement structure.

The exposed sub-grade should be inspected and approved by geotechnical personnel and any soft areas evident should be sub-excavated and replaced with suitable earth borrow approved by the geotechnical engineer. The sub-grade should be shaped and crowned to promote drainage of the roadway area granular. Following inspection and 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 access roadway and parking lot area 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 hot mix 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/sqy) 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 and/or incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material. The adequacy of the design of the pavement thickness should be assessed by the geotechnical personnel at the time of construction.

CONSTRUCTION CONSIDERATIONS

It is suggested that the final design drawings for the project, including the proposed site grading plan, be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended 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, 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 footing areas and any engineered fill areas for the proposed commercial 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 roadways and driveway 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 above the bedrock at this site will be sensitive to disturbance from construction operations, from rainwater or snow melt, and frost. In order to minimize disturbance, construction traffic operating directly on the subgrade should be kept to an absolute minimum and the subgrade should be protected from below freezing temperatures.

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

Regards,
Kollaard Associates Inc.

LICENSED PROFESSIONAL ENGINEER
June 11, 2019
S.E. deWit
100079612
PROVINCE OF ONTARIO

Dean Tataryn, B.E.S., EP. Steve DeWit, P.Eng.

Attachments: Record of Boreholes
List of Abbreviations
Key Plan - Figure 1
Site Plan - Figure 2

APPENDIX A – SUMMARY OF GEOTECHNICAL RECOMMENDATIONS

This report provides geotechnical recommendations under the Headings: Geotechnical Guidelines and Recommendations; Foundation For Proposed Commercial Building; Site Services; Access Roadway Pavements; Construction Considerations:

These geotechnical recommendations include:

Potential for Corrosivity and Sulphate Attach
Foundation Design
Allowable Bearing Capacity
Settlement
Maximum Grade Raise
Excavation for Foundation
Effect of Foundation excavation on adjacent structures and City Services
Subgrade preparation
Engineered Fill and Compaction
Frost Protection
Foundation Drainage
Foundation Backfill
Floor Slab
Seismic Design
Excavation for Services and Sewers
Bedding and Cover
Trench Backfill
Subgrade Preparation for Pavements
Pavement Structures
Pavement Placement and compaction
Inspection Requirements.

RECORD OF BOREHOLE BH1

PROJECT: Proposed Mixed Development
CLIENT: Royal Lepage Team Realty
LOCATION: 5536 Manotick Main Street, Manotick, City of Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 190117
DATE OF BORING: March 18, 2019
SHEET 1 of 1
DATUM: GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	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		89.30							
0	ASPHALTIC CONCRETE		0.00							
	Grey crushed stone (FILL)		88.90 0.40	1	SS	39				
	Yellow brown sand and gravel, trace of asphalt and topsoil (FILL)									
1	Grey brown silty clay, trace sand and organics (FILL)			2	SS	9				
	Grey brown SILTY CLAY		87.60 1.70	3	SS	9				
2				4	SS	3				Borehole dry at time of drilling, March 18, 2019.
3	End of Borehole, Refusal on BEDROCK		86.36 2.94							
4										
5										
6										
7										
8										

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 Mixed Development
CLIENT: Royal Lepage Team Realty
LOCATION: 5536 Manotick Main Street, Manotick, City of Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 190117
DATE OF BORING: March 18, 2019
SHEET 1 of 1
DATUM: GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	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		88.80							
0	Grey crushed stone (FILL)		0.00							
	Grey brown silty clay, trace sand and organics (FILL)		88.50 0.30	1	SS	13				
1				2	SS	15				
	Grey brown SILTY CLAY		87.20 1.60	3	SS	9				
2				4	SS	50				
	End of Borehole, Refusal on BEDROCK		86.32 2.48							Borehole dry at time of drilling, March 18, 2019.
3										
4										
5										
6										
7										
8										

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 Mixed Development
CLIENT: Royal Lepage Team Realty
LOCATION: 5536 Manotick Main Street, Manotick, City of Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 190117
DATE OF BORING: March 18, 2019
SHEET 1 of 1
DATUM: GEODETIC

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	87.80							
0	Grey crushed stone (FILL)	0.00	1	SS	24				
		87.35							
	Yellow brown silty clay, trace sand and organics (FILL)	0.45							
1	Grey brown silty clay, trace sand and topsoil (FILL)		2	SS	6				
		86.15							
	Grey brown SILTY CLAY	1.65	3	SS	33				
		85.85							
2	End of Borehole, Refusal on BEDROCK	1.95							
3									Borehole dry at time of drilling, March 18, 2019.
4									
5									
6									
7									
8									

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

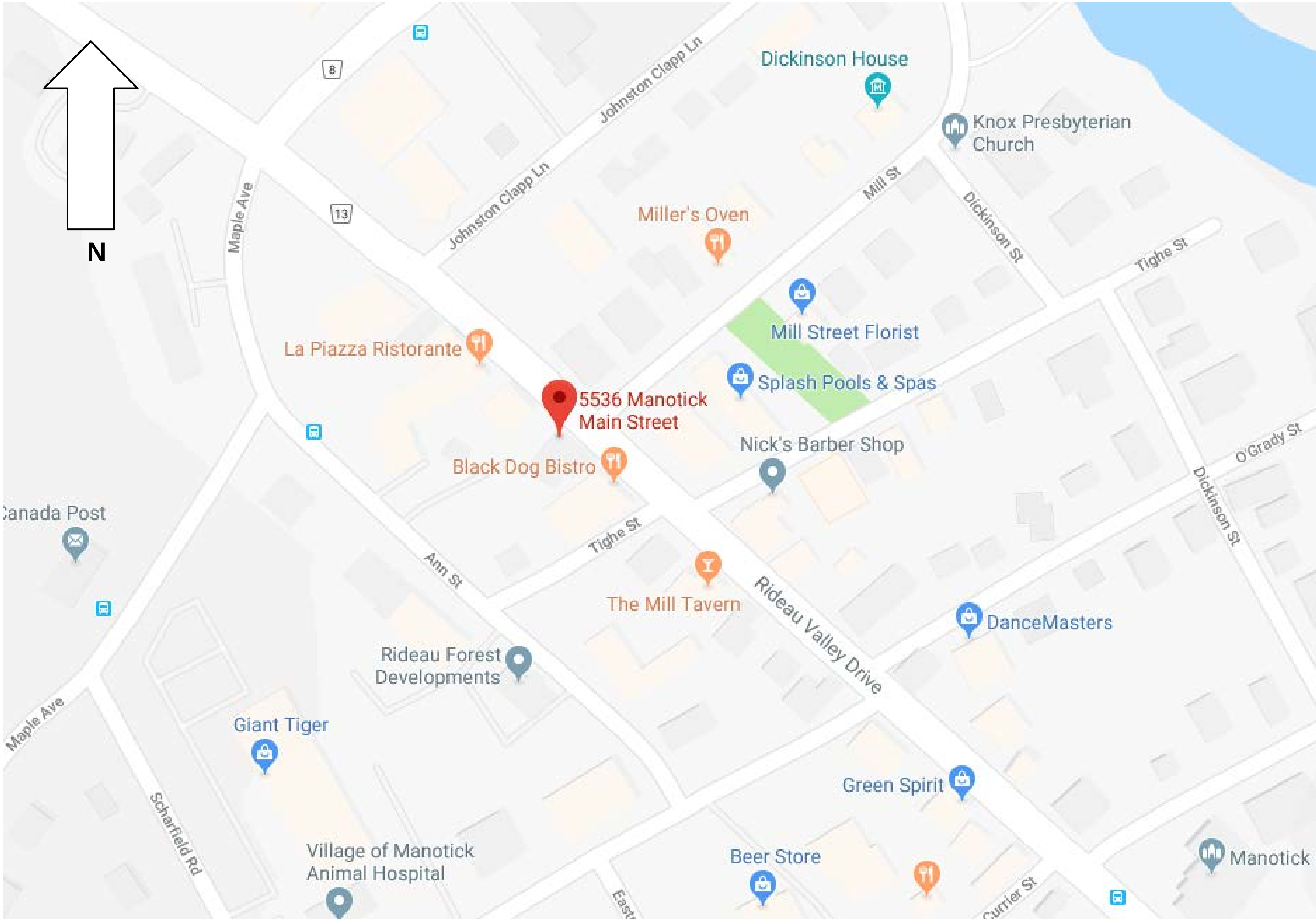
Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

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

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

KEY PLAN	FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 190117

Date June 2019

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

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:
ROYAL LEPAGE TEAM REALTY

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED MIXED USE BUILDING

LOCATION:
5536 MANOTICK MAIN STREET
MANOTICK
CITY OF OTTAWA, ONTARIO

DESIGNED BY: --	DATE: JUNE 2019
DRAWN BY: DT	SCALE: N.T.S

KOLLAARD FILE NUMBER:
190117

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: 20-MAR-19
Report Date: 27-MAR-19 14:51 (MT)
Version: FINAL

Client Phone: 613-860-0923

Certificate of Analysis

Lab Work Order #: L2247277

Project P.O. #: NOT SUBMITTED
Job Reference: 190117
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
L2247277-1 BH2 SS3 5'-7' Sampled By: CLIENT on 18-MAR-19 Matrix: SOIL							
Physical Tests							
Conductivity	0.785		0.0040	mS/cm		27-MAR-19	R4585302
% Moisture	27.2		0.10	%	22-MAR-19	23-MAR-19	R4580098
pH	7.35		0.10	pH units		25-MAR-19	R4580950
Resistivity	1270		1.0	ohm*cm		27-MAR-19	
Leachable Anions & Nutrients							
Chloride	0.0241		0.00050	%	25-MAR-19	25-MAR-19	R4584727
Anions and Nutrients							
Sulphate	0.0245		0.0020	%	22-MAR-19	25-MAR-19	R4584727

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

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-R511-WT	Soil	Chloride-O.Reg 153/04 (July 2011)	EPA 300.0

5 grams of dried soil is mixed with 10 grams of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

EC-WT	Soil	Conductivity (EC)	MOEE E3138

A representative subsample is tumbled with de-ionized (DI) water. The ratio of water to soil is 2:1 v/w. After tumbling the sample is then analyzed by a conductivity meter.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

MOISTURE-WT	Soil	% Moisture	CCME PHC in Soil - Tier 1 (mod)
PH-WT	Soil	pH	MOEE E3137A

A minimum 10g portion of the sample is extracted with 20mL of 0.01M calcium chloride solution by shaking for at least 30 minutes. The aqueous layer is separated from the soil and then analyzed using a pH meter and electrode.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	APHA 2510 B

Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	MOECC E3138

Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.

SO4-WT	Soil	Sulphate	EPA 300.0

5 grams of soil is mixed with 50 mL of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Chain of Custody Numbers:

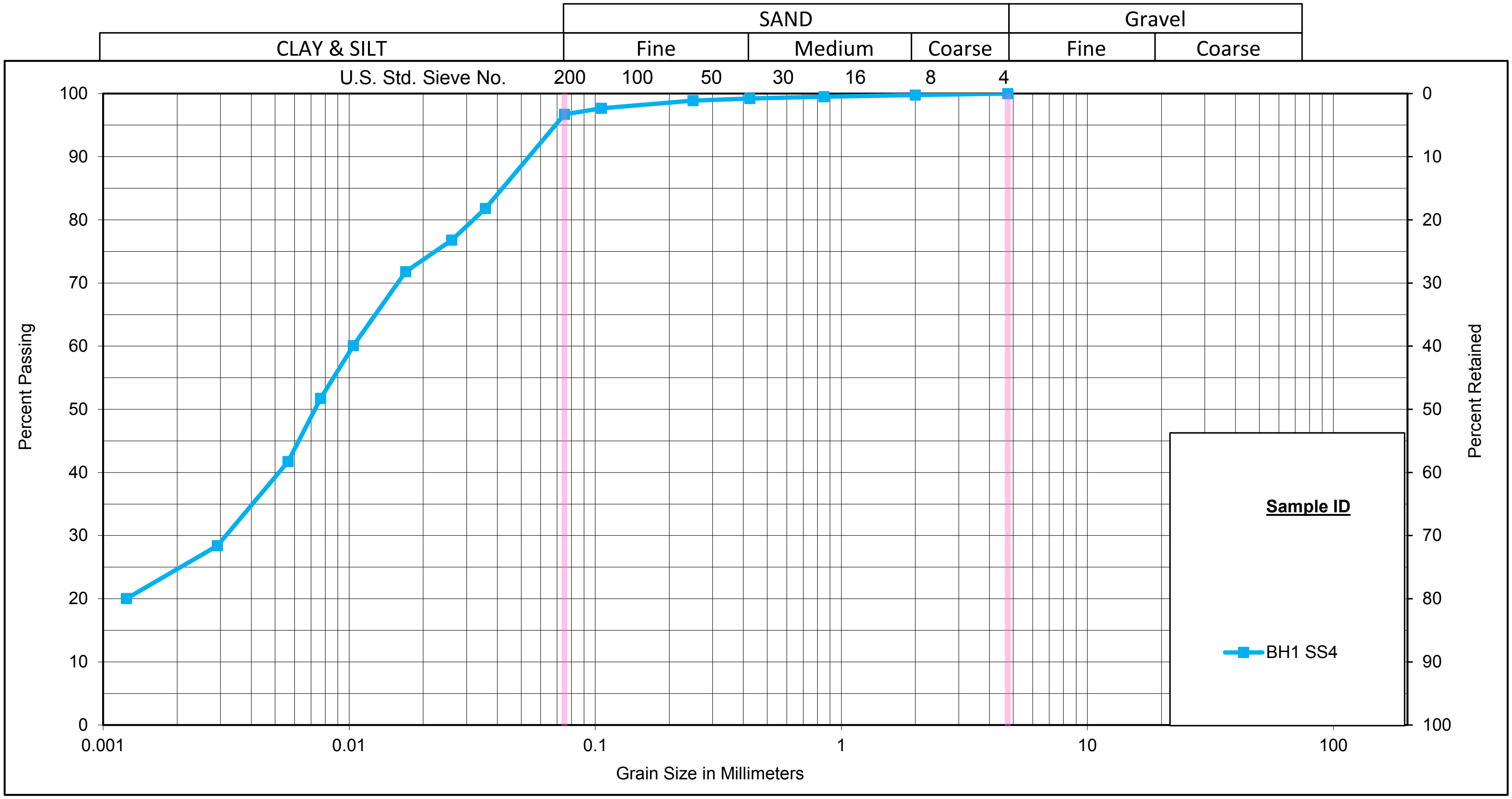
GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid weight of sample
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Laboratory Test Results for Physical Properties

Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH1 SS4	7'6"-9'6"	0.0	3.3	72.7	24.0

Stantec

GRAIN SIZE DISTRIBUTION

Kollaard Associates Engineers, File # 190117
5536 Manotick Main St.

Figure No.

Project No. 122410003

Stantec

Particle-Size Analysis of Soils

LS702

ASTM D422

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File # 190117	Project No.:	122410003
Project:	5536 Manotick Main St.	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH1	Date Sampled:	March 19, 2019
Sample No.:	SS4	Tested By:	Denis Rodriguez
Sample Depth	7'6"-9'6"	Date Tested:	March 26, 2019

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	58.47
Sample Weight after Hydrometer and Wash (g)	1.79
Percent Passing No. 200 Sieve (%)	96.9
Percent Passing Corrected (%)	96.70

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	165.00
Sample Weight After Sieve (g)	164.00
Percent Loss in Sieve (%)	0.61

SOIL INFORMATION

Liquid Limit (LL)		
Plasticity Index (PI)		
Soil Classification		
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	40	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	49.12
Air Dried Mass (W_a), (g)	49.66
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9891
Air Dried Mass in Analysis (M_a), (g)	59.11
Oven Dried Mass in Analysis (M_o), (g)	58.47
Percent Passing 2.0 mm Sieve (P_{10}), (%)	99.76
Sample Represented (W), (g)	58.61

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		100.0
19.0		100.0
13.2		100.0
9.5		100.0
4.75	0.0	100.0
2.00	0.4	99.8
Total (C + F)[1]	164.00	
0.850	0.15	99.50
0.425	0.32	99.21
0.250	0.52	98.87
0.106	1.23	97.66
0.075	1.78	96.72
PAN	1.79	

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)	27.25
Meniscus Correction (H_m), (g/L)	1.0

START TIME 10:59 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
26-Mar-19	11:00 AM	1	56.0	7.0	21.5	49.0	81.80	7.53404	9.73081	0.013047	0.03581
26-Mar-19	11:01 AM	2	53.0	7.0	21.5	46.0	76.79	7.99904	9.73081	0.013047	0.02609
26-Mar-19	11:04 AM	5	50.0	7.0	21.5	43.0	71.78	8.46404	9.73081	0.013047	0.01698
26-Mar-19	11:14 AM	15	43.0	7.0	21.5	36.0	60.10	9.54904	9.73081	0.013047	0.01041
26-Mar-19	11:29 AM	30	38.0	7.0	21.5	31.0	51.75	10.32404	9.73081	0.013047	0.00765
26-Mar-19	11:59 AM	60	32.0	7.0	21.5	25.0	41.73	11.25404	9.73081	0.013047	0.00565
26-Mar-19	3:09 PM	250	24.0	7.0	21.5	17.0	28.3783	12.49404	9.73081	0.013047	0.00292
27-Mar-19	10:59 AM	1440	19.0	7.0	22.0	12.0	20.0317	13.26904	9.61570	0.012970	0.00125

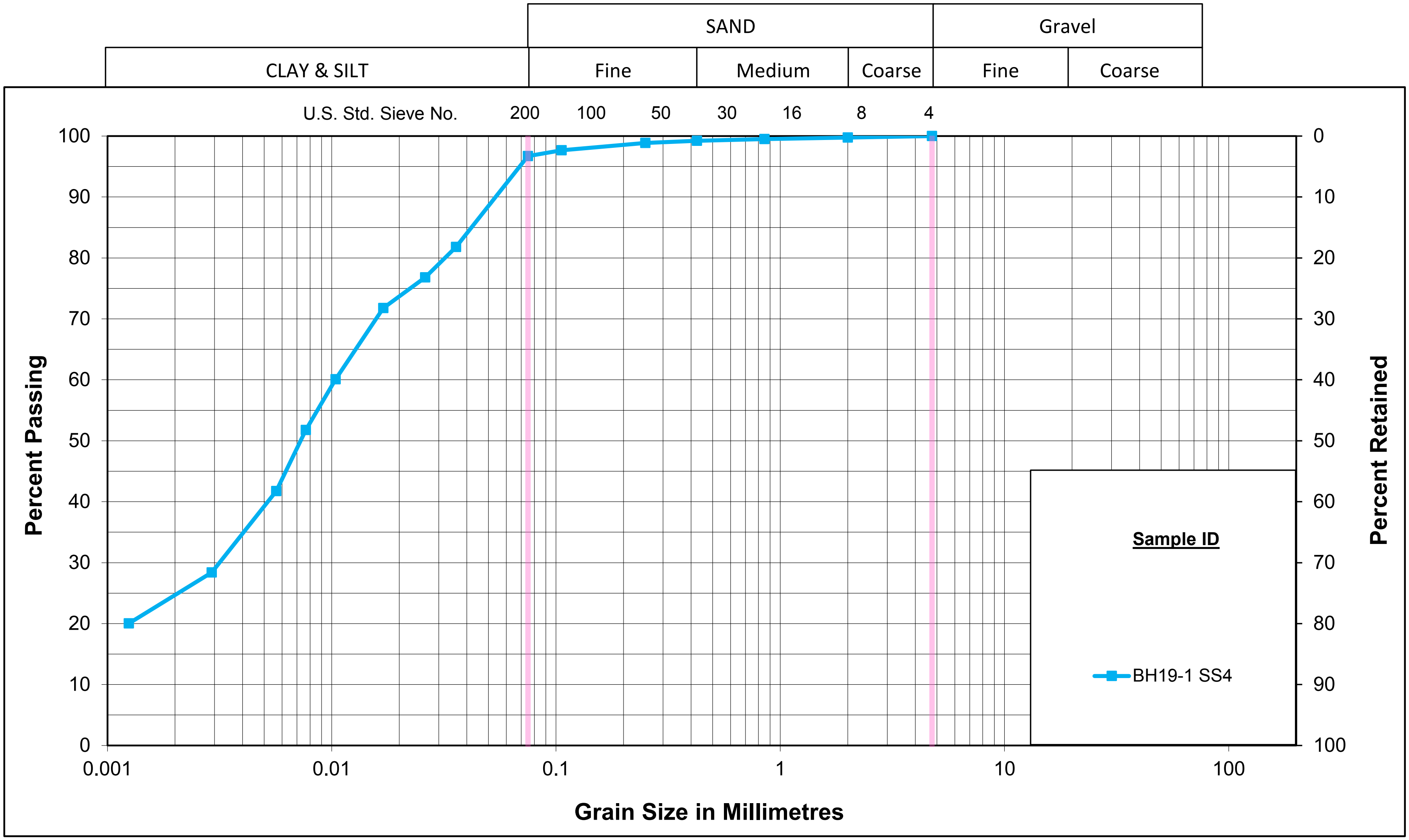
Remarks:

Reviewed By: Denis Rodriguez

Date: March 29, 2019

V:\01216\active\laboratory_standing_offers\2019 Laboratory Standing Offers\122410003 Kollaard Associates Engineers\March 19, One MC & Hydro, File # 190117\Hydrometer Analysis MTO Projects May2014.xlsx

Unified Soil Classification System



Stantec	GRAIN SIZE DISTRIBUTION Kollaard Associates Engineers, File # 190117 5536 Manotick Main St.	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 Engineers, File # 190117	Project No.:	122410003
Project:	5536 Manotick Main St.	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH19-1	Date Sampled:	March 19, 2019
Sample No.:	SS4	Tested By:	Denis Rodriguez
Sample Depth:	7'6"-9'6"	Date Tested:	March 26, 2019

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	58.47
Sample Weight after Hydrometer and Wash (g)	1.79
Percent Passing No. 200 Sieve (%)	96.9
Percent Passing Corrected (%)	96.70

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	165.00
Sample Weight After Sieve (g)	164.40
Percent Loss in Sieve (%)	0.36

SOIL INFORMATION

Liquid Limit (LL)		
Plasticity Index (PI)		
Soil Classification		
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	40	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	49.12
Air Dried Mass (W_a), (g)	49.66
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Air Dried Mass in Analysis (M_a), (g)	59.11
Oven Dried Mass in Analysis (M_o), (g)	58.47
Percent Passing 2.0 mm Sieve (P_{10}), (%)	99.76
Sample Represented (W), (g)	58.61

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		100.0
19.0		100.0
13.2		100.0
9.5		100.0
4.75	0.0	100.0
2.00	0.4	99.8
Total (C + F)[1]	164.40	
0.850	0.15	99.50
0.425	0.32	99.21
0.250	0.52	98.87
0.106	1.23	97.66
0.075	1.78	96.72
PAN	1.79	

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 10:59 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
26-Mar-19	11:00 AM	1	56.0	7.0	21.5	49.0	81.80	7.53191	9.7308	0.0130	0.0358
26-Mar-19	11:01 AM	2	53.0	7.0	21.5	46.0	76.79	7.99691	9.7308	0.0130	0.0261
26-Mar-19	11:04 AM	5	50.0	7.0	21.5	43.0	71.78	8.46191	9.7308	0.0130	0.0170
26-Mar-19	11:14 AM	15	43.0	7.0	21.5	36.0	60.10	9.54691	9.7308	0.0130	0.0104
26-Mar-19	11:29 AM	30	38.0	7.0	21.5	31.0	51.75	10.32191	9.7308	0.0130	0.0077
26-Mar-19	11:59 AM	60	32.0	7.0	21.5	25.0	41.73	11.25191	9.7308	0.0130	0.0057
26-Mar-19	3:09 PM	250	24.0	7.0	21.5	17.0	28.38	12.49191	9.7308	0.0130	0.0029
27-Mar-19	10:59 AM	1440	19.0	7.0	22.0	12.0	20.03	13.26691	9.6157	0.0130	0.0012

Remarks:

Reviewed By: Denis Rodriguez

Date: March 29, 2019

ATTACHMENT A

National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

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

Site: 45.22536N 75.685102W **User File Reference:** 5536 Manotick Main Street, Manotick, Ontario 2019-03-26 15:09 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.447	0.243	0.144	0.041
Sa (0.1)	0.522	0.295	0.181	0.057
Sa (0.2)	0.437	0.251	0.157	0.052
Sa (0.3)	0.331	0.192	0.121	0.042
Sa (0.5)	0.234	0.136	0.086	0.030
Sa (1.0)	0.116	0.068	0.044	0.015
Sa (2.0)	0.055	0.032	0.020	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.279	0.160	0.099	0.031
PGV (m/s)	0.194	0.109	0.066	0.021

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

References

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

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

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

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

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