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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL BUILDING
2726 MOODIE DRIVE
CITY OF OTTAWA, ONTARIO

Project # 221099

Submitted to:

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DISTRIBUTION

1000198532 Ontario Inc.
City of Ottawa
Kollaard Associates Inc.

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EXECUTIVE SUMMARY

Kollaard Associates Inc. (Kollaard) is pleased to present the results of the geotechnical investigation completed for the proposed Industrial Warehouse Development to be located at 2726 Moodie Drive, City of Ottawa Ontario.

The geotechnical investigation was completed in conjunction with a stormwater management plan report as well as civil engineering drawings which are presented under separate covers.

The Conceptual Site Plan prepared by Alexander Wilson Architect Inc. (Architect) Rev 1 dated April 22, 2025 indicates that the proposed development will consist of 5 large warehouse buildings ranging in size from about 3690 square metres to 4647 square metres. The development will occupy an about 6.6 hectare (16.3 acres), irregular shaped property located southwest of the intersection of Moodie Drive and Fallowfield Road in the City of Ottawa, Ontario. The development will be serviced by asphalt surfaced roadways and parking areas. It is understood that the proposed buildings will be of steel frame construction with flat roofs and interior columns. It is understood that each building will be structurally divided by a firewall located at the approximate midpoint of the building.

Based on a review of the surficial geology map for the site area, it is expected that the site is generally underlain by coarse textured glaciomarine deposits consisting of sand, gravel, minor silt and clay (Glacial Till) overlying bedrock. As such, the subsurface investigation was completed by boreholes in keeping with Section 2.3 of the Geotechnical Investigation and Reporting Guidelines for the City of Ottawa.

The fieldwork for this subsurface investigation was carried on January 30 and February 2, 2023, at which time ten (10) boreholes numbered BH1 to BH10 were put down at the site using a track mounted drill rig equipped with a hollow stem auger owned and operated by CCC Environment and Geotechnical Drilling of Ottawa. Additionally, six test pits numbered TP1 to TP6 were put down on April 14, 2023 using a track mounted excavator owned and operated by a local excavation contractor. The field work was supervised on a full time basis by Kollaard. The boreholes revealed that the subsurface conditions are, in general, comprised of a layer of topsoil followed by glacial till, then bedrock. A relatively thin layer of fill materials was encountered at Borehole BH1. The test pits also encountered topsoil followed by glacial till overlying bedrock, with a thin layer of fill materials at the surface in test pits TP4 and TP6. Additional field work was completed on November 28, 2023 during which time permeability testing was completed on the upper glacial till soils at the site using a Guelph Premeameter to determine their hydraulic conductivity.

It is noted that the location and number of buildings has been revised since the field work was completed in 2023. However, due to the underlying subsurface conditions, it is considered that the borehole spacing across the site is appropriate to obtain the necessary geotechnical information required provide geotechnical recommendations for the current proposed development. **In addition, t**est pits TP1 to TP6 were put down to confirm the subsurface conditions in the areas not encompassed by the boreholes and to confirm depth to bedrock

For the purposes of this report, Moodie Drive is considered to be oriented along a north south axis and is located along the east side of the site. In general, the ground surface is higher at about the center of the site and slopes downward towards Moodie Drive and towards the southwest corner of the site. The ground surface elevations range across the site from about 118.10 metres to about 114.75 metres. The underlying bedrock surface varies from about 111.0 to 115.9 metres at the borehole and test pit locations. Ground water was encountered at depths of between 0.8 and 2.3 metres below the existing ground surface (elevations of between about 113.7 and 116.6 metres) across the site. Perched groundwater was encountered at the surface of the glacial till during the Guelph Permeameter testing.

Based on the findings of the subsurface investigation, there is no sensitive marine clay deposits present at the site or other subsurface geotechnical conditions that would preclude normal construction practices. Thin soils or karst topography is not present at the site. There is no potential that the development of the site will cause adverse effects or aggravate a hazard either on site or elsewhere from a geotechnical perspective.

A Multichannel Analysis of Surface Waves (MASW) test was performed for the purpose of Seismic Site Classification by WSP Canada Inc. personnel on December 21, 2022. Based on the results of the MASW test, the site has been classified as seismic site Class A. It noted that, in order for a seismic site Class A to be used for the proposed development, the proposed underside of footing must be less than 3.0 metres from the bedrock surface. The on-site soils are not considered to be liquefiable during a seismic event.

The geotechnical investigation has revealed that conditions are suitable for the construction of the proposed industrial buildings on spread and strip footing foundations founded either directly on an approved glacial till subgrade or on engineered fill placed on an approved glacial till subgrade. Due to relatively loose density of the upper deposits of glacial till at some of the borehole and test pit locations, it is recommended that the subgrade be inspected and approved by Kollaard Associates. In general, the excavations should be advanced through the loose glacial till to a minimum of 0.6 metres below the underside of footing and an engineered pad should be placed on the approved subgrade surface. Footings prepared as per the geotechnical recommendations in the report may be designed using a serviceability limit state bearing pressure (SLS) of 200 kPa when founded on the engineered fill placed on the approved subgrade.

Based on the requirement to be less than 3.0 metres from the bedrock surface and on the presence of the relatively loose material, it is expected that the underside of footing level will vary across the building footprints. It is recommended that prior to the time of excavation, after the final building locations are staked in the field by an Ontario Land Surveyor, additional test holes be put down immediately outside of the building footprints to verify the bedrock surface elevation. The USF can be adjusted accordingly.

There are no geotechnical limitations with respect to the expected landscape grade raise at the site.

Excavation of bedrock or deep excavations are not expected at the site. As such, significant seepage of groundwater into the excavations is not expected. Surface water flowing into excavations during rainfall or snow melt events should controlled by redirecting surface drainage and by pumping.

The internal roadway and warehouse access surfaces should be built up using a heavy pavement structure consisting of 110 mm of asphaltic concrete underlain by 200 mm of OPSS Granular A base and 300 mm of OPSS Granular B Type II subbase. A biaxial Geogrid such as Terrafix Geosynthetics TBX2500 or equivalent, should be placed immediately below the sub-base on top of the geotextile fabric. A non-woven 6 ounce per square yard geotextile fabric should be placed between the native subgrade and the granular sub-base. The portions of the roadway and the vehicular parking area not subject to heavy traffic can consist of 50 mm of asphaltic concrete underlain by 150 mm of OPSS Granular A base and 300 mm of OPSS Granular B Type II subbase followed by a biaxial geogrid. A non-woven 6 ounce per square yard geotextile fabric should be placed between the native subgrade and the biaxial geogrid.

The above and other related considerations are discussed in greater detail in the main body of the report.

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RE: GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL WAREHOUSE DEVELOPMENT
2726 MOODIE DRIVE
CITY OF OTTAWA, ONTARIO

1 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the above noted proposed industrial warehouse development to be located at 2726 Moodie Drive, City of Ottawa, Ontario (see Key Plan, Figure 1).

The purpose of the investigation was to:

- Identify the subsurface conditions at the site by means of a limited number of boreholes;
- Based on the factual information obtained, provide recommendations and guidelines on the geotechnical engineering aspects of the project design; including bearing capacity and other construction considerations, which could influence design decisions.

2 BACKGROUND INFORMATION AND SITE GEOLOGY

2.1 Existing Conditions and Site Geology

The site was formerly occupied by a commercial greenhouse operation and a single family dwelling. The greenhouses and single family dwelling have since been demolished and removed. The site is bordered on the west and south by residential development and farmland, on the north by Fallowfield Road followed by vacant property and on the east by Moodie Drive followed by commercial development.

Based on a review of the surficial geology map for the site area, it is expected that the site is generally underlain by coarse textured glaciomarine deposits consisting of sand, gravel, minor silt and clay (Glacial Till) overlying bedrock. A review of the bedrock geology map indicates that the bedrock underlying the site consists of limestone, dolostone, shale, arkose and sandstone of the Ottawa Group, Simcoe Group and Shadow Lake Formation.

2.2 Proposed Development

The site consists of about a 6.6 hectare (16.3 acres), irregular shaped property located southwest of the intersection of Moodie Drive and Fallowfield Road in the City of Ottawa, Ontario (see Key Plan, Figure 1).

Based on information provided for the development, it is proposed to construct three industrial warehouse buildings. The proposed warehouse buildings will consist of the following:

- Building A: 3968 square metres, 15 units
- Building B: 4647 square metres, 17 units
- Building C: 4091 square metres, 15 units.
- Building D: 3690 square metres,
- Building E: 3690 square metres

The number of units in each building will vary from 14 to 17. There is the potential for units to be combined within each building which will increase the unit size and reduce the number of units. It is understood that combining units will not affect the interior column spacing. Each building will be divided by a structural fire wall located at about the midpoint of the length of the building.

Preliminary information provided by the client indicates that the proposed buildings will consist of single storey, steel frame metal clad structures. The proposed buildings will be placed on conventional concrete spread footing foundations with a concrete slab-on-grade construction (no basement). It is noted however, there is a potential that fire water storage or stormwater storage may be located in a cistern under the floor slab of one of the buildings. If a cistern is located below the building, the portion of the floor above the cistern will be constructed as a structural floor supported by columns. The interior layout of the buildings are not known at this time, however, it is understood the interiors will consist mostly of warehouse space along with some associated office spaces. The proposed buildings will be provided with an asphaltic concrete surfaced access roadway and parking lot.

The proposed development will be serviced by municipal water and by a private onsite septic system and a stormwater management facility.

3 PROCEDURE

The field work for this investigation was carried out on January 30, February 2 and April 14, 2023, at which time ten (10) boreholes numbered BH1 to BH10 were put down at the site using a track mounted drill rig equipped with a hollow stem auger owned and operated by CCC Environment and Geotechnical Drilling of Ottawa, Ontario and 6 test pits numbered TP1 to TP6 were put down using a track mounted excavator owned and operated by a local excavation contractor. The boreholes were put down within or in close proximity to the proposed building locations. It is noted that the location and number of buildings has been revised since the field work was completed in 2023. However, due to the underlying subsurface conditions, it is considered that the borehole spacing across the site is appropriate to obtain the necessary geotechnical information required to provide geotechnical recommendations for the current proposed development. Test pits TP1 to TP6 were put down to confirm the subsurface conditions in the areas not encompassed by the boreholes and to confirm depth to bedrock.

The subsurface soil conditions encountered at the test pits and boreholes were classified based on visual and tactile examination of the samples recovered (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), standard penetration tests (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils as well as laboratory test results on select samples. In situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil) was not carried out as cohesive materials were not encountered. The soils were classified using the Unified Soil Classification System. Groundwater conditions at the boreholes and test pits were noted at the time of drilling. Groundwater was measured at a later date in standpipes put down within the boreholes. The test pits and boreholes were loosely backfilled with the excavated materials and auger cuttings upon completion of the fieldwork.

Three soil samples (BH4 – SS4 – 2.3 – 2.9 m, BH8 – SS3 – 1.5 – 2.1 m & BH9 – SS5 – 3.0 – 3.6 m) were submitted for Particle Size Analysis. The samples were selected based on depth and tactile examination to be representative of the various soil conditions encountered at the site.

One sample of soil (BH2 – SS3 – 1.5 – 2.1 m) 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.

A total of 37 soil samples recovered from the boreholes were also tested for moisture content (ASTM D2216).

The field work was supervised throughout by a member of our engineering staff who located the boreholes and test pits in the field, logged the boreholes and cared for the samples obtained. A description of the subsurface conditions encountered at the boreholes and test pits is given in the attached Record of Borehole and Test Pits Sheets. The results of the laboratory testing of the soil samples are presented in the Laboratory Test Results section and Attachment B following the text in this report. The approximate locations of the boreholes and test pits are shown on the attached Site Plan, Figure 2. The ground surface elevation was obtained at each borehole and test pit location using GPS survey equipment and is referenced to the vertical geodetic datum of CGVD28:78.

4 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, a description of the subsurface conditions encountered at the boreholes is provided in the attached Record of Borehole and Test Pits Sheets following the text of this report. The borehole and test pit logs indicate the subsurface conditions at the specific hole 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 and test pit locations may vary from the conditions encountered at the boreholes and test pits.

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 with select samples being classified by laboratory testing. The soils were classified in the field based on visual and tactile inspection (ASTM D2488) and by the results of the standard penetration tests. 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 and test pit logs. Groundwater conditions may vary seasonally, or may be affected by construction activities on or in the vicinity of the site.

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

4.2 Fill

Fill materials consisting of topsoil and either yellow brown silty sand with some gravel or yellow brown sand and gravel with some silt were encountered from the surface at boreholes BH1 and BH3. Fill materials consisting of grey brown silty clay were encountered at test pits TP4 and TP6. The fill materials extended to a depth of about 0.5 to 0.6 metres at the borehole and test pit locations. The fill materials were fully penetrated at all locations where encountered.

4.3 Topsoil

About a 0.1 to 0.7 metre thickness of topsoil was encountered from the ground surface or below the fill materials at all of the test holes. The material was classified as topsoil based on the colour and the presence of organic materials. The identification of the topsoil layer is for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustainable plant growth.

4.4 Glacial Till

A deposit of red brown or yellow brown to grey brown silty sand with some gravel, cobbles, boulders and a trace of clay (glacial till) was encountered beneath the topsoil in all boreholes. The glacial till was encountered at depths ranging between 0.2 and 0.8 metres below the existing ground surface. The results of standard penetration tests completed in the glacial till gave N values of between 3 and 100 blows per 0.3 metres, indicating a very loose to very dense state of compaction. Boreholes BH5 to BH7 were terminated within the glacial till at a depth of about 1.8 metres below the existing ground surface. The glacial till was fully penetrated in boreholes BH1 to BH4 and boreholes BH8 to BH10, as well as in test pits TP1 to TP6. The glacial till was about 0.7 to 4.0 metres thick where fully penetrated.

The results of three hydrometer tests (ASTM D422 and D2216) on samples of soil (BH4 –SS4 – 2.3 – 2.9 m, BH8 – SS3 – 1.5 – 2.1 m & BH9 – SS5 – 3.0 – 3.6 m) indicate the samples have the following:

Sample	Depth(metres)	% Gravel	% Sand	% Silt	% Clay
BH4-SS4	2.3 – 2.9	10.5	42.4	38.1	9.0
BH8-SS3	1.5 – 2.1	7.3	54.7	30.0	8.0
BH9-SS5	3.0 – 3.6	16.8	46.3	30.9	6.0

The results are located in Attachment B.

4.5 Bedrock

Practical refusal on bedrock or large boulders was encountered in boreholes BH1 to BH4 and BH8 to BH10 as well as test pits TP1 to TP6 at depths between 1.5 and 4.2 metres.

4.6 Moisture Contents

A total of 37 soil samples were also tested for moisture content (ASTM D2216). The measured moisture contents of the soil samples ranged from 8 to 51 percent. The results of the moisture content are included on the Record of Borehole sheets following the text of this report.

4.7 Groundwater

Some groundwater was encountered in boreholes BH1 to BH4, BH6 and BH7 at the time of drilling on January 30, 2023, at depths ranging from about 0.9 to 1.8 metres below the existing ground surface. Boreholes BH5 and BH8 to BH10 were dry at the time of drilling on January 30 and February 2, 2023. Some groundwater was encountered at test pits TP1 to TP6 at depths between 1.0 to 1.5 metres below the existing ground surface at the time of excavation, April 14, 2023. Groundwater was measured in standpipes installed within boreholes BH8 and BH10 at depths of about 2.0 and 0.8 metres, respectively, below the existing ground surface on February 14, 2023.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as early spring. A peizometer was installed in borehole BH8 to monitor the groundwater level.

4.8 Corrosivity on Reinforcement and Sulphate Attack on Portland Cement

The results of the laboratory testing of a soil sample 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 %	0.00054	Negligible
pH	pH < 5.5	7.87	Negligible concern
Resistivity	R < 20,000 ohm-cm	11400	Mildly Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	<0.0020	Negligible concern

The results of the laboratory testing of a soil sample for sulphate gave a percent sulphate of less than 0.0020. 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 7.87, 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 non-corrosive with a resistivity of >20,000 ohm-cm as follows:

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

The soil resistivity was found to be 11400 ohm-cm for the sample analyzed making the soil mildly corrosive for buried steel. Increasing the specified strength and increasing concrete cover or increasing the specified strength and adding air entrainment into any reinforced concrete in contact with the soil is recommended. Additional special protection, other than listed above, is not required for reinforcement steel within the concrete foundation walls.

The laboratory results are presented in Attachment C following this report.

5 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the information from the 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.

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 foundations and the thickness of the soils deposit beneath the footings.

5.2 Foundations for Proposed Commercial Buildings

It is understood that the proposed commercial buildings will consist of conventional concrete spread footing foundations complete with cast-in-place concrete foundation walls and concrete slab-on-grade construction and no basements.

In order for a Seismic Site Classification of Site Class A to be utilized for the design of the proposed buildings, the USF must be less than 3.0 metres from the surface of the underlying bedrock. As such, it is expected that the underside of footing elevation will vary and the proposed cast-in-place concrete foundation walls may be taller than normally required to provide sufficient cover for frost protection purposes.

5.3 Foundation Excavation

As previously indicated, the subsurface conditions encountered at the boreholes advanced during the investigation consisted of fill materials (silty sand or sand and gravel) and/or topsoil overlying glacial till, followed by bedrock with depth.

The excavations for the foundation should be taken through any fill, topsoil or otherwise deleterious material to bear on the native, undisturbed red brown or yellow brown glacial till subgrade. The sides of the excavations 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 3 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 and native glacial till to the bottom of the excavation and provided no excavated materials are stockpiled within 3 metres of the top of the excavations.

5.4 Groundwater in Excavation and Construction Dewatering

Groundwater was measured in boreholes BH1 to BH4, BH6 and BH7 at the time of drilling on January 30 and February 2, 2023 at about 0.9 to 1.8 metres below the existing ground surface. Boreholes BH5 and BH8 to BH10 were dry at the time of drilling. Water was measured in standpipes placed within the boreholes BH8 and BH10 at about 2.0 and 0.8 metres, respectively, below the existing ground surface on February 14, 2023. It is considered that the groundwater encountered within the boreholes and test pits consists of surface infiltration trapped above the relatively impervious very dense glacial till and bedrock. It is expected that the proposed USF for the building foundations will be placed below the level at which water was encountered within the test holes.

The groundwater inflow from the native soils into the foundation excavations during construction, if any, should be handled by pumping from sumps within the excavations. The permeability of the compact to dense glacial till is relatively low. As such significant flow into the excavation from any elevated ground water is unlikely. There may be some inflow from groundwater perched between the topsoil and or fill layer and the upper layers of the glacial till during the seasonally wetter portion of the year. This inflow is also expected to be limited. As such it is expected that a permit to take water will not be necessary prior to the excavation. If groundwater is encountered, registration on the Enviromental Activity Sector Registry for construction dewatering as per O.Reg 63/15 is expected to be sufficient.

5.5 Subsurface Conditions at the Underside of Footing Level and USF Elevation

It is expected that the subgrade immediately below the proposed footing level will consist of glacial till. Once the excavations for the foundations are complete, the exposed subgrade should be inspected by a qualified geotechnical person. Should the subgrade consist of loose or loose to compact glacial till, the subgrade should be sub-excavated to remove the loose or loose to compact material to a depth of 0.6 metres below the underside of footing elevation.

The proposed underside of footing elevation for each section of foundation wall for each of the proposed buildings is provided on the attached drawing 221099-USF. As previously indicated, these elevations have been set to ensure that the footings are not greater than 3 metres above the underlying bedrock and that the footings will be founded below the depth of seasonal frost penetration.

5.6 Conventional Spread Footing Foundations

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

For the proposed commercial buildings, a maximum allowable bearing pressure of 200 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 400 kilopascals using ultimate limit states design, may be used for the design of conventional strip footings or pad footings founded on the dense glacial till or on a suitably constructed engineered pad placed on the compact to dense glacial till.

The maximum total and differential settlement of the footings are expected to be less than 25 millimetres and 20 millimetres, respectively, using the above allowable bearing pressure and resistance. There is no maximum grade raise associated with the above allowable bearing pressure.

The subgrade surface should be inspected and approved by geotechnical personnel prior to placement of any granulars.

5.7 Engineered Fill

Should the complete removal of all 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 building to founding level should consist of granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 300 millimetre thick loose lifts to 98 percent of the standard Proctor maximum dry density. It is considered that the engineered fill should be compacted using dynamic compaction with a large diameter vibratory steel drum roller or diesel plate compactor. If a diesel plate compactor is used, the lift thickness may need to be restricted to less than 200 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 first lift of engineered fill material should have a thickness of 300 millimetres in order to protect the subgrade during compaction. It is considered that the placement of a geotextile fabric between the engineered fill and the subgrade is not necessary where granular materials meeting the grading requirements for OPSS Granular A or Granular B Type I or Type II are placed on a glacial till subgrade above the normal ground water level.

Where the subgrade surface consists of glacial till below the water table, a 6 ounce per square yard nonwoven geotextile fabric should be placed between the engineered fill and the glacial till subgrade. The first lift of engineered fill should consist of coarse (106 mm or 4 inch) Granular B Type II material placed in a minimum lift thickness of 300 mm. This lift should be compacted to 95 percent of the standard Proctor maximum dry density using a large steel drum vibratory roller. All vibration should be stopped during compaction immediately if pumping starts to occur. It is recommended that trucks are not used to place the engineered fill on the subgrade. The fill should be dumped at the edge of the excavation and moved into place with a tracked bulldozer or excavator.

The native 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.

5.8 Frost Protection Requirements for Spread Footing Foundations

In general, 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.

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

5.9 Foundation Wall Drainage

5.9.1 Slab on Grade Construction

Provided the proposed finished floor surfaces are everywhere above the exterior finished grade, the granular materials beneath the proposed floor slab are properly compacted and provided the exterior grade is adequately sloped away from the proposed buildings, no perimeter foundation drainage system is required.

5.9.2 Structural Slab with Cistern

There is a potential that a cistern is to be constructed below the floor slab in one or more of the buildings for water storage for firefighting purposes or for stormwater storage.

Where the cistern is used for fire water storage, the cistern should be designed to resist any potential buoyancy. Perimeter and under slab drainage can be placed around the cistern walls and beneath the cistern floor slab. The drainage can discharge by gravity to the storm system. In this case, the maximum buoyancy forces can be assumed equal to the 100 year storage level of 116.75 m. Alternatively, no drainage system may be used and the buoyancy forces should be assumed equal to the exterior ground surface adjacent the building in question. The perimeter drain can consist of a conventional perforated weeping tile complete with a surround of 20 mm clear stone. The under slab drainage tile should be spaced at a maximum of 6 m on center. The weeping tile should be sock covered.

Where the cistern is used for stormwater storage it is suggested that the cistern floor consist of a minimum 6 ounce per square yard non-woven geotextile filter topped with a 0.3 m thick layer of 20 mm clearstone. If the cistern floor consists of non-woven geotextile topped with clear stone there will be no buoyancy forces and foundation drainage will not be required. The cistern will discharge by gravity to the storm system.

5.10 Foundation Wall Backfill

The native soils encountered at this site are considered to be 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 (such as Platon System Membrane) against the foundation wall. 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. To mitigate this potential, the upper approximately 0.6 metres of the foundation should be backfilled 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.

5.11 Slab on Grade Support

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

The fill materials beneath the proposed concrete floor slab on grades 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 slabs 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-grades, 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 slabs and foundations can occur freely.

The concrete floor slabs 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 slabs should be cut as soon as it is possible to work on the slabs without damaging the surface of the slabs.

5.12 Seismic Design for the Proposed Commercial Building

5.12.1 Seismic Site Classification (MASW Analysis)

On December 21, 2022, a multichannel analysis of surface waves (MASW) test was performed by Golder Associates Ltd. Personnel. Based on the results of the MASW test performed at the site, the average shear wave velocity (Vs30) in the upper soils is 2023 m/s. Part 4 of the Ontario Building Code provides the following information in sentence 4.1.8.4: If the average properties of the soils in the upper 30 meters have an average shear wave velocity of greater than 1500 m/s the Site Class is Site Class A. The separation distance between the rock and the footing must not be greater than 3.0 m in order to use a Site Class A.

As discussed in Section 5.2 of this report, the footings will be founded less than 3.0 metres from the underlying bedrock. Based on the shear wave velocities for this site and the separation distance between the footing and the bedrock, the Seismic Site Classification according to the 2012 Ontario Building Code Table 4.1.8.4A is Site Class A (Rock). The results of the analysis are included as Attachment A.

5.12.2 National Building Code Seismic Hazard Calculation

The online 2015 National Building Code Seismic Hazard Calculation was used to verify the seismic conditions at the site. The design Peak Ground Acceleration (PGA) for the site was calculated as 0.262 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The seismic site classification for the site is indicated to be Seismic Site Class A. The results of the calculation are attached in Attachment D following the text of this report.

5.12.3 Potential for Soil Liquefaction

As previously indicated, the soils below the proposed foundations will consist of glacial till overlying bedrock at about 1.5 to 4.2 metres below the existing ground surface. Soils of this nature and thickness are not considered to be susceptible to liquefaction under seismic conditions. As such there is no risk to the buildings at the site resulting from seismic liquefaction.

6 SITE SERVICES

6.1 Excavation

The excavations for the site services will be carried out through fill materials, topsoil 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 it is expected that some groundwater will be encountered during excavation. Permeability tests were completed in the upper soils at the site using a Guelph Permeameter. The results of the test are discussed in section 8.2 of this report. Due to the relatively low permeability of the glacial till and the varying depth at which the groundwater was encountered, it is considered that significant groundwater flow into any excavation is unlikely and a permit to take water will not be required. It is it is anticipated that there will be some groundwater inflow into the excavation for the site services and registration on the Environmental Activity Sector Registry (EASR) as per O.Reg. 63/16 is expected to be sufficient.

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

6.2 Pipe Bedding and Cover Materials

It is suggested that the service pipe bedding material consist of at least 150 millimetres of granular material meeting OPSS requirements for Granular A. A provisional allowance should, however, be made for 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.

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

6.3 Trench Backfill

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

In areas where the service trench will be located below or in close proximity to existing or future roadway or parking areas, acceptable native materials should be used as backfill between the roadway or parking area 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. 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 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.

7 ACCESS ROADWAY AND PARKING LOT PAVEMENTS

7.1 Subgrade Preparation

In preparation for pavement construction at this site any topsoil, fill materials, soft, wet or deleterious materials should be removed from the proposed access roadway and parking lot area. The exposed subgrade surface should then be proof inspected and approved by geotechnical personnel. It is considered that the subgrade should consist of glacial till. Any soft or unacceptable areas evident should be subexcavated and replaced with suitable earth borrow material. The subgrade should be shaped and crowned to promote drainage of the roadway and parking area granulars. Following approval of the preparation of the subgrade, the pavement granulars may be placed.

For any areas of the site that require the subgrade to be raised to proposed roadway and parking area subgrade level, the material used should consist of OPSS select subgrade material or OPSS Granular B Type I or Type II. Recycled asphaltic concrete or Portland cement concrete crushed to meet the grading requirements for OPSS Granular A or Granular B Type II may also be used to raise the site to the proposed roadway or parking area subgrade level. Materials used for raising the subgrade to proposed roadway and parking area subgrade level should be placed in maximum 300 millimetre thick loose lifts and be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

It is noted that the most earth borrow and select subgrade materials are sensitive to changes in water content, precipitation and frost heaving. As such, unless the fill material placement is planned during the dry periods of the year (June to August), precipitation and freezing conditions may prevent adequate compaction of these materials.

7.2 Parking Area and Roadway Structure

The subgrade of the proposed parking areas, access roads at the site will be comprised of glacial till, or approved material used to raise the grades to the proposed subgrade level. The following pavement structures for the proposed parking areas and roadways can be placed following approval of the subgrade surface by geotechnical personnel:

7.2.1 Light Duty Pavement

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
    (50 or 100 millimetre minus crushed stone)
Biaxial Geogrid such as Terrafix Geosynthetics TBX2500 or equivalent
Non-woven geotextile fabric (6 oz/sqy) such as Terrafix 360R or Thrace-Ling 150EX or approved alternative.

Performance grade PG 58-34 asphaltic concrete should be specified

7.2.2 Heavy Duty Pavement

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

50 millimetres of Superpave 12.5 hot mix asphaltic concrete over
60 millimetres of Superpave 19 hot mix asphaltic concrete over
200 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase
    (50 or 100 millimetre minus crushed stone)
Biaxial Geogrid such as Terrafix Geosynthetics TBX2500 or equivalent
Non-woven geotextile fabric (6 oz/sy) such as Terrafix 360R or Thrace-Ling 150EX or approved alternative.

Traffic Category D - Performance grade PG 64-34 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. Both the geogrid and the geotextile fabric should be installed with a minimum overlap of 0.3 m. The overlap location in the geogrid should not be at the same location as the overlap location of the geotextile fabric.

As discussed in section 8.1 below, the clearstone over the storm chambers used as part of the stormwater management facility will encroach on the pavement subbase. It is considered that the specified geogrid and geotextile fabric be placed over the clearstone cover over the chambers and that the subbase thickness be reduced accordingly.

The above pavement structures will be adequate on an acceptable subgrade, that is, one where any roadway fill has been adequately compacted. If the roadway subgrade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase. The locations where the proposed light duty pavement structure is intended to be used has been illustrated on Kollaard drawing 221099-USF (attached). All areas not intended as light duty should be paved with the heavy duty pavement structure.

7.3 Roadway Embankments

It is understood that the proposed roadway will require significant grade raise in relatively close proximity to the south property line of the site in order to achieve the desired grade of the proposed roadway and associated parking area.

As indicated above, the subgrade for the proposed roadway can be raised to the underside of the proposed subbase layer using native glacial till material or other approved material. This material should be placed in lifts and compacted to a minimum of 95% SPMDD. The temporary side slopes of the embankment, formed by the placement of this material, will be stable during construction provided they are not steeper than 1 horizontal to 1 vertical.

Final fill slopes of 3.0 horizontal to 1 vertical, or flatter, could be used for Granular B Type I, earth borrow, or Select Subgrade Material. Final slopes constructed with Granular B Type II could be constructed at about 2.5 horizontal to 1 vertical or flatter. Side slopes of 1.5 horizontal to 1 vertical, or flatter, could be used for embankments supported by a minimum thickness of 0.6 metres of well shattered and graded rock fill material. Fractured or crushed rock meeting the grading requirements for OPSS 1004 R-50 riprap with a maximum equivalent cube size of 26 cm could be used.

8 STORMWATER MANAGEMENT DESIGN CONSIDERATIONS

8.1 Underground Storage Chambers

The proposed stormwater management design includes a system of underground storage chambers which will be located under the roadway north of Buildings A, B and C and along both sides of Buildings E and F. The stormwater storage chambers should be designed to accommodate HS25 loading when installed in accordance with manufactures recommendations.

It is noted that the stormwater management design provided by Kollaard under separate report indicates that the proposed chambers will consist of both NDS SC-18 and NDS SC34E chambers. These chambers require a minimum 0.457 m cover with rigid pavement and 0.6 m for flexible pavement. Based on the design inverts, the cover will range from 0.61 to 1.16 m. As such the minimum cover will be achieved. The clearstone used as bedding, backfill and cover for the chambers should be well consolidated using the compaction equipment specified by the manufacturer.

The chambers require 0.23 m of 20 to 50 mm angular crushed washed clear stone below the chambers and 0.15 m of 20 to 50 mm angular crushed washed clear stone above the chambers. This 0.15 m of clearstone will interfere with the pavement structure clearstone above include Since the heavy duty pavement structure has total thickness of 0.61 m, the clearstone above the chambers will encroach on the pavement structures. Where the clearstone encroaches into the pavement structure, the subbase thickness can be reduced accordingly.

8.2 Infiltration – Permeameter Testing

The native soils below the topsoil layer consist of glacial till material as confirmed by particle size analysis completed on samples selected to be representative of the soils encountered at the site. As previously indicated the results of the particle size analysis are included in Attachment B of this report.

In-situ hydraulic conductivity testing was competed at the site using a Guelph Permeameter following the testing procedure specified for the Guelph Permeameter. During the testing procedure, hand auger holes were advanced to the depths at which the testing was completed. The soil conditions were observed in the hand auger holes verified that the soils encountered at the test locations were in keeping with the subsurface conditions expected based on the bore holes and test pits previously advanced.

The results of the testing and associated calculations are included as Attachment E following this report.

The results of the calculations based on the in-situ hydraulic conductivity tests gave a coefficient of permeability of between 2.0 $*10^{-7}$ and 2.2 $*10^{-7}$ cm/s.

The following table obtained from the Low Impact Development Stormwater Management Planning and Design Guide - Appendix C produced by Credit Valley Conservation and Toronto and Region Conservation indicates the relationship between the Percolation Time, Coefficient of Permeability and Infiltration Rate.

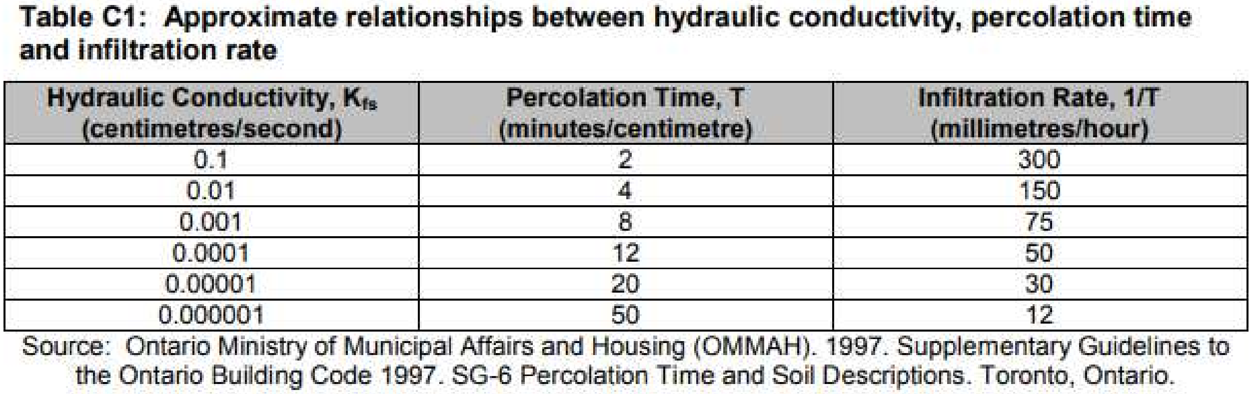


Table C1: Approximate relationships between hydraulic conductivity, percolation time and infiltration rate

Hydraulic Conductivity, K_{fs} (centimetres/second)	Percolation Time, T (minutes/centimetre)	Infiltration Rate, 1/T (millimetres/hour)
0.1	2	300
0.01	4	150
0.001	8	75
0.0001	12	50
0.00001	20	30
0.000001	50	12

Source: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.

From the above comparison, the existing soils within 1 metre of the bottom of the infiltration trenches would have an estimated infiltration rate of 12 millimetres/hour.

Based on these correlations, it is considered that a percolation time of T = 50 min/cm, a coefficient of permeability of K = $2x10^{-7}$ cm/sec and an infiltration rate of 0.12 cm/hr should be used to calculate infiltration when using low impact design for stormwater management.

8.3 Seasonably High Ground Water Level for LID design

As previously discussed, the groundwater was measured in boreholes BH1 to BH4, BH6 and BH7 at the time of drilling on January 30 and February 2, 2023 at about 0.9 to 1.8 metres below the existing ground surface. Boreholes BH5 and BH8 to BH10 were dry at the time of drilling. Water was measured in standpipes placed within the boreholes BH8 and BH10 at about 2.0 and 0.8 metres, respectively, below the existing ground surface on February 14, 2023.

Perched groundwater was encountered at the interface between the topsoil and glacial till at the time of the Permeameter testing It is expected that the perch groundwater resulted from the resent rainfall event an water being trapped on the relatively impervious glacial till layer below the topsoil Since this perched water could impact LID design, it is recommended that LID infiltration techniques do not be relied on at the site.

9 CONSTRUCTION CONSIDERATIONS

9.1 Protection of Subgrade and Temporary Construction Access Roads

The native 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.

Significant quantities of fill material will be required to raise the existing ground surface elevations to the proposed design grade. In addition, significant quantities of granular materials will be required for the road structure and for engineered fill within the building foot prints. The proposed heavy duty pavement structure when complete will be sufficient to accommodate the truck traffic necessary to bring in these materials. However, a partially completed pavement structure will not be sufficient. In addition, the existing ground surface and the existing native glacial till are not structurally adequate to accommodate any repeated truck loads required to bring fill onto the site.

In order to prevent damage to the subgrade during the import of grade raise fill materials and engineered fill materials, temporary construction access roads should be constructed. These temporary roads should have a minimum top width of 4 metres and should have a minimum granular thickness of 0.6 metres before subjected to any significant truck traffic. The side slopes of the temporary road can be constructed at 1H to !V. The granular material used to construct the temporary road can consist of OPSS Granular B Type II or any other granular material approved by the geotechnical engineer. The temporary road could be constructed by temporarily thickening the Granular B subbase on a portion of the access roadways and parking area. The temporary road thickness may have to be increased depending on the granular material used. The temporary road should be constructed on an approved subgrade. The temporary road should be maintained as required.

9.2 Review of Final Design Drawings

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.

9.3 Inspections

The engagement of the services of the geotechnical consultant during construction is required 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.

The engagement of the services of the geotechnical consultant during all stages of the placement of the storm chambers is required to ensure that the storm chambers are placed on an adequately prepared subgrade, the granular thickness below the storm chambers conforms to project design and that the backfill and cover has been placed in a manner that will support the roadways and parking areas above the storm chambers.

All foundation areas and any engineered fill areas for the proposed buildings 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 access roadway and parking areas should be inspected and approved by geotechnical personnel. The placement of the non-woven geotextile fabric and the geogrid should be verified prior to the placement of any granular material. In situ density testing should be carried out on the roadway and parking area granular materials to ensure the materials meet the specifications from a compaction point of view.

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
2025.OCT.24
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

BOREHOLE BH01

PROJECT: Proposed Commercial Development
CLIENT: 1000198532 Ontario Inc.
LOCATION: 2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 221099
DATE OF BORING: 2023-01-30
SHEET: 1 of 1
DATUM: GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)
	Topsoil (FILL)	0.00	▨	116.20
	Yellow brown sand and gravel, some silt (FILL)	0.30	▨	115.90
0.5	TOPSOIL	0.61	░	115.59
1.0	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.81	▧	115.39
1.5	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	1.52	▧	114.68
2.0				
2.5				
	Practical refusal on bedrock or large boulder	2.79		113.41

SAMPLES NUMBER	TYPE	BLOWS/0.3m	UNDIST SHEAR STRENGTH x Cu. kPa x REM SHEAR STRENGTH o Cu. kPa o 0 20 40 60 80 100	DYNAMIC CONE PENETRATION TEST blows/300 mm 0 20 40 60 80 100	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
1	SS	12			13	
2	SS	6			21	
3	SS	5			20	∇
4	SS	14			51	

Groundwater encountered at about 1.8 metres below existing ground surface, January 30, 2023.

DEPTH SCALE: 1 to 25
LOGGED: CI
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
CHECKED: SD

BOREHOLE BH02

PROJECT: Proposed Commercial Development
CLIENT: 1000198532 Ontario Inc.
LOCATION: 2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 221099
DATE OF BORING: 2023-01-30
SHEET: 1 of 1
DATUM: GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x REM SHEAR STRENGTH o Cu. kPa o 0 20 40 60 80 100	DYNAMIC CONE PENETRATION TEST blows/300 mm 0 20 40 60 80 100	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m				
	TOPSOIL	0.00		116.11							
0.5	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.20		115.91	1	SS	12			16	
1.0					2	SS	61			12	
1.5	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	1.22		114.89							
2.0					3	SS	66				∇
					4	SS	100			10	
	Practical refusal on bedrock or large boulder	2.41		113.70							

Groundwater encountered at about 1.8 metres below existing ground surface, January 30, 2023.

DEPTH SCALE: 1 to 25

LOGGED: CI

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem

CHECKED: SD

BOREHOLE BH03

PROJECT: Proposed Commercial Development
CLIENT:1000198532 Ontario Inc.
LOCATION:2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER:63.5 kg, Drop, 0.76 mm

PROJECT NUMBER:221099
DATE OF BORING: 2023-01-30
SHEET:1 of 1
DATUM:GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o 0 20 40 60 80 100	DYNAMIC CONE PENETRATION TEST blows/300 mm 0 20 40 60 80 100	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m				
	Topsoil (FILL)	0.00		116.79							
0.5	Yellow brown silty sand, some gravel (FILL)	0.20		116.59	1	SS	5			20	
	TOPSOIL	0.61		116.18							
1.0	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.81		115.98	2	SS	3			28	
1.5											▽
2.0	Grey brown silty sand, some gravel cobbles, boulders, trace clay (GLACIAL TILL)	1.52		115.27	3	SS	18			13	
2.5					4	SS	65			12	
	Practical refusal on bedrock or large boulder	2.74		114.05							

Groundwater encountered at about 1.5 metres below existing ground surface, January 30, 2023.

DEPTH SCALE: 1 to 25

LOGGED: CI

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem

CHECKED: SD

BOREHOLE BH04

PROJECT: Proposed Commercial Development
CLIENT:1000198532 Ontario Inc.
LOCATION:2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER:63.5 kg, Drop, 0.76 mm

PROJECT NUMBER:221099
DATE OF BORING: 2023-01-30
SHEET:1 of 1
DATUM:GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o 0 20 40 60 80 100	DYNAMIC CONE PENETRATION TEST blows/300 mm 0 20 40 60 80 100	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m				
	TOPSOIL	0.00		117.20	1	SS	7			15	
0.5	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.30		116.90							
1.0					2	SS	44			9	∇
1.5	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	1.52		115.68							
2.0					3	SS	33			13	
2.5					4	SS	59			20	
	Practical refusal on bedrock or large boulder	2.77		114.43							

Groundwater encountered at about 0.9 metres below existing ground surface, January 30, 2023.

DEPTH SCALE: 1 to 25
LOGGED: CI

BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
CHECKED: SD

BOREHOLE BH05

PROJECT: Proposed Commercial Development
CLIENT: 1000198532 Ontario Inc.
LOCATION: 2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 221099
DATE OF BORING: 2023-01-30
SHEET: 1 of 1
DATUM: GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x	DYNAMIC CONE PENETRATION TEST	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m	REM SHEAR STRENGTH o Cu. kPa o 0 20 40 60 80 100	blows/300 mm 0 20 40 60 80 100		
	TOPSOIL	0.00		117.86							
	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.15		117.71	1	SS	13			10	
0.5											
1.0					2	SS	6			23	
1.5					3	SS	11			17	
	End of borehole in GLACIAL TILL	1.83		116.03							

Borehole dry at time of drilling, January 30, 2023.

DEPTH SCALE: 1 to 25

LOGGED: CI

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem

CHECKED: SD

BOREHOLE BH06

PROJECT: Proposed Commercial Development
CLIENT: 1000198532 Ontario Inc.
LOCATION: 2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 221099
DATE OF BORING: 2023-01-30
SHEET: 1 of 1
DATUM: GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o	DYNAMIC CONE PENETRATION TEST	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m	0 20 40 60 80 100	blows/300 mm 0 20 40 60 80 100		
	TOPSOIL	0.00		117.60							
0.5	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.15		117.45	1	SS	4			17	
1.0					2	SS	18			14	
1.5	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	1.22		116.38	3	SS	39			12	∇
	End of borehole in GLACIAL TILL	1.83		115.77							

Groundwater encountered at about 1.5 metres below existing ground surface, January 30, 2023.

DEPTH SCALE: 1 to 25
LOGGED: CI
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
CHECKED: SD

BOREHOLE BH07

PROJECT: Proposed Commercial Development
CLIENT: 1000198532 Ontario Inc.
LOCATION: 2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 221099
DATE OF BORING: 2023-01-30
SHEET: 1 of 1
DATUM: GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o (0 20 40 60 80 100)	DYNAMIC CONE PENETRATION TEST blows/300 mm (0 20 40 60 80 100)	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m				
	TOPSOIL	0.00		116.82							
0.5	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.15		116.67	1	SS	6			14	
1.0					2	SS	6				
1.5	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	1.22		115.60	3	SS	15			12	▽
	End of borehole in GLACIAL TILL	1.83		114.99							

Groundwater encountered at about 1.5 metres below existing ground surface, January 30, 2023.

DEPTH SCALE: 1 to 25
LOGGED: CI
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
CHECKED: SD

BOREHOLE BH08

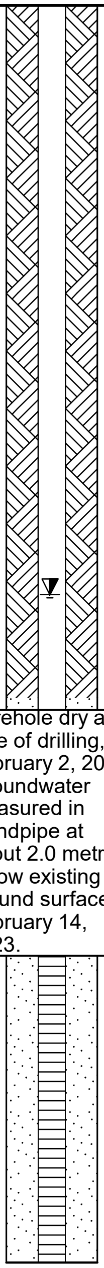
PROJECT: Proposed Commercial Development
CLIENT: 1000198532 Ontario Inc.
LOCATION: 2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 221099
DATE OF BORING: 2023-02-02
SHEET: 1 of 1
DATUM: GEODETIC

SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o (0 20 40 60 80 100)	DYNAMIC CONE PENETRATION TEST blows/300 mm (0 20 40 60 80 100)	MOISTURE CONTENT (%)
DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m			
TOPSOIL	0.00		116.79						
Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.15		116.64	1	SS	9			18
				2	SS	4			13
				3	SS	5			18
Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	2.13		114.66	4	SS	34			10
				5	SS	31			9
				6	SS	88			8
Practical refusal on bedrock or large boulder	4.16		112.63						

DEPTH SCALE (meters): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0

PIEZOMETER OR STANDPIPE INSTALLATION



Borehole dry at time of drilling, February 2, 2023. Groundwater measured in standpipe at about 2.0 metres below existing ground surface, February 14, 2023.

DEPTH SCALE: 1 to 25

LOGGED: CI

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem

CHECKED: SD

BOREHOLE BH09

PROJECT: Proposed Commercial Development
CLIENT:1000198532 Ontario Inc.
LOCATION:2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER:63.5 kg, Drop, 0.76 mm

PROJECT NUMBER:221099
DATE OF BORING: 2023-02-02
SHEET:1 of 1
DATUM:GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x / REM SHEAR STRENGTH o Cu. kPa o (0 20 40 60 80 100)	DYNAMIC CONE PENETRATION TEST blows/300 mm (0 20 40 60 80 100)	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m				
	TOPSOIL	0.00		117.97							
0.5	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.20		117.77	1	SS	7			24	
1.0 1.5					2	SS	6			21	
2.0	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	1.68		116.29	3	SS	21			11	
2.5					4	SS	76			8	
3.0 3.5					5	SS	63			8	
					6	SS	100			10	
	Practical refusal on bedrock or large boulder	3.89		114.08							

Borehole dry at time of drilling, February 2, 2023.

DEPTH SCALE: 1 to 25

LOGGED: CI

BORING METHOD: Power Auger

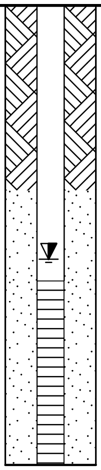
AUGER TYPE: 200 mm Hollow Stem

CHECKED: SD

BOREHOLE BH10

PROJECT: Proposed Commercial Development
CLIENT:1000198532 Ontario Inc.
LOCATION:2726 to 2732 Moodie Drive
PENETRATION TEST HAMMER:63.5 kg, Drop, 0.76 mm

PROJECT NUMBER:221099
DATE OF BORING: 2023-02-02
SHEET:1 of 1
DATUM:GEODETIC

DEPTH SCALE (meters)	SOIL PROFILE				SAMPLES			UNDIST SHEAR STRENGTH x Cu. kPa x REM SHEAR STRENGTH o Cu. kPa o 0 20 40 60 80 100	DYNAMIC CONE PENETRATION TEST blows/300 mm 0 20 40 60 80 100	MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m				
	TOPSOIL	0.00		117.41							
	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	0.20		117.21	1	SS	10			10	
0.5											
1.0					2	SS	32			10	
1.5											
	Practical refusal on bedrock or large boulder	1.52		115.89							

Borehole dry at time of drilling, February 2, 2023. Groundwater measured in standpipe at about 0.8 metres below existing ground surface, February 14, 2023.

DEPTH SCALE: 1 to 25
LOGGED: CI

BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
CHECKED: SD

TABLE I

RECORD OF TEST PITS
LIMITED SUBSURFACE INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
2726 MOODIE DRIVE
CITY OF OTTAWA, ONTARIO

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP1 (Elev. 115.05 m)	0.00 – 0.70	TOPSOIL
	0.70 – 1.30	Yellow brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.30 – 1.80	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.80 – 4.00	Grey silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	4.00	Practical refusal on BEDROCK

Some groundwater intrusion at about 1.3 metres below the existing ground surface, April 14, 2023.

The glacial till was observed to be compact to dense based on difficulty of excavation and tactile examination.

TABLE I (Continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP2 (Elev. 115.20 m)	0.00 – 0.60	TOPSOIL
	0.60 – 1.20	Yellow brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.20 – 1.80	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.80 – 3.40	Grey silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	3.40	Practical refusal on BEDROCK

Some groundwater intrusion at about 1.2 metres below the existing ground surface, April 14, 2023.

The glacial till was observed to be compact to dense based on difficulty of excavation and tactile examination.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP3 (Elev. 117.55 m)	0.00 – 0.50	TOPSOIL
	0.50 – 1.50	Red brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.50 – 3.40	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	3.40	Practical refusal on BEDROCK

Some groundwater intrusion at about 1.5 metres below the existing ground surface, April 14, 2023.

The glacial till was observed to be loose to compact to about 1.5 metres, then compact to dense based on difficulty of excavation and tactile examination.

TABLE I (Continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP4 (Elev. 117.54 m)	0.00 – 0.50	Grey silty clay, trace to some organics (FILL)
	0.50 – 0.70	TOPSOIL
	0.70 – 1.30	Yellow brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.30 – 3.20	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	3.20	Practical refusal on BEDROCK

Some groundwater intrusion at about 1.3 metres below the existing ground surface, April 14, 2023.

The glacial till was observed to be compact to dense based on difficulty of excavation and tactile examination.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP5 (Elev. 116.87 m)	0.00 – 0.60	TOPSOIL
	0.60 – 1.00	Yellow brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.00 – 1.80	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.80 – 3.00	Grey silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	3.00	Practical refusal on BEDROCK

Some groundwater intrusion at about 1.0 metres below the existing ground surface, April 14, 2023.

The glacial till was observed to be compact to dense based on difficulty of excavation and tactile examination.

TABLE I (Continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP6 (Elev. 116.33 m)	0.00 – 0.60	Grey brown silty clay, trace to some organics (FILL)
	0.60 – 0.80	TOPSOIL
	0.80 – 1.20	Yellow brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.20 – 1.50	Grey brown silty sand, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)
	1.50	Practical refusal on BEDROCK

Some groundwater intrusion at about 1.2 metres below the existing ground surface, April 14, 2023.

The glacial till was observed to be compact to dense based on difficulty of excavation and tactile examination.

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

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

PENETRATION RESISTANCE

Standard Penetration Resistance, N

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

Dynamic Penetration Resistance

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

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drill rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	sieve and hydrometer analysis
U	unconfined compression test
Q	undrained triaxial test
V	field vane, undisturbed and remolded shear strength

SOIL DESCRIPTIONS

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

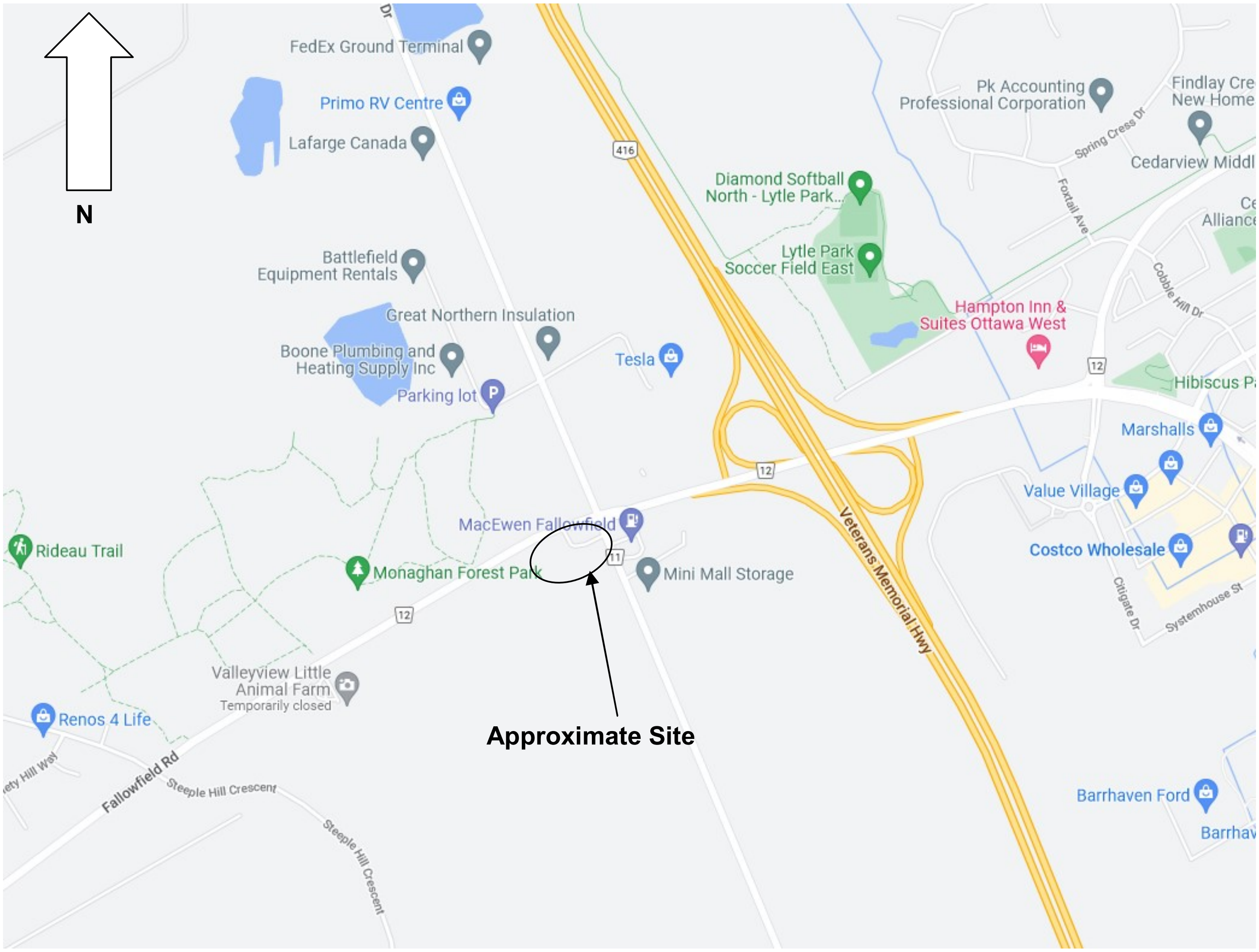
Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to100
Very Stiff	over100

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

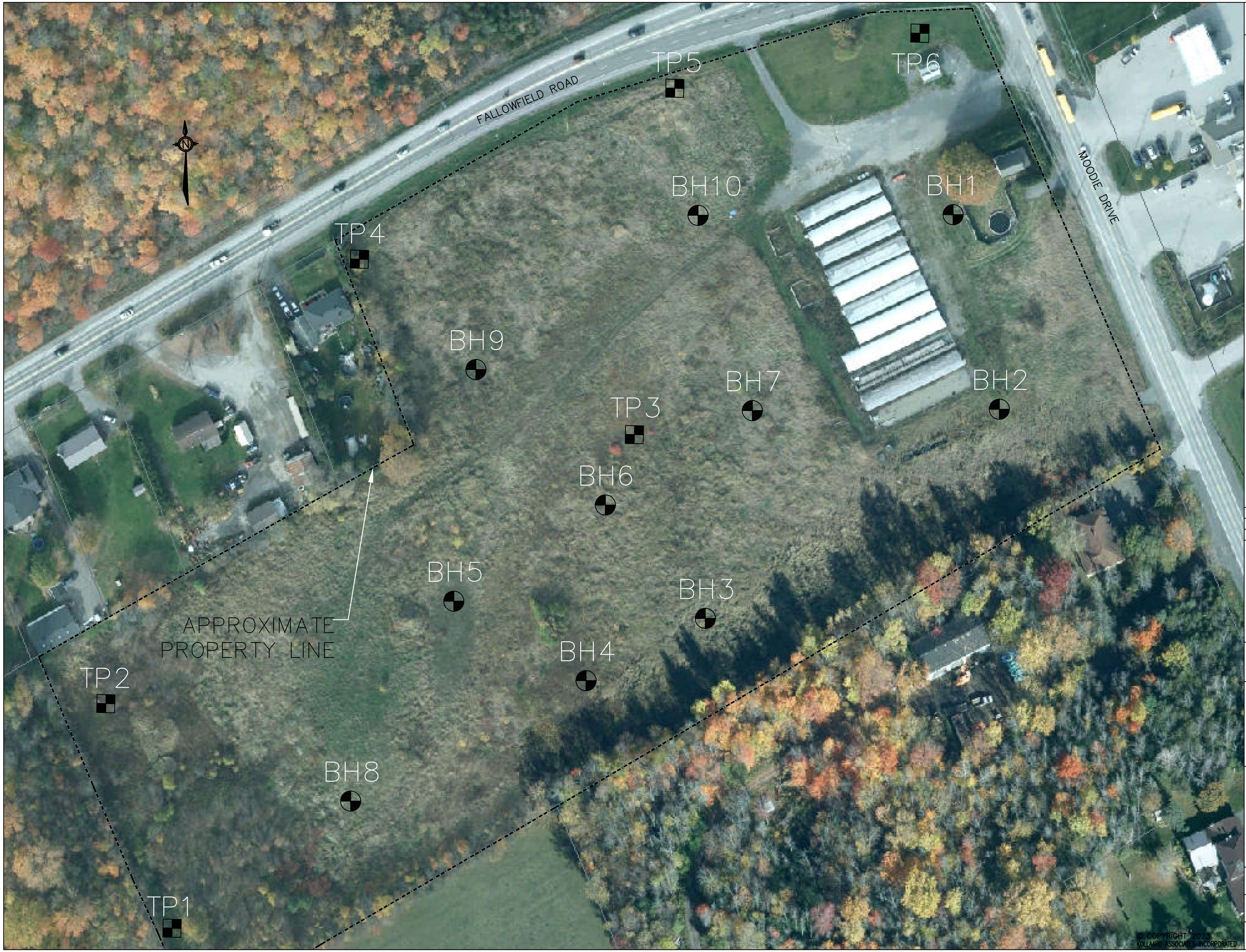


NOT TO SCALE

Kollaard Associates
Engineers

Project No. 221099

Date September 2023



DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

● BH1	APPROXIMATE BOREHOLE LOCATION
▨ TP1	APPROXIMATE TEST PIT LOCATION

REFERENCE: PLAN SUPPLIED BY
COSINE MAPPING

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
KOG 1J0 FAX (613) 258-0475
http://www.kollaard.ca

(613) 860-0923
info@kollaard.ca

CLIENT:
1000198532 ONTARIO INC.

PROJECT:
GEOTECHNICAL INVESTIGATION FOR
PROPOSED COMMERCIAL DEVELOPMENT

LOCATION:
2726 MOODIE DRIVE
OTTAWA, ONTARIO

DESIGNED BY: --
DATE: SEP 12, 2023
DRAWN BY: DT
SCALE: N.T.S
KOLLAARD FILE NUMBER: 221099

ATTACHMENT A

MASW Survey Results

TECHNICAL MEMORANDUM

DATE January 17, 2023 **Project No.** 22578721

TO Dean Tataryn
Kollaard Associates

CC

FROM Alex Bilson Darko, Christopher Phillips **EMAIL alex.bilson.darko@wsp.com; christopher.phillips@wsp.com**

MASW SURVEY RESULTS – 2732 MOODIE DRIVE, NEPEAN, ONTARIO

This technical memorandum presents the processing and results of the Multichannel Analysis of Surface-Waves (MASW) test performed for the purpose of Seismic Site Classification for a site on 2732 Moodie Drive, located in Nepean, Ontario. The geophysical testing was performed by WSP personnel on December 21st, 2022, along the survey line shown in Figure 1, below.

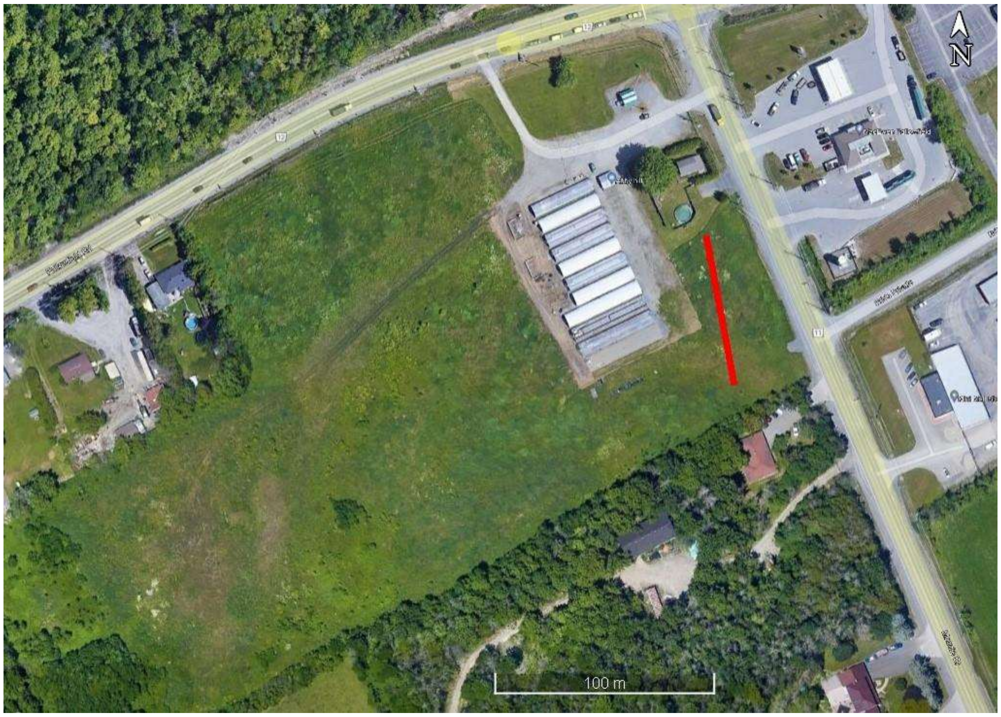


Figure 1: MASW Survey Line Location in red.

Methodology

The MASW method measures variations in surface-wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

A typical MASW survey requires a seismic source, to generate surface-waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface-waves are a special type of seismic wave whose propagation is confined to the near surface medium.

The depth of penetration of a surface-wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium surface-waves are dispersive, i.e., each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that wavelength of surface-wave propagates through. The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, sledgehammer and vibrating pads. Examples of passive sources are road traffic, micro-tremors and water-wave action (in near-shore environments).

The geophone receivers measure the wave-train associated with the surface-wave travelling from a seismic source at different distances from the source.

The participation of surface-waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to wavelength (called the ‘dispersion curve’) is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear modulus of the medium as a function of depth.

Field Work

The MASW field work was conducted on December 21st, 2022, by personnel from the WSP Mississauga office. For the MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 3 metre intervals. An 8-kilogram (kg) sledgehammer and 40 kg seismic weight drop were used as seismic sources for this investigation. Seismic records were collected with seismic sources located 5 and 10 metres from and collinear to the geophone array. An example of an active seismic record collected at the site is shown in Figure 2.

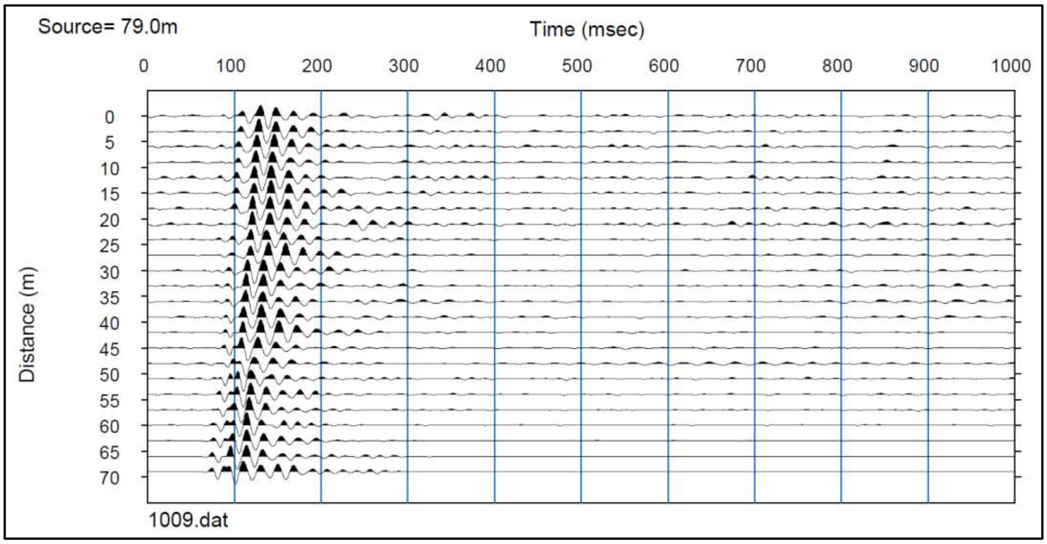


Figure 2: Typical seismic record collected for the MASW Line.

Data Processing

Processing of the MASW test results consisted of the following main steps:

- Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;
- Calculation of the phase for each frequency component;
- Linear regression to calculate phase velocity for each frequency component;
- Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r^2) between the data and the linear regression best fit line used to calculate phase velocity;
- Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and
- Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown in Figure 3. Shear-wave velocity (V_s) profiles were generated through inverse modelling to best fit the calculated fundamental mode dispersion curves.

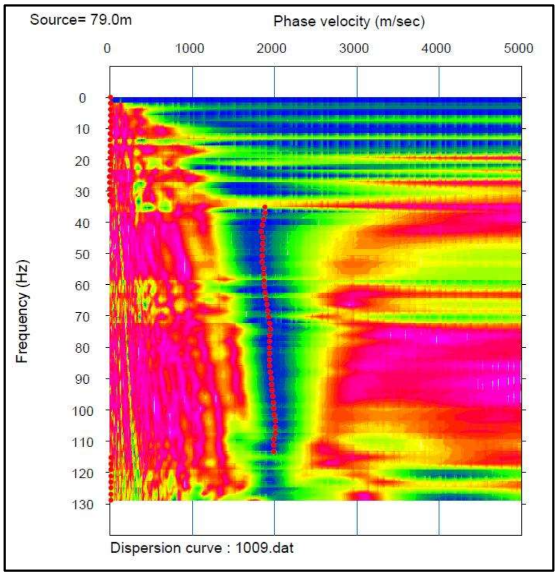


Figure 3: MASW Dispersion Curve Picks (red dots) for the MASW Line.

The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 35 Hz for the MASW Line.

Results

The MASW test result is presented in Figure 4 as the calculated shear-wave velocity profile measured from the MASW Line. There is good correlation between the field collected and model calculated dispersion curves, with a root mean squared error of less than 5%.

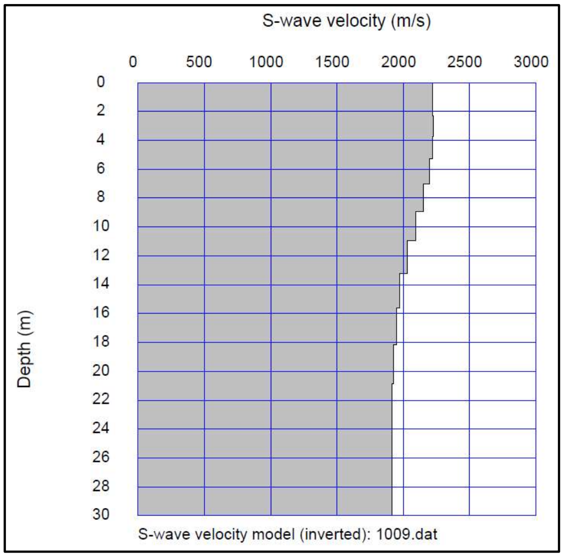


Figure 4: MASW Modelled Shear-Wave Velocity Depth profile for the MASW Line.

Table 1: Shear-Wave Velocity Profile for the MASW Line

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0	1.1	1.1	2218	0.000496
1.1	2.3	1.2	2221	0.000540
2.3	3.7	1.4	2226	0.000629
3.7	5.3	1.6	2221	0.000720
5.3	7.0	1.7	2196	0.000774
7.0	8.9	1.9	2152	0.000883
8.9	11.0	2.1	2096	0.001002
11.0	13.2	2.2	2032	0.001083
13.2	15.6	2.4	1974	0.001216
15.6	18.1	2.5	1950	0.001282
18.1	20.9	2.8	1927	0.001453
20.9	23.7	2.8	1916	0.001461
23.7	26.8	3.1	1914	0.001620
26.8	30.0	3.2	1917	0.001670
Vs Average to 30 mbgs (m/s)				2023

To calculate the average shear-wave velocity as required by Seismic Site Classification, the results were modelled to 30 metres below ground surface (mbgs).

The time-averaged shear-wave velocity (Vs30) for the MASW Line was found to be 2023 m/s (Table 1).

Closure

We trust that this technical memorandum meets your needs at the present time. If you have any questions or require clarification, please contact the undersigned at your convenience.

WSP Canada Inc.

Alex Bilson Darko, MSc
Geophysicist

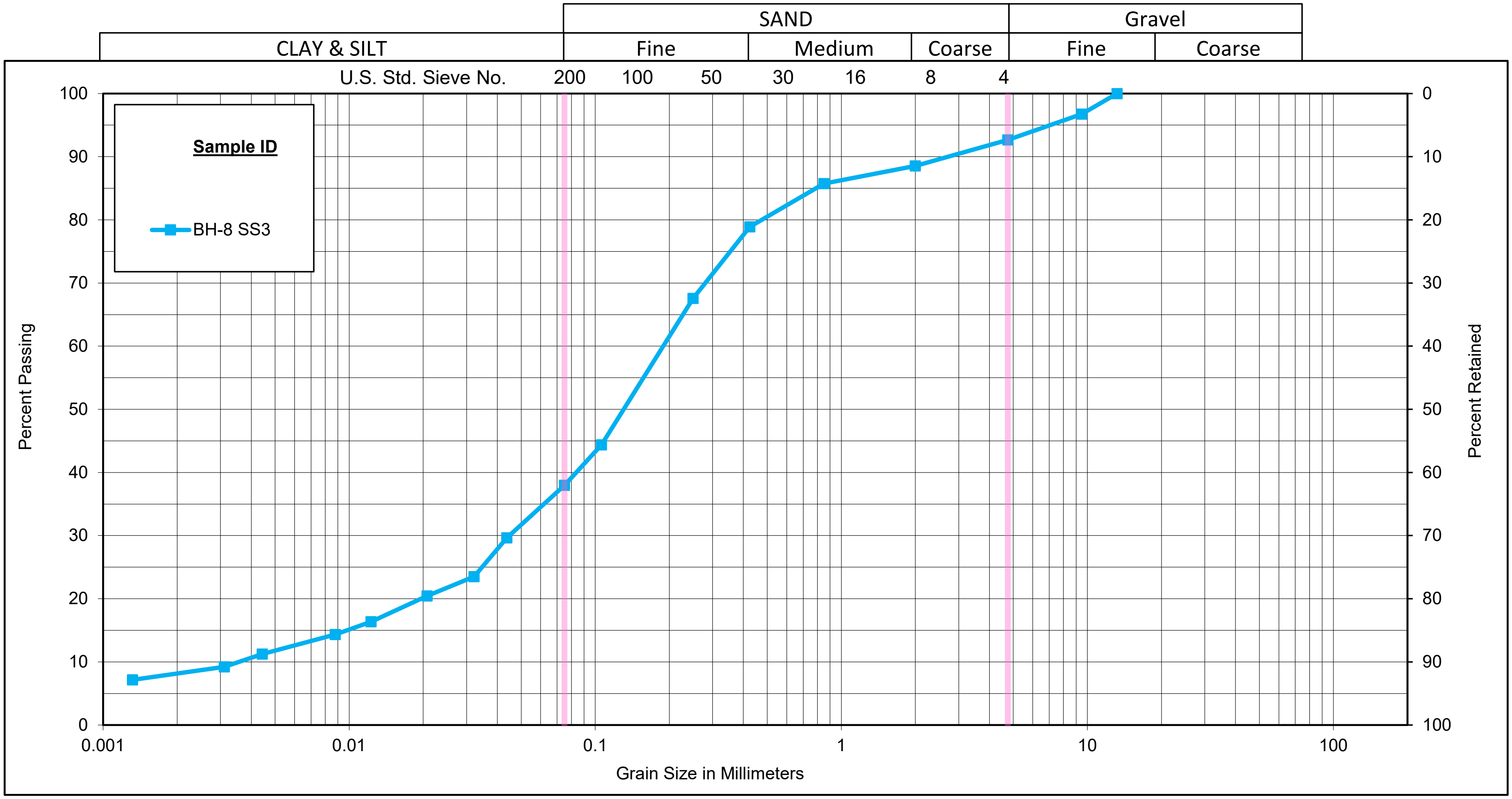
Christopher Phillips, MSc, PGeo
Geophysicist VII, Senior Principal

ABD/CRP/jl

ATTACHMENT B

Laboratory Test Results for Physical Properties

Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH-8 SS3	5'-7'	7.3	54.7	30.0	8.0

Stantec

GRAIN SIZE DISTRIBUTION

Kollaard Associates Engineers, File #221099

2726 to 2732 Moodie Drive

Figure No.

Project No. 122410003

Stantec

Particle-Size Analysis of Soils
LS702
AASHTO T88

PROJECT DETAILS

Client:	**Kollaard Associates Engineers, File #221099**	Project No.:	**122410003**
Project:	**2726 to 2732 Moodie Drive**	Test Method:	**LS702**
Material Type:	**Soil**	Sampled By:	**Kollaard Associates Engineers**
Source:	**BH-8**	Date Sampled:	**February 2, 2023**
Sample No.:	**SS3**	Tested By:	**Brian Prevost**
Sample Depth	**5'-7'**	Date Tested:	**March 20, 2023**

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	84.74
Sample Weight after Hydrometer and Wash (g)	49.10
Percent Passing No. 200 Sieve (%)	**42.1**
Percent Passing Corrected (%)	**37.24**

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	**438.50**
Sample Weight After Sieve (g)	**436.70**
Percent Loss in Sieve (%)	**0.41**

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	24	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	170.12
Air Dried Mass (W_a), (g)	170.75
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9963
Air Dried Mass in Analysis (M_a), (g)	85.05
Oven Dried Mass in Analysis (M_o), (g)	84.74
Percent Passing 2.0 mm Sieve (P_{10}), (%)	88.55
Sample Represented (W), (g)	95.69

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	0.0	100.0
9.5	14.2	96.8
4.75	32.2	92.7
2.00	50.2	88.6
Total (C + F)^1	436.70	
0.850	2.70	85.73
0.425	9.24	78.90
0.250	20.10	67.55
0.106	42.28	44.37
0.075	48.42	37.95
PAN	48.68	

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 9:05 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
20-Mar-23	9:06 AM	1	33.0	4.0	21.0	29.0	29.65	11.09904	9.84835	0.013126	0.04373
20-Mar-23	9:07 AM	2	27.0	4.0	21.0	23.0	23.52	12.02904	9.84835	0.013126	0.03219
20-Mar-23	9:10 AM	5	24.0	4.0	21.0	20.0	20.45	12.49404	9.84835	0.013126	0.02075
20-Mar-23	9:20 AM	15	20.0	4.0	21.0	16.0	16.36	13.11404	9.84835	0.013126	0.01227
20-Mar-23	9:35 AM	30	18.0	4.0	21.0	14.0	14.31	13.42404	9.84835	0.013126	0.00878
20-Mar-23	11:05 AM	120	15.0	4.0	21.5	11.0	11.25	13.88904	9.73081	0.013047	0.00444
20-Mar-23	1:15 PM	250	13.0	4.0	21.5	9.0	9.2018	14.19904	9.73081	0.013047	0.00311
21-Mar-23	9:05 AM	1440	11.0	4.0	21.0	7.0	7.1570	14.50904	9.84835	0.013126	0.00132

Remarks:

Reviewed By: Brian Prevost
Date: March 22, 2023

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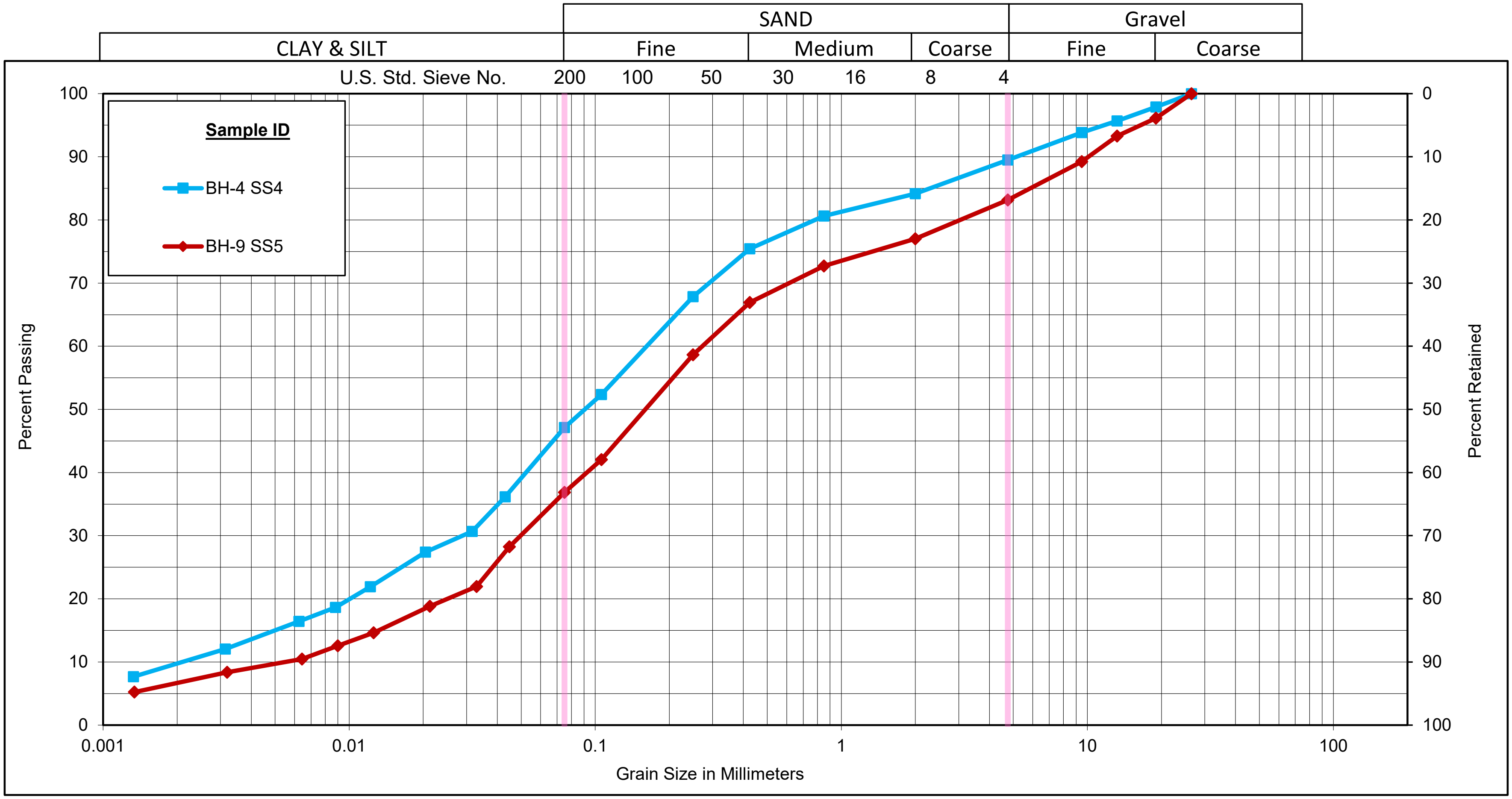
Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH-4 SS4	7'6"-9'6"	10.5	42.4	38.1	9.0
BH-9 SS5	10'-12'	16.8	46.3	30.9	6.0

Stantec

GRAIN SIZE DISTRIBUTION

Kollaard Associates, File #221099

2732 Moodie Drive, Ottawa

Figure No.

Project No. 122410003

Stantec

Particle-Size Analysis of Soils

LS702

AASHTO T88

PROJECT DETAILS

Client:	**Kollaard Associates, File #221099**	Project No.:	**122410003**
Project:	**2732 Moodie Drive, Ottawa**	Test Method:	**LS702**
Material Type:	**Soil**	Sampled By:	**Kollaard Associates**
Source:	**BH-4**	Date Sampled:	**February 6, 2023**
Sample No.:	**SS4**	Tested By:	**Brian Prevost**
Sample Depth	**7'6"-9'6"**	Date Tested:	**February 9, 2023**

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	75.13
Sample Weight after Hydrometer and Wash (g)	33.78
Percent Passing No. 200 Sieve (%)	**55.0**
Percent Passing Corrected (%)	**46.31**

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	**996.50**
Sample Weight After Sieve (g)	**994.40**
Percent Loss in Sieve (%)	**0.21**

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)	70.38
Air Dried Mass (W_a), (g)	70.52
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9980
Air Dried Mass in Analysis (M_a), (g)	75.28
Oven Dried Mass in Analysis (M_o), (g)	75.13
Percent Passing 2.0 mm Sieve (P_{10}), (%)	84.14
Sample Represented (W), (g)	89.29

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	21.1	97.9
13.2	42.9	95.7
9.5	61.4	93.8
4.75	104.5	89.5
2.00	158.0	84.1
Total (C + F)[1]	994.40	
0.850	3.13	80.64
0.425	7.79	75.42
0.250	14.55	67.85
0.106	28.39	52.35
0.075	33.06	47.12
PAN	33.62	

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 9:22 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
09-Feb-23	9:23 AM	1	40.0	7.0	18.0	33.0	36.16	10.01404	10.60820	0.013623	0.04311
09-Feb-23	9:24 AM	2	35.0	7.0	18.0	28.0	30.68	10.78904	10.60820	0.013623	0.03164
09-Feb-23	9:27 AM	5	32.0	7.0	18.0	25.0	27.39	11.25404	10.60820	0.013623	0.02044
09-Feb-23	9:37 AM	15	27.0	7.0	18.0	20.0	21.92	12.02904	10.60820	0.013623	0.01220
09-Feb-23	9:52 AM	30	24.0	7.0	18.0	17.0	18.63	12.49404	10.60820	0.013623	0.00879
09-Feb-23	10:22 AM	60	22.0	7.0	18.5	15.0	16.44	12.80404	10.47474	0.013537	0.00625
09-Feb-23	1:32 PM	250	18.0	7.0	18.5	11.0	12.0533	13.42404	10.47474	0.013537	0.00314
10-Feb-23	9:22 AM	1440	14.0	7.0	19.0	7.0	7.6703	14.04404	10.34409	0.013452	0.00133

Remarks:

Reviewed By: Brian Prevost

Date: February 13, 2023

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Stantec

Particle-Size Analysis of Soils

LS702

AASHTO T88

PROJECT DETAILS

Client:	**Kollaard Associates, File #221099**	Project No.:	**122410003**
Project:	**2732 Moodie Drive, Ottawa**	Test Method:	**LS702**
Material Type:	**Soil**	Sampled By:	**Kollaard Associates**
Source:	**BH-9**	Date Sampled:	**February 6, 2023**
Sample No.:	**SS5**	Tested By:	**Brian Prevost**
Sample Depth	**10'-12'**	Date Tested:	**February 9, 2023**

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	72.04
Sample Weight after Hydrometer and Wash (g)	38.22
Percent Passing No. 200 Sieve (%)	**46.9**
Percent Passing Corrected (%)	**36.16**

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	**1260.80**
Sample Weight After Sieve (g)	**1257.40**
Percent Loss in Sieve (%)	**0.27**

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)	77.89
Air Dried Mass (W_a), (g)	78.05
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9980
Air Dried Mass in Analysis (M_a), (g)	72.19
Oven Dried Mass in Analysis (M_o), (g)	72.04
Percent Passing 2.0 mm Sieve (P_{10}), (%)	77.02
Sample Represented (W), (g)	93.53

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	48.8	96.1
13.2	84.7	93.3
9.5	135.9	89.2
4.75	212.4	83.2
2.00	289.7	77.0
Total (C + F)[1]	1257.40	
0.850	4.04	72.70
0.425	9.43	66.94
0.250	17.17	58.67
0.106	32.71	42.05
0.075	37.56	36.87
PAN	37.57	

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 9:51 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
9-Feb-23	9:52 AM	1	34.0	7.0	18.5	27.0	28.24	10.94404	10.47474	0.013537	0.04478
9-Feb-23	9:53 AM	2	28.0	7.0	18.5	21.0	21.97	11.87404	10.47474	0.013537	0.03298
9-Feb-23	9:56 AM	5	25.0	7.0	18.5	18.0	18.83	12.33904	10.47474	0.013537	0.02127
9-Feb-23	10:06 AM	15	21.0	7.0	18.5	14.0	14.64	12.95904	10.47474	0.013537	0.01258
9-Feb-23	10:21 AM	30	19.0	7.0	18.5	12.0	12.55	13.26904	10.47474	0.013537	0.00900
9-Feb-23	10:51 AM	60	17.0	7.0	18.5	10.0	10.46	13.57904	10.47474	0.013537	0.00644
9-Feb-23	2:01 PM	250	15.0	7.0	18.5	8.0	8.37	13.88904	10.47474	0.013537	0.00319
10-Feb-23	9:51 AM	1440	12.0	7.0	19.1	5.0	5.23	14.35404	10.31830	0.013435	0.00134

Remarks:

Reviewed By: Brian Prevost

Date: February 13, 2023

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Stantec
2781 Lancaster Rd. Suite 100 A&B
Ottawa ON, K1B 1A7

Determination of Moisture Content of Soil

ASTM D2216

Project: Kollaard File #221099
Project No.: 122410003
Date Tested: February 7, 2023
Tested By: Brian Prevost

Moisture Content Test Results

Borehole / Test Pit No.		BH-4		BH-9			
Sample		SS4		SS5			
Moisture Content (%)		**14.5**		**7.7**			
Borehole / Test Pit No.							
Sample							
Sample Depth (ft)							
Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Moisture Content (%)							

Borehole / Test Pit No.							
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Borehole / Test Pit No.							
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Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Sample Depth (ft)							
Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Sample Depth (ft)							
Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Moisture Content (%)							

Reviewed By: Brian Prevost

Date: February 13, 2023

Stantec

2781 Lancaster Rd.
Ottawa ON, K1B 1A7

Determination of Moisture Content of Soil

LS 701
ASTM D2216

Project: Kollaard, File #221099
Project No.: 122410003

Date Tested: March 17, 2023
Tested By: Brian Prevost

Moisture Content Test Results

Borehole / Test Pit No.		BH-8					
Sample		SS3					
Moisture Content (%)		**17.0**					
Borehole / Test Pit No.							
Sample							
Sample Depth (ft)							
Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Moisture Content (%)							
Borehole / Test Pit No.							
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Borehole / Test Pit No.							
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Moisture Content (%)							

Borehole / Test Pit No.							
Sample							
Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Sample Depth (ft)							
Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Sample Depth (ft)							
Moisture Content (%)							
Borehole / Test Pit No.							
Sample							
Moisture Content (%)							

Reviewed By: Brian Prevost

Date: March 22, 2023

Moisture Content

LS - 701 / ASTM D 2216

PROJECT NO.: 221099	DATE SAMPLED: Jan 30 & Feb 2/23	DATE TESTED: February 6, 2023
CLIENT: 1000198532 Ontario Inc	DATE RECEIVED:	TESTED BY: CI
LOCATION: 2726 to 2732 Moodie Drive	DATE REQUESTED:	FILE NO.:

METHOD A ■ METHOD B ☐

Water Content Recorded to +/- 1%			Water Content Recorded to +/- 0.1%		
Sieve Size, mm	Specimen Mass	Balance Readability, g	Sieve Size, mm	Specimen Mass	Balance Readability, g
75.0	5 kg	10	75.0	5 kg	10
37.5	1 kg	10	37.5	1 kg	10
19	250 g	0.1	19	250 g	0.1
9.5	50 g	0.1	9.5	50 g	0.1
4.75	20 g	0.1	4.75	20 g	0.1
2.00	20 g	0.1	2.00	20 g	0.1

ASTM D 2216 TABLE 1

Bore Hole:	BH1	BH1	BH1	BH1	BH2	BH2	BH2	BH3	BH3
Sample No.:	SS1	SS2	SS3	SS4	SS1	SS2	SS4	SS1	SS2
Depth:	0-0.6	0.8-1.4	1.5-2.1	2.3-2.9	0-0.6	0.8-1.4	2.1-2.7	0-0.6	0.8-1.4
Tare No.:	1	2	3	4	5	6	7	8	9
Tare +Wet Soil (gms)	100.74	72.18	96.46	85.67	84.72	104.04	90.08	72.23	77.60
Tare + Dry Soil (gms)	91.27	63.18	84.26	63.92	75.82	95.29	83.70	63.64	65.32
Mass of Water (gms)	9.47	9.00	12.20	21.75	8.90	8.75	6.38	8.59	12.28
Mass of Tare (gms)	20.95	21.10	21.71	21.17	21.35	21.23	21.50	21.42	21.45
Mass of Solids (gms)	70.32	42.08	62.55	42.75	54.47	74.06	62.20	42.22	43.87
WATER CONTENT (%)	13	21	20	51	16	12	10	20	28
Drying Tempterature (ºC), if other than 110 ±5ºC									

Bore Hole:	BH3	BH3	BH4	BH4	BH4	BH4	BH5	BH5	BH5
Sample No.:	SS3	SS4	SS1	SS2	SS3	SS4	SS1	SS2	SS3
Depth:	1.5-2.1	2.3-2.9	0-0.6	0.8-1.4	1.5-2.1	2.3-2.9	0-0.6	0.6-1.2	1.2-1.8
Tare No.:	10	11	12	13	14	15	16	17	18
Tare +Wet Soil (gms)	117.74	88.76	76.82	87.20	100.38	95.15	77.31	78.34	85.19
Tare + Dry Soil (gms)	106.90	81.60	69.72	81.55	91.55	82.62	72.22	67.50	76.05
Mass of Water (gms)	10.84	7.16	7.10	5.65	8.83	12.53	5.09	10.84	9.14
Mass of Tare (gms)	21.23	20.97	21.11	21.08	20.98	21.36	21.20	20.96	21.05
Mass of Solids (gms)	85.67	60.63	48.61	60.47	70.57	61.26	51.02	46.54	55.00
WATER CONTENT (%)	13	12	15	9	13	20	10	23	17
Drying Tempterature (ºC), if other than 110 ±5ºC									

Bore Hole:	BH6	BH6	BH6	BH7	BH7	BH8	BH8	BH8	BH8
Sample No.:	SS1	SS2	SS3	SS1	SS3	SS1	SS2	SS3	SS4
Depth:	0-0.6	0.6-1.2	1.2-1.8	0-0.6	1.2-1.8	0-0.6	0.8-1.4	1.5-2.1	2.3-2.9
Tare No.:	19	20	21	22	23	24	25	26	27
Tare +Wet Soil (gms)	79.87	114.19	101.93	95.59	98.47	96.36	98.62	79.16	102.70
Tare + Dry Soil (gms)	71.30	102.46	93.17	86.32	90.13	85.00	89.88	70.35	95.27
Mass of Water (gms)	8.57	11.73	8.76	9.27	8.34	11.36	8.74	8.81	7.43
Mass of Tare (gms)	21.23	21.37	21.38	21.58	21.21	21.00	21.35	21.49	21.09
Mass of Solids (gms)	50.07	81.09	71.79	64.74	68.92	64.00	68.53	48.86	74.18
WATER CONTENT (%)	17	14	12	14	12	18	13	18	10
Drying Tempterature (ºC), if other than 110 ±5ºC									

Bore Hole:	BH8	BH8	BH9	BH9	BH9	BH9	BH9	BH9	BH10
Sample No.:	SS5	SS6	SS1	SS2	SS3	SS4	SS5	SS6	SS1
Depth:	3.0-3.6	3.8-4.4	0-0.6	0.8-1.4	1.5-2.1	2.3-2.9	3.0-3.6	3.8-4.4	0-0.6
Tare No.:	28	29	30	31	32	33	34	35	36
Tare +Wet Soil (gms)	93.30	114.34	83.15	81.31	84.23	88.08	87.64	93.59	77.64
Tare + Dry Soil (gms)	87.25	107.32	71.30	70.83	78.20	83.00	82.49	87.17	72.58
Mass of Water (gms)	6.05	7.02	11.85	10.48	6.03	5.08	5.15	6.42	5.06
Mass of Tare (gms)	21.03	21.09	21.05	21.37	21.12	21.18	21.25	20.97	20.99
Mass of Solids (gms)	66.22	86.23	50.25	49.46	57.08	61.82	61.24	66.20	51.59
WATER CONTENT (%)	9	8	24	21	11	8	8	10	10
Drying Tempterature (ºC), if other than 110 ±5ºC									

Bore Hole:	BH10								
Sample No.:	SS2								
Depth:	0.8-1.4								
Tare No.:	37								
Tare +Wet Soil (gms)	84.89								
Tare + Dry Soil (gms)	78.86								
Mass of Water (gms)	6.03								
Mass of Tare (gms)	21.41								
Mass of Solids (gms)	57.45								
WATER CONTENT (%)	10								
Drying Tempterature (ºC), if other than 110 ±5ºC									

ATTACHMENT C

Laboratory Test Results for Chemical Properties

CERTIFICATE OF ANALYSIS

Work Order	: **WT2302742**	Page	: 1 of 3
Client	: **Kollaard Associates Inc.**	Laboratory	: Waterloo - Environmental
Contact	: Dean Tataryn	Account Manager	: Costas Farassoglou
Address	: 210 Prescott Street Unit 1 Kemptville ON Canada K0G1J0	Address	: 60 Northland Road, Unit 1 Waterloo ON Canada N2V 2B8
Telephone	: 613 860 0923	Telephone	: 613 225 8279
Project	: 221099	Date Samples Received	: 03-Feb-2023 13:01
PO	: ----	Date Analysis Commenced	: 06-Feb-2023
C-O-C number	: ----	Issue Date	: 08-Feb-2023 16:41
Sampler	: ----		
Site	: ----		
Quote number	: SOA 2022		
No. of samples received	: 1		
No. of samples analysed	: 1		

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted. This document shall not be reproduced, except in full.

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

Additional information pertinent to this report will be found in the following separate attachments: Quality Control Report, QC Interpretive report to assist with Quality Review and Sample Receipt Notification (SRN).

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is conducted in accordance with US FDA 21 CFR Part 11.

Signatories	*Position*	*Laboratory Department*
Greg Pokocky	Supervisor - Inorganic	Inorganics, Waterloo, Ontario

General Comments

The analytical methods used by ALS are developed using internationally recognized reference methods (where available), such as those published by US EPA, APHA Standard Methods, ASTM, ISO, Environment Canada, BC MOE, and Ontario MOE. Refer to the ALS Quality Control Interpretive report (QCI) for applicable references and methodology summaries. Reference methods may incorporate modifications to improve performance.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.

Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

Please refer to Quality Control Interpretive report (QCI) for information regarding Holding Time compliance.

Key : CAS Number: Chemical Abstracts Services number is a unique identifier assigned to discrete substances

LOR: Limit of Reporting (detection limit).

Unit	*Description*
µS/cm	microsiemens per centimetre
mg/kg	milligrams per kilogram
ohm cm	ohm centimetres (resistivity)
pH units	pH units

<: less than.

>: greater than.

Surrogate: An analyte that is similar in behavior to target analyte(s), but that does not occur naturally in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery.

Test results reported relate only to the samples as received by the laboratory.

UNLESS OTHERWISE STATED on SRN or QCI Report, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.

Analytical Results

Sub-Matrix: **Soil/Solid**

(Matrix: **Soil/Solid**)

				Client sample ID	**BH2-SS3**	----	----	----	----
				Client sampling date / time	30-Jan-2023 11:00	----	----	----	----
Analyte	*CAS Number*	*Method*	*LOR*	*Unit*	**WT2302742-001**	--------	--------	--------	--------
					Result	----	----	----	----
Physical Tests									
Conductivity (1:2 leachate)	----	E100-L	5.00	µS/cm	88.0	----	----	----	----
pH (1:2 soil:CaCl2-aq)	----	E108A	0.10	pH units	7.87	----	----	----	----
Resistivity	----	EC100R	100	ohm cm	11400	----	----	----	----
Leachable Anions & Nutrients									
Chloride, soluble ion content	16887-00-6	E236.Cl	5.0	mg/kg	5.4	----	----	----	----
Sulfate, soluble ion content	14808-79-8	E236.SO4	20	mg/kg	<20	----	----	----	----

Please refer to the General Comments section for an explanation of any qualifiers detected.

ALS Environmental
www.alsglobal.com

Chain of Custody (COC) / Analytical Request Form

Canada Toll Free: 1 800 668 9878

COC Numb

Affix ALS barcode label here
(lab use only)

Environmental Division
Waterloo
Work Order Reference
WT2302742

Telephone : +1 519 886 6910

Report To	Contact and company name below will appear on the final report
Company:	Kollaard Associates (27196)
Contact:	Dean Tataryn
Phone:	613.860.0923, ext.225
	Company address below will appear on the final report
Street:	210 Prescott Street, Unit 1 P.O. Box 189
City/Province:	Kemptville, Ontario
Postal Code:	K0G 1J0
Invoice To	Same as Report To ☐ YES ☐ NO
	Copy of Invoice with Report ☐ YES ☐ NO
Company:	
Contact:	

Report Format / Distribution

Select Report Format: ☑ PDF ☑ EXCEL ☐ EDD (DIGITAL)
Quality Control (QC) Report with Report ☐ YES ☑ NO
☐ Compare Results to Criteria on Report - provide details below if box checked
Select Distribution: ☐ EMAIL ☐ MAIL ☐ FAX
Email 1 or Fax dean@kollaard.ca
Email 2
Email 3

Invoice Distribution

Select Invoice Distribution: ☐ EMAIL ☐ MAIL ☐ FAX
Email 1 or Fax mary@kollaard.ca
Email 2

Select Service Level Below - Contact your

Regular [R] ☐ Standard TAT if received

PRIORITY (Business Days)		EMERGENCY	
4 day [P4-20%]	☐	1 Bus	
3 day [P3-25%]	☐	Same	
2 day [P2-50%]	☐	(Labo	

Date and Time Required for all E&P TATs:

For tests that can not be performed according to the service l

Project Information

ALS Account # / Quote #: Q71021
Job #: 221099
PO / AFE:
LSD:

Oil and Gas Required Fields (client use)

AFE/Cost Center:	PO#
Major/Minor Code:	Routing Code:
Requisitioner:	
Location:	

ALS Lab Work Order # (lab use only): WT2302742 ~~FA~~ JG

ALS Contact: Melanie M.

Sampler:

Anal

Indicate Filtered (F), Preserved (P) or Filtered and Preserved (F/P) below

ALS Sample # (lab use only)	Sample Identification and/or Coordinates (This description will appear on the report)	Date (dd-mmm-yy)	Time (hh:mm)	Sample Type	Corrosivity (KOLLAARD-CORR-WT)	VOC, F1-F4 (VOC-RSII-F1-F4-WT)	Metals & Inorganics (RSII-Inorganics)	PAH (PAH-511-WT)	BTEX / F1-F4	SAMPLES ON HOLD	Sample is hazardous (please provide further detail
	BH12-SS3	Jan 30, 2023	11:00	Soil	X						

Drinking Water (DW) Samples[1] (client use)

Are samples taken from a Regulated DW System?
☐ YES ☐ NO

Are samples for human consumption/ use?
☐ YES ☐ NO

Special Instructions / Specify Criteria to add on report by clicking on the drop-down list below (electronic COC only)

SAMPLE CONDITION AS RECEIVED (lab use only)

Frozen ☐ SIF Observations Yes ☐ No
Ice Packs ☐ Ice Cubes ☐ Custody seal intact Yes ☐ No
Cooling Initiated ☐

INITIAL COOLER TEMPERATURES °C				FINAL COOLER TEMPERATURES °C		
15.4				-0.4	0.2	

SHIPMENT RELEASE (client use)			INITIAL SHIPMENT RECEPTION (lab use only)			FINAL SHIPMENT RECEPTION (lab use only)		
Released by:	Date: Feb 1, 2023	Time: 1:00	Received by: COSTA F.	Date: 2/3/23	Time: 1:10PM	Received by: FH	Date: 2023-02-04	Time: 11:15A

REFER TO BACK PAGE FOR ALS LOCATIONS AND SAMPLING INFORMATION

WHITE - LABORATORY COPY YELLOW - CLIENT COPY

Failure to complete all portions of this form may delay analysis. Please fill in this form LEGIBLY. By the use of this form the user acknowledges and agrees with the Terms and Conditions as specified on the back page of the white - report copy.

1. If any water samples are taken from a Regulated Drinking Water (DW) System, please submit using an Authorized DW COC form.

SOL-673

ATTACHMENT D

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: 45.269N 75.804W

2023-02-01 19:21 UT

Requested by: 2726 - 2732 Moodie Drive

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)	0.415	0.225	0.133	0.040
Sa (0.1)	0.488	0.275	0.170	0.055
Sa (0.2)	0.410	0.236	0.148	0.051
Sa (0.3)	0.312	0.182	0.116	0.041
Sa (0.5)	0.222	0.130	0.083	0.029
Sa (1.0)	0.112	0.066	0.043	0.015
Sa (2.0)	0.053	0.031	0.020	0.006
Sa (5.0)	0.014	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.262	0.150	0.093	0.030
PGV (m/s)	0.185	0.104	0.064	0.020

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

References

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

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

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

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

Natural Resources Canada Ressources naturelles Canada

Canada

ATTACHMENT E

GUELPH PERMEAMETER TEST RESULTS

SoilMoisture
Soil Hydrology Instruments for Research Since 1955

Guelph Permeameter

🟩 **Input**
🟨 **Result**

Single Head Method - Feb 27, 2024

Reservoir Cross-sectional area in cm^2
(enter "35.22" for Combined and "2.16" for Inner reservoir): 2.18
Enter water Head Height ("H" in cm): 5
Enter the Borehole Radius ("a" in cm): 6

Enter the soil texture-structure category (enter one of the below numbers): 2

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min): 0.0100

* "R" = three values in a row with matching Δh/Δt

Res Type	2.18
H	5
a	6
H/a	0.833
a*	0.04
C0.01	0.524
C0.04	0.538
C0.12	0.489
C0.36	0.489
C	0.538
R	0.010
Q	4E-04
pi	3.142

α*= 0.04 *cm^{-1}*

C = 0.537554
Q = 0.000363

K_{fs} =	1.95E-07	*cm/sec*
	1.17E-05	*cm/min*
	1.95E-09	*m/sec*
	4.60E-06	*inch/min*
	7.66E-08	*inch/sec*

Φ_m = 4.87E-06 *cm^2/min*

Single Head Method - Nov 28, 2024

Reservoir Cross-sectional area in cm^2
(enter "35.22" for Combined and "2.16" for Inner reservoir): 2.18
Enter water Head Height ("H" in cm): 25
Enter the Borehole Radius ("a" in cm): 6

Enter the soil texture-structure category (enter one of the below numbers): 2

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min): 0.0333

* "R" = three values in a row with matching Δh/Δt

Res Type	2.18
H	25
a	6
H/a	4.16667
a*	0.04
C0.01	1.37818
C0.04	1.46966
C0.12	1.48715
C0.36	1.48715
C	1.46966
R	0.033
Q	0.00121
pi	3.1415

α*= 0.04 *cm^{-1}*

C = 1.469659
Q = 0.00121

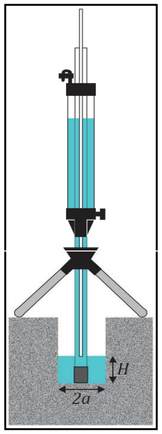
K_{fs} =	2.22E-07	*cm/sec*
	1.33E-05	*cm/min*
	2.22E-09	*m/ses*
	5.24E-06	*inch/min*
	8.73E-08	*inch/sec*

Φ_m = 5.54E-06 *cm^2/min*

Average

K_{fs} =	2.08E-07	*cm/sec*
	1.25E-05	*cm/min*
	2.08E-09	*m/s*
	4.92E-06	*inch/min*
	8.20E-08	*inch/sec*

Φ_m = 5.20E-06 *cm^2/min*



Calculation formulas related to shape factor (C). Where H_1 is the first water head height (cm), H_2 is the second water head height (cm), a is borehole radius (cm) and α^* is microscopic capillary length factor which is decided according to the soil texture-structure category. For one-head method, only C_1 needs to be calculated while for two-head method, C_1 and C_2 are calculated (Zang et al., 1998).

Soil Texture-Structure Category	α^*(cm^{-1})	Shape Factor
Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.	0.01	$C_1 = \left(\frac{H_2/a}{2.081 + 0.121\left(H_2/a\right)}\right)^{0.672}$
Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.	0.04	$C_1 = \left(\frac{H_1/a}{1.992 + 0.091\left(H_1/a\right)}\right)^{0.683}$ $C_2 = \left(\frac{H_2/a}{1.992 + 0.091\left(H_2/a\right)}\right)^{0.683}$
Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.	0.12	$C_1 = \left(\frac{H_1/a}{2.074 + 0.093\left(H_1/a\right)}\right)^{0.754}$ $C_2 = \left(\frac{H_2/a}{2.074 + 0.093\left(H_2/a\right)}\right)^{0.754}$
Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macro pores, etc.	0.36	$C_1 = \left(\frac{H_1/a}{2.074 + 0.093\left(H_1/a\right)}\right)^{0.754}$ $C_2 = \left(\frac{H_2/a}{2.074 + 0.093\left(H_2/a\right)}\right)^{0.754}$

Calculation formulas related to one-head and two-head methods. Where R is steady-state rate of fall of water in reservoir (cm/s), K_{fs} is Soil saturated hydraulic conductivity (cm/s), Φ_m is Soil matric flux potential (cm^2/s), a^* is Macroscopic capillary length parameter (from Table 2), a is Borehole radius (cm), H_1 is the first head of water established in borehole (cm) , H_2 is the second head of water established in borehole (cm) and C is Shape factor (from Table 2).

One Head, Combined Reservoir	$Q_1 = \bar{R}_1 \times 35.22$	$K_{fs} = \frac{C_1 \times Q_1}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi\left(\frac{H_1}{a^*}\right)}$ $\Phi_m = \frac{C_1 \times Q_1}{(2\pi H_1^2 + \pi a^2 C_1)a^* + 2\pi H_1}$
One Head, Inner Reservoir	$Q_1 = \bar{R}_1 \times 2.16$	
Two Head, Combined Reservoir	$Q_1 = \bar{R}_1 \times 35.22$ $Q_2 = \bar{R}_2 \times 35.22$	$G_1 = \frac{H_2 C_1}{\pi\left(2H_1 H_2(H_2 - H_1) + a^2(H_1 C_2 - H_2 C_1)\right)}$ $G_2 = \frac{H_1 C_2}{\pi\left(2H_1 H_2(H_2 - H_1) + a^2(H_1 C_2 - H_2 C_1)\right)}$ $K_{fs} = G_2 Q_2 - G_1 Q_1$ $G_3 = \frac{(2H_2^2 + a^2 C_2)C_1}{2\pi\left(2H_1 H_2(H_2 - H_1) + a^2(H_1 C_2 - H_2 C_1)\right)}$ $G_4 = \frac{(2H_1^2 + a^2 C_1)C_2}{2\pi\left(2H_1 H_2(H_2 - H_1) + a^2(H_1 C_2 - H_2 C_1)\right)}$ $\Phi_m = G_3 Q_1 - G_4 Q_2$
Two Head, Inner Reservoir	$Q_1 = \bar{R}_1 \times 2.16$ $Q_2 = \bar{R}_2 \times 2.16$	

NOTES:

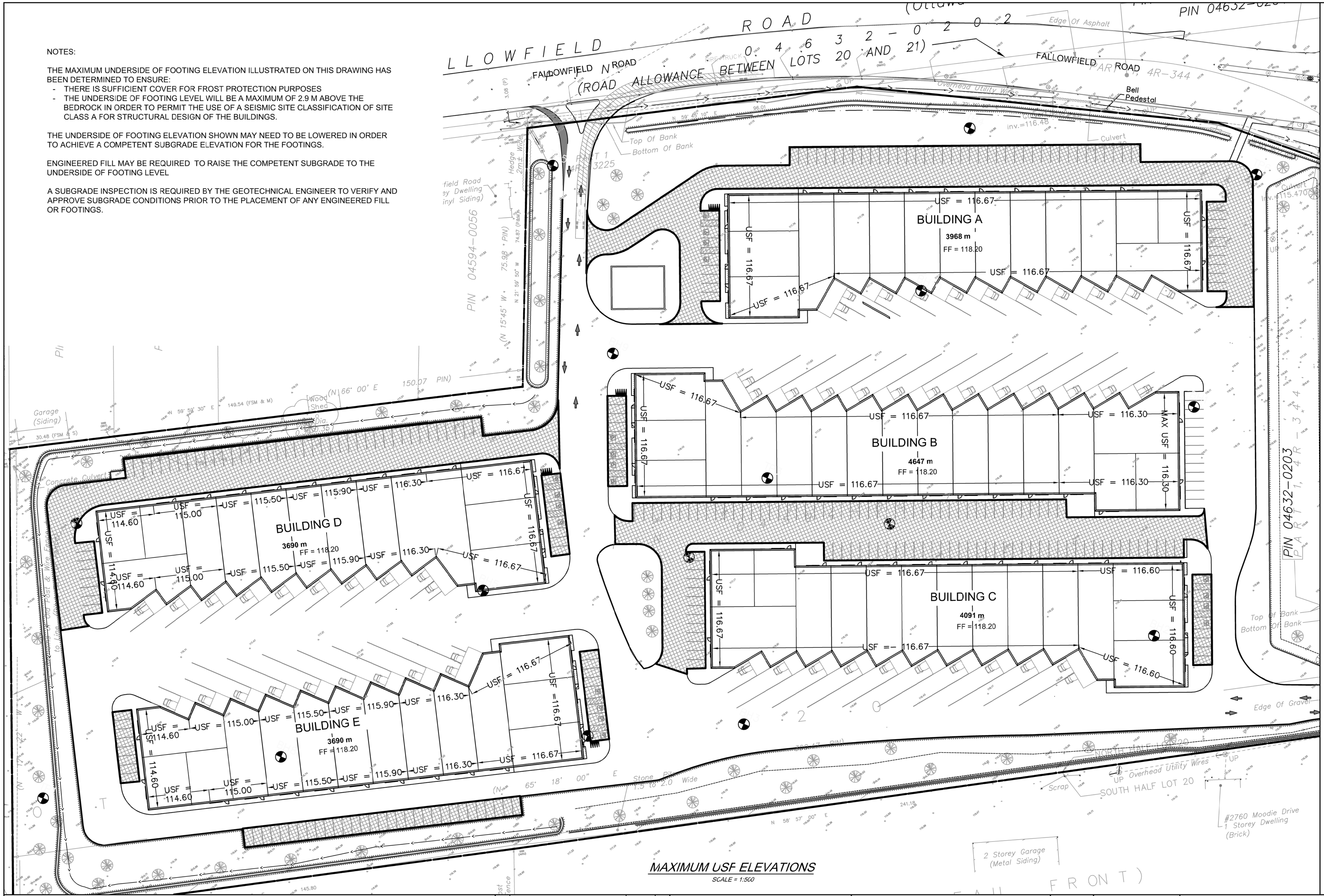
THE MAXIMUM UNDERSIDE OF FOOTING ELEVATION ILLUSTRATED ON THIS DRAWING HAS BEEN DETERMINED TO ENSURE:

- THERE IS SUFFICIENT COVER FOR FROST PROTECTION PURPOSES
- THE UNDERSIDE OF FOOTING LEVEL WILL BE A MAXIMUM OF 2.9 M ABOVE THE BEDROCK IN ORDER TO PERMIT THE USE OF A SEISMIC SITE CLASSIFICATION OF SITE CLASS A FOR STRUCTURAL DESIGN OF THE BUILDINGS.

THE UNDERSIDE OF FOOTING ELEVATION SHOWN MAY NEED TO BE LOWERED IN ORDER TO ACHIEVE A COMPETENT SUBGRADE ELEVATION FOR THE FOOTINGS.

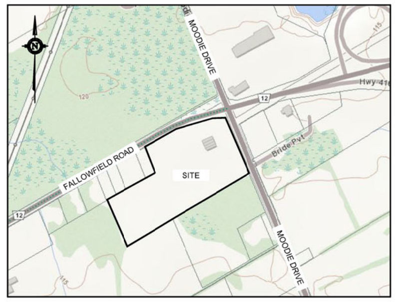
ENGINEERED FILL MAY BE REQUIRED TO RAISE THE COMPETENT SUBGRADE TO THE UNDERSIDE OF FOOTING LEVEL

A SUBGRADE INSPECTION IS REQUIRED BY THE SUBGRADE GEOTECHNICAL ENGINEER TO VERIFY AND APPROVE SUBGRADE CONDITIONS PRIOR TO THE PLACEMENT OF ANY ENGINEERED FILL OR FOOTINGS.



MAXIMUM USF ELEVATIONS

SCALE = 1:500



KEY PLAN

NOT TO SCALE

0 25 50
1:750 METRES

LEGEND

+ 74.50	EXISTING ELEVATION
◐	BOREHOLE LOCATIONS (JAN 30 - FEB 2, 2023)
◐	TEST PIT LOCATIONS (APR 14, 2023)
	MAXIMUM ELEVATION FOR THE UNDERSIDE OF FOOTING THAT WOULD ENSURE BOTH SUFFICIENT SOIL COVER FOR FROST PROTECTION PURPOSES AS WELL AS ENSURING THE USF IS A MAXIMUM OF 2.9 METRES ABOVE THE BEDROCK
	FINISHED FLOOR ELEVATION
▨	LIGHT DUTY PAVEMENT STRUCTURE

NOTES:
1. ALL DIMENSIONS ARE IN METRES, UNLESS OTHERWISE SPECIFIED; ALL ELEVATIONS ARE IN METRES.
2. THIS IS NOT A LEGAL SURVEY.
3. EXISTING SERVICES INFORMATION SHOWN ARE BASED ON BEST CURRENT INFORMATION. CONTRACTOR TO VERIFY EXACT LOCATION AND REPORT ANY DISCREPANCIES TO KOLLAARD ASSOCIATES INC.
4. CLIENT IS RESPONSIBLE FOR ACQUIRING ALL NECESSARY PERMITS.
5. CONTRACTOR TO VERIFY THAT APPROPRIATE PERMITS HAVE BEEN ACQUIRED PRIOR TO ANY CONSTRUCTION.
6. CONTRACTOR IS RESPONSIBLE FOR LOCATION AND PROTECTION OF UTILITIES.
7. ALL DIMENSIONS TO BE VERIFIED ON SITE BY CONTRACTOR PRIOR TO CONSTRUCTION.
8. THIS DRAWING IS NOT FOR CONSTRUCTION UNTIL ALL APPROVALS HAVE BEEN GRANTED.
9. INSPECTION OF ROUGH GRADE BY KOLLAARD ASSOCIATES INC. AND CITY OF OTTAWA MUST BE CONDUCTED PRIOR TO PLACEMENT OF TOPSOIL OR SOD.
10. HYDRO SERVICE TO BE INSTALLED ACCORDING TO THE SPECIFICATIONS OF SERVICE PROVIDER AND THE MECHANICAL ENGINEER.
11. ALL MATERIALS AND CONSTRUCTION TO BE IN ACCORDANCE WITH CITY OF OTTAWA STANDARDS AND ONTARIO PROVINCIAL STANDARDS AND SPECIFICATIONS
12. ANY CHANGES MADE TO THIS PLAN MUST BE VERIFIED AND APPROVED BY KOLLAARD ASSOCIATES, INC.
13. THIS DRAWING IS PART OF KOLLAARD ASSOCIATES DESIGN REPORT #221099.

No.	REVISION	DATE	BY

CONSULTANTS

Kollaard Associates
Engineers

BOX 189
210 PRESCOTT STREET
KEMPTVILLE, ONTARIO
K0G 1J0
FACSIMILE (613) 258-0475

(613) 860-0923

DESIGN	SD
DRAWN	SD
CHECKED	SD
APPROVED	SD

STAMP

CLIENT NAME	1000198532 ONTARIO INC.
PROJECT NAME	PROPOSED WAREHOUSING
PROJECT LOCATION	2726-2732 MOODIE DRIVE, OTTAWA, ON
DRAWING	MAXIMUM FOOTING ELEVATION (FROST AND SEISMIC)
PROJECT No.	221099
DATE	2025.OCT.24
SCALE	AS_NOTED
DRAWING No.	221099-USF

ARCH D