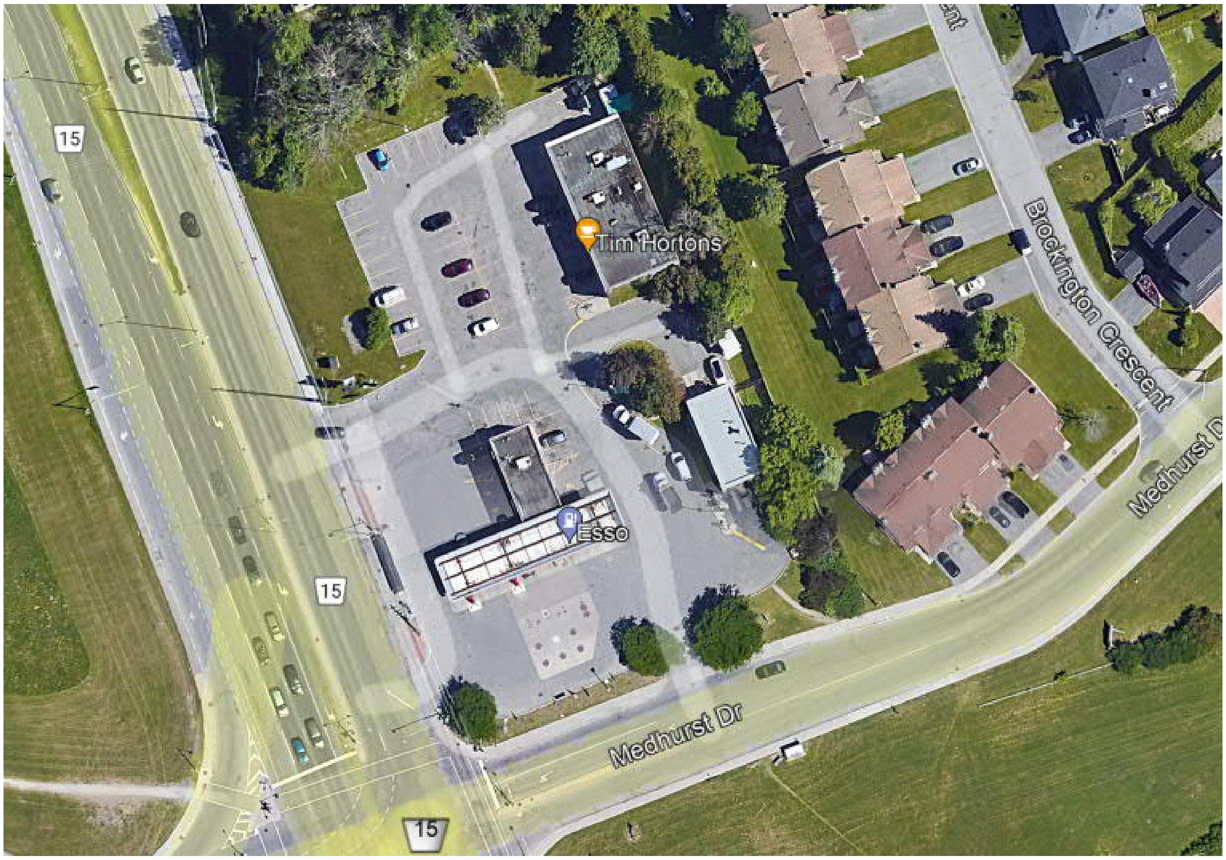
GEOTECHNICAL INVESTIGATION AND DESIGN REPORT
CIRCLE K STATION
1545 WOODROFFE AVE., OTTAWA, ON



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Prepared for:

Circle K Canada Division

REVISED REPORT

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1.0 INTRODUCTION

McIntosh Perry Consulting Engineers (McIntosh Perry) was retained by Circle K (the Client) to complete this foundation investigation and pavement recommendation report. This report presents the factual findings obtained from a geotechnical investigation performed for the proposed reconstruction of the gas station at Woodroffe Avenue and Medhurst Drive in Nepean, at the south side of Ottawa, Ontario. A geotechnical investigation was carried out on August 18th, 2021, and comprised of eight (8) boreholes, to a maximum depth of 8.2 m below the existing surface.

The purpose of the investigation was to explore and understand the subsurface conditions based on field investigations, laboratory testing, and other background information that may be available in the public domain, to enable us to address the geotechnical design objectives on this project. This report provides anticipated geotechnical conditions influencing the design and construction of the proposed development which includes reconstruction of the gas station, underground tanks, store building and the carwash at various locations. Design recommendations are offered based on the authors' interpretation of the subsurface investigation and test results. The readers are referred to Appendix E, Limitations of Report, which is an integral part of this document.

2.0 PROJECT UNDERSTANDING

The proposed development plan is shown in Figure 3, the proposed site plan, in Appendix A. The site plan shows the proposed location of pumps, store, restaurant, carwash, and underground tanks. The aerial image of Figure 2 in Appendix A shows the current layout of the site. The proposed location of the underground tanks is different from the current location. The site is proposed to be asphalt paved similar to its current condition.

3.0 BACKGROUND INFORMATION AND SITE DESCRIPTION

3.1 Local Geology

Based on the published physiography maps of the area (Ontario Geological Survey), the site is located within the Ottawa Valley Clay Plains. Surficial geology maps of southern Ontario indicate the proposed development is located within organic deposits such as peat, muck, or marl bordered by fine-textured glaciomarine deposits composed mainly of silt and clay at its north. The organic plain is bordered at its south by a mix of clay, silt, sand, and gravel with organic remains.

Bedrock geology maps of Southern Ontario indicate the bedrock formation which covers the project area to be dolostone or sandstone of Beekmantown group.

The Ottawa Valley between Pembroke and Hawkesbury, Ontario, consists of clay plains interrupted by ridges of rock or sand. It is naturally divided into two parts, above and below Ottawa, Ontario. Within the valley, the bedrock is further faulted causing some of the uplifted blocks to appear above the clay beds.

3.2 Site Description

The gas station site is located at the northeast corner of Medhurst Drive and Woodroffe Avenue in Nepean, Ontario. The entire site, including the buildings, pumping stations, and parking area, is approximately 8,700 m^2. The site includes a building that hosts the convenience store and the other one which used to be a coffee shop, an automated carwash, three rows of fuel pumps and an underground tank area. The site also includes other ancillary facilities such as air and vacuum stations. The gas station is accessed from both Medhurst Drive and Woodroffe Avenue from its south and west sides. From the north and east, the site is bordered by residential townhouses. The site topography was observed to be relatively flat. Site location and site layout are shown in Figures 1 and 2 in Appendix A.

4.0 FIELD INVESTIGATION

McIntosh Perry staff visited the site before the drilling investigation to mark out the proposed borehole locations. Utility clearance was carried out by USL-1, a private locator on behalf of McIntosh Perry. Public and private utility authorities were informed, and all utility clearance documents were obtained before the commencement of drilling work.

Eight boreholes were marked for drilling. Upon completion of the utility locate process, all boreholes were drilled. The equipment used for drilling was owned and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario. Boreholes were advanced using hollow stem rotary augers aided by a truck-mounted CME 75 drill rig. Boreholes were advanced to a maximum depth of 8.2 m below the ground level (El. 80.7 m) in borehole BH21-2. Soil samples were obtained at 0.75 m intervals in boreholes using a 51 mm outside diameter split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. Since native fine-textured cohesive soil was encountered, when possible, SPT sampling was alternated with field shear vane testing to measure in-situ undrained shear strength. Five monitoring wells were installed in boreholes BH21-1, BH21-2, BH21-3, BH21-5, and BH21-6. Monitoring wells were capped with cast aluminum lids flushed to the pavement surface. Well assembly is shown on borehole logs included in Appendix B. The rest of the boreholes were backfilled with auger cuttings, sealed with bentonite hole plug, and restored to the original surface with cold patch asphalt. Borehole locations are shown in Figure 2, included in Appendix A.

5.0 LABORATORY INVESTIGATIONS

All samples were tagged and logged as retrieved, and visual description and soil type identification were added to the field logs. Subsequently, soil descriptions were confirmed by additional tactile examination of the soils in the laboratory. Laboratory testing on representative SPT spoon samples was performed at the McIntosh Perry geotechnical lab in Ottawa and included measurement of natural moisture content, grain size distribution, and Atterberg Limit tests. The laboratory tests to determine index properties were performed in accordance with the American Society for Testing Materials (ASTM) test procedures.

The relevant test procedures adopted are listed below:

ASTM D2216 – Laboratory Determination of Water Content of Soil and Rock by Mass
ASTM C117 –Materials Finer than 75 µm (No. 200) Sieve by Washing (LS-601)
ASTM C136 – Sieve Analysis of Fine and Coarse Aggregates (LS-602)
LS-702 – Determination of Particle Size Analysis of Soils
ASTM D4318 – Liquid Limit, Plastic Limit, and Plasticity Index of Soils (LS-703/704)

The rest of the soil samples recovered will be stored in a McIntosh Perry storage facility for a period of three months after submission of the final report. Samples will be disposed of after this period unless otherwise requested in writing by the Client.

6.0 SUBSURFACE CONDITIONS

6.1 Subsoil Conditions

In general, the site stratigraphy consists of asphalt, granular material as pavement structure, grading fill, silty sand to sandy silt, over clayey silt to silty clay, over sand. For classification purposes, site stratigraphy can be divided into four different zones.

a) Pavement Structure and Grading Fill
b) Silty Sand to Sandy Silt
c) Clayey Silt to Silty Clay
d) Sand

Site stratigraphy encountered during the investigation, together with the field and laboratory test results, are shown on the Record of Borehole sheets included in Appendix B. In addition, laboratory summary test results are shown in Table 6-1 and Table 6-2.

6.1.1 Pavement Structure and Grading Fill

The top of asphalt elevations ranged from 88.4 to 89.0 across the site. Asphalt thickness observed on average was approximately 90 mm across the site ranging from 38 mm to 150 mm. Thicker asphalt is expected on those areas with a patch over depressions. The asphalt surface is underlain by a layer of sand and gravel functioning as the pavement structure. In some occasions a similar material was used as the grading fill whereas, in some occasions the fill was observed to be silty sand similar to the native soil. Sample GS-1 from borehole BH21-1 was tested for grain-size distribution and the test results are shown in Figure 4 in Appendix C. In this figure, the gradation test result is compared with OPSS Granular B Type III specifications. The test result indicated 44% Gravel, 41% Sand, and 15% fine particles smaller than 75 microns and the grain size distribution curve of the tested sample shown in Figure 4 does exceeds the OPSS 1010 Granular B Type III grading envelope. The standard allows for a maximum of 8% to 10% fines based on the aggregate source.

A distinctive layer of organic soil, approximately 0.3 m thick, encountered below the fill in boreholes BH21-1 and BH21-8 are interpreted as native organic soil which was not cleared during the previous construction.

6.1.2 Silty Sand to Sandy Silt

A layer of silt and sand with various proportions was observed below the grading fill and above the clay. The bottom of this layer ranges between El. 85.0 and 85.9. SPT ‘N’ value is between 3 to 16 and the layer is categorized as very loose to compact. Three samples from this layer were tested for grain-size distribution and the test results are presented in Figure 5 in Appendix C. Test results are also summarized in Table 6-1.

Table 6-1: Grain Size Analysis – Silty Sand to Sandy Silt

Sample	Size Fraction (%)			
	Gravel	Sand	Silt	Clay
BH21-2/SS-4	0	61	31	8
BH21-3/SS-5	4	46	40	10
BH21-8/SS-4	0	25	57	18

6.1.3 Clayey Silt to Silty Clay

This layer varied across the site from silty clay to clayey silt. Gradation test results of this layer are shown in Figure 6 and the plasticity chart in Figure 8 is included in Appendix C. Intercepted top elevation of the deposit ranged from 85.0 to 85.9 and the bottom of the layer ranged from El. 84.2 to 83.4 underlain by sand layer. In borehole BH21-8 the silty clay layer extended to the termination depth of the borehole to El. 82.3. The liquid limit of this layer ranged from 21 to 30 as tested. The plasticity index ranged from 5 to 15. The gravimetric water content as tested for this layer ranged between 31% to 44%. Laboratory test results completed for this layer are shown in Table 6-2.

Table 6-2: Grain Size Analysis – Silty Clay to Clayey Silt

Sample	Size Fraction (%)				Atterberg Limits (%)			wc (%)	Liquidity Index
	Gravel	Sand	Silt	Clay	LL	PL	PI		
BH21-3/SS-7	0	5	67	27	---	---	---	---	---
BH21-4/SS-6	0	2	66	31	30	17	13	40	1.77
BH21-5/SS-4	---	---	---	---	33	18	15	44	1.73
BH21-7/SS-5	---	---	---	---	21	16	5	31	3.00

Based on the plasticity chart, the deposit can be classified as low plasticity. A liquidity index of greater than 1 indicates the sensitive state of the soil. In-situ Undrained Shear strength was measured with the field vane share apparatus for those strata with higher clay portions. The measured undrained shear strength ranged between 39 and 46 kPa with sensitivity between 4 and 11. The cohesive stratum is classified as “firm” based on the undrained shear strength. Soils with sensitivity values between 4 and 8 are classified as Sensitive, and between 8 to 16 are classified as Extra Sensitive.

6.1.4 Sand

The silty clay layer is underlain by a layer of loose to compact sand. The SPT ‘N’ values range between 3 and 16 with an average value of 9. Sand extends to the end of the investigation in all boreholes except borehole BH21-8 which terminated within the silty clay stratum. Gradation results of a representative sample of this layer are presented in Figure 7 in Appendix C. The gradation results indicated 4% gravel, 95% sand, and 1% fines. Sand is classified as ‘poorly graded’.

6.2 Groundwater Level Observations

Groundwater was measured in monitoring wells two weeks after drilling. Groundwater monitoring results are shown in Table 6-3.

Table 6-3: Groundwater Monitoring Records

BH No.	Drilling Date	Top of Filter Pack El. (m)	Bott. of Filter Pack		Monitoring Date	GW in Well	
			Depth (m)	El. (m)		Depth (m)	El. (m)
BH21-1	Aug. 17, 2021	86.1	5.9	82.8	Sep. 03, 2021	2.7	86.0
BH21-2	Aug. 17, 2021	85.0	7.6	81.4	Sep. 03, 2021	4.4	84.6
BH21-3	Aug. 17, 2021	85.2	7.0	81.9	Sep. 03, 2021	4.8	84.1
BH21-5	Aug. 18, 2021	85.7	6.1	82.3	Sep. 03, 2021	4.7	83.7
BH21-6	Aug. 18, 2021	85.9	6.1	82.3	Sep. 03, 2021	3.3	85.1

These measured groundwater levels are susceptible to change due to seasonal and extreme climatic/rainfall events.

6.3 Chemical Analysis

The chemical tests were conducted by Paracel Laboratories in Ottawa, ON., to determine the resistivity, pH, sulphate and chloride content of two representative soil samples collected from two different boreholes. The samples were chosen from within the expected foundation depths and shown in Table 6-4.

Table 6-4: Soil Chemical Analysis Results

BH No.	Sample	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (Ohm-cm)
BH21-1	SS-4	2.3 – 2.9	7.76	0.0059	0.0156	2470
BH21-5	SS-4	3.1 – 3.7	7.68	0.0206	0.0492	7810

7.0 DISCUSSION AND RECOMMENDATIONS

7.1 General

This section of the report provides general and project-specific guidelines for the design and construction of the proposed development. The provided recommendations are only intended for the defined limits and the scope of the project. These recommendations shall not be used without the consent of the authors for engineering applications other than those specified in this scope of work. The readers are referred to Limitations of Report included in Appendix E.

The design team should satisfy themselves of the adequacy of the information presented in this report for their intended design. The constructors must make their own interpretation of the provided factual information as it affects their construction equipment, efficiency, schedule, and the safety of their staff.

The foundation recommendations presented in this section have been developed following Part 4 of the 2012 Ontario Building Code (OBC) extending the Limit State Design approach.

7.2 Scope of Foundation Design

It is understood that all proposed structures will be low-rise and light frame buildings including the corner store, the restaurant, carwash tunnel, and the pumping station canopy. Also, all buildings are without basements.

Information regarding the finished floor elevation of the proposed development is provided in Grading Plan drawing C01, submitted on December 3, 2021. Based on this drawing the finished floor elevation of the store and the restaurant is El. 88.96, and the finished floor elevation of the carwash is El. 88.88. Site grading does not indicate any substantial grade raise. There are only minor grade adjustments proposed for drainage and access purposes.

7.3 Ground Characterization

7.3.1 Overview of Subsurface conditions

The existing site stratigraphy consists of a pavement structure and grading fill over a layer of silty sand to sandy silt, over silty clay to clayey silt, underlain by sand.

For conventional construction, footings are expected to be bearing within the silty sand and sandy silt layer. The silty sand layer is classified as loose to compact. The stress bulb below the footings is expected to influence the silt and clay layer below the silty sand.

All footings shall be constructed below frost penetration depth at minimum 1.8 m below surface. The expected founding elevation is above the observed groundwater level. Seismic Site Classification

Seismic site classification is completed based on OBC 2012 Section 4.1.8.4 and Table 4.1.8.4.A. This classification system is based on the average soil properties in the upper 30 m and accounts for site-specific shear wave velocity, standard penetration resistance, and plasticity parameters of cohesive soils. Selected spectral responses in the general vicinity of the site are provided in Appendix D.

Based on the subsurface condition, field and SPT values, and undrained shear strength of clay, the site can be classified as Seismic Site Class (E).

7.4 Foundation Design

Foundation design recommendations are prepared based on the assumption of shallow footings with no basement. The groundwater level is noted in various boreholes at El. 86.0 the highest and El. 83.7 the lowest groundwater table recorded. The founding level is expected at approximately El. 87 ± 0.3 m. The bearing capacities are calculated based on these anticipated elevations.

7.4.1 Geotechnical Resistance

The governing mode of the design is the controlling stress for tolerable settlement. In the absence of structural design, hypothetical cases of 1 m and 2 m wide (shorter dimension) were considered for strip and spread footings. It is recalled that the top of the silty clay layer is 2.6 m to 4.0 m below the existing grades across the site.

In the calculation of bearing capacities, the internal friction angle was considered 28° for the silty sand to sandy silt layer. For the strip footings, the stress bulb is deeper and will be influenced by the silty clay layer which is significantly more compressible than the overlying sandy silt/silty sand layer. Also presumed the groundwater table remains lower than the founding level, as measured on site. The proposed frost cover of 1.8 m is considered as the surcharge over the founding elevation for the calculation of ultimate bearing capacities. The bearing resistances are calculated for founding level at El. 86.5 or higher.

Table 7-1: Geotechnical Resistance for Specific Shallow Footing Case

Footing Type and Size	SLS (kPa)	ULS (kPa) Factored
Strip Footing 1 m wide	85	125
Spread Footing 1 m wide (square)	90	170
Strip Footing 2 m wide	75	110
Spread Footing 2 m wide (square)	110	215

The calculation for SLS geotechnical resistance is for a total settlement of 25 mm and differential settlement of 19 mm.

Grade adjustment may be needed below the founding level in case of over excavation. Grade adjustment can be done by placing OPSS Granular A and compacting to 100% Standard Proctor Maximum Dry Density (SPMDD).

7.4.2 Frost Protection

Based on the subsurface investigation results, the encountered native sandy silt to silty sand is classified as moderate to high frost susceptibility material due to its high silt content. Frost penetration depth is 1.8 m below the finished grade for the subject site. Frost penetration depth is estimated based on the OPSD 3090.101, Foundation Frost Penetration Depths for Southern Ontario.

All perimeter and exterior foundation elements, or interior foundation elements in unheated areas should be provided with a minimum of 1.8 m of earth cover or equivalent thermal rigid insulation for frost protection purposes. Frost penetration depth can be reduced to 1.5 m for heated structures when the heat flux is not blocked through the foundation.

7.4.3 Slabs-on-Grade

Slab-on-grade is considered free-floating (not attached to the foundation walls) and should be supported on a minimum of 200 mm of Granular A bedding compacted to 100% SPMDD. The requirements of the fill underneath slab-on-grade is noted in Section 7.6.

If the slab on grade is proposed to support concentrated linear or point loads, the design loading shall be indicated in the structural specifications.

Given the relatively low groundwater table, and since the proposed site grade is designed for positive drainage, there are no concerns with potential saturation of the slab-on-grade granular bedding for the store and restaurant

However, for the case of the carwash, to prevent hydrostatic pressure build up beneath the floor slab, it is recommended that rigid perforated pipes are used to provide drainage for the granular base beneath the floor slab. The drainage pipes shall be installed below the moisture barrier and should be provided with a 200 mm of clear stone. This moisture barrier should be placed between the floor slab and the granular bedding. To prevent cross migration of fine particles between subgrade and the clear stone, the clear stone should be fully wrapped with a Class II nonwoven geotextile.

It is recommended that subgrade preparation and compaction efforts are approved under the supervision of a geotechnical representative.

If the slab-on-grade is to be designed for structural function, the modulus of subgrade reaction (k) might be needed for the design of the slab-on-grade to support local loads in the basement. Simplified assumptions were made to estimate the spring modulus for slab-on-grade on compacted Granular A. To estimate the modulus of subgrade reaction, it was assumed that a 2 m square section of the concrete slab-on-grade under the applied loads supported on the granular bedding as noted above. Since the modulus of subgrade reaction is needed for the design of the slab, a subgrade reaction coefficient of 10 MN/m^3 can be used for the design of the interior slab-on-grade. This k-value is only valid for the construction of slab-on-grade on compacted Granular A bedding. This value shall not be used for the native subgrade.

7.5 Lateral Earth Pressure

Free draining material should be used as backfill below surface. The following parameters are recommended for the granular backfill. Where the wall support and structure allow lateral yielding, for example for unrestrained retaining walls, active earth pressure may be used in the geotechnical design of the wall. Where the support does not allow lateral yielding, such as for the basement walls, at-rest conditions can be applied for the geotechnical design. The unfactored parameters given in Table 7-2 may be used for free draining materials.

Table 7-2: Lateral Pressure Parameters for Granular Backfill

Pressure Parameter		Expected Value			
		Granular A	Granular B	Other OPSS1010 'Granular'	Native Silty Sand
Unit Weight (γ) kN/m^3	Above groundwater	22.5	21.7	21.7	19.0
	Below groundwater	12.7	11.9	11.9	9.2
Angle of Internal Friction (ϕ')		35°	32°	31°	28°
Coefficient of Active Earth Pressure (k_a)		0.27	0.31	0.32	0.36

Coefficient of Passive Earth Pressure (k_p)	3.69	3.23	3.12	2.77
Coefficient of Earth Pressure at Rest (k_o)	0.43	0.47	0.48	0.53

If a shored excavation is used as part of the formwork for the wall, the lateral earth pressures for the foundation walls are based on the existing retained soils (i.e., overburden) and the unfactored lateral earth pressure parameter given in Table 6-1 may be used.

7.6 Site Preparation

Excavation for construction of footings shall be extended to the native subgrade as the existing fill is not considered as engineered fill and was found with debris and decomposed material. Before excavation, if groundwater is higher than those observed in monitoring wells, groundwater shall be lowered to approximately 1 m below the excavation depth to reduce the risk of overly disturbing the natural subgrade. Based on the current understanding of the groundwater levels observed in the monitoring wells (varied from 2.7 m to 4.7 m), no dewatering, to possibly some minor dewatering is anticipated for foundation construction. Dewatering requirement for installation or proposed tanks are discussed in details in Section 8.

All fill and disturbed subgrade shall be removed from the footprint of the footings. If over excavation (below proposed founding level) becomes necessary, the grade adjustment shall be done using OPSS Granular A compacted to a minimum of 100% SPMDD to underside of the footings. The Granular bedding shall extend beyond the edge of the footings from either side for a minimum equal to the fill thickness. The granular fill shall be separated from the subgrade by a layer of Class II Non-woven geotextile. Alternatively grade adjustment can be achieved by lean concrete. If the grade adjustment becomes substantial, the authors of this report shall be informed to check the risks of additional consolidation settlements induced by the difference in the unit weight of the concrete fill in comparison to the native soil.

The prepared subgrade shall be reviewed and approved by geotechnical staff before footing construction.

7.7 Excavation and Supporting System

Any excavation deeper than 1.2 m shall be either shored or to receive proper slope to maintain stability. Designing the shoring system or stability check of the excavation slope is the general contractors' responsibility. In accordance with the Occupational Health and Safety Act (OHSA) of Ontario and based on engineering experience, the overburden within the limits of the project can be classified as Type 4 Soil, for both above and below groundwater. Unsupported excavation should not be sloped steeper than 3H:1V. Excavation can be advanced with side slopes not steeper than 3H:1V or need to be shored. If space restrictions exist, excavations deeper than 1.2 m can be supported by a temporary support system in compliance with OPSS.MUNI 539, OHSA, O.Reg. 213/91.

For shored excavations, dewatering of the cohesionless soils or especially, if excavations go into the cohesive deposit, depressurization of the clayey silt deposit will be required to increase the stand-up time as the presence of silt or fine sand seams cannot be ruled out that will significantly compromise unsupported stand-up times. The hydrological specialist contractor should be consulted for dewatering issues and management of such issues is considered the responsibility of the specialist contractor.

Excavation for the proposed shallow footings is not expected to affect the neighboring structures for the anticipated founding depths as noted in this report, unless positive dewatering is undertaken.

Should groundwater lowering becomes inevitable based on specialist contractor advice, authors of this report should be informed to assess the implications on surrounding structure settlements.

Based on the information obtained from this drilling program, an application for a Permit to Take Water (PTTW) is not required. However, some positive dewatering is expected during shoring and excavation and application for an Environmental Activity and Sector Registry (EASR) is recommended.

7.8 Cement Type and Corrosion Potential

Two soil samples were submitted to Parcel laboratories for testing of chemical properties relevant to exposure of concrete elements to sulphate attacks as well as potential soil corrosivity effects on buried metallic structural elements. Test results are presented in Table 6-4.

The potential for sulphate attack on concrete structures is moderate to low. Therefore, Type GU Portland cement may be adequate to protect buried concrete elements in the subsurface conditions encountered.

Based on electrical resistivity results and chloride content, the corrosion potential for buried steel elements is within the slightly aggressive to nonaggressive range.

8.0 CONSTRUCTION OF UNDERGROUND TANKS

It is understood both stormwater retention tanks and fuel tanks will be installed or replaced as part of the proposed project. All recommendations included in the above segments such as site preparation, dewatering, design for lateral backfill pressure, excavation and shoring, and foundation recommendations should be generally applicable for the design of underground tanks, should they need to be replaced.

It is understood the bottom of the fuel tanks will be approximately 4.5 m below the proposed finished grade and the bottom of the underground stormwater storage chamber will be approximately 4.1 m below the surface. Based on the average proposed surface elevations shown on drawing C01, Grading Plan (sub. Dec. 3, 2021), the bottom elevation of both storm tank and fuel tanks will be at an approximate elevation of El. 84.2 ± 0.2.

Also, based on the groundwater level measurements completed in the five monitoring wells installed on site, the average groundwater level is at El. 84.7. Therefore, it is quite probable that subgrade preparation needs to extend to a level below the existing water table for tank installation. The buoyance effects applied on both structures may disrupt their function based on fluctuations of the groundwater table. The following recommendation shall be used for the construction of the fuel tanks and stormwater retention chamber.

8.1 Excavation Stability for Tank Installation

Based on the borehole data, the stormwater retention chamber is most likely founded with the sand layer. By interpolating the stratigraphy between boreholes BH21-5 and BH21-6, the fuel tanks are expected to be underlain by partially sand and partially clay subgrade. Excavation for both structures has to be dug through the existing fill. Before any excavation, the groundwater shall be lowered to an elevation lower than the proposed bottom of the pit (not less than 0.6 m lower) and then provide shoring and support for the excavation. The contractor is responsible to provide detailed design for shoring.

8.2 Dewatering for Tank Installation

Hydraulic conductivity of the silty sand layer underlain the clay is expected to govern dewatering at this site. Hydraulic conductivity of the sand is expected to be of the order of 10^{-6} m/s order of magnitude which should be accounted for dewatering. A commonly used dewatering method is through the installation of a series of wellpoints driven to the target dewatering depth but this is the responsibility of the dewatering contractor to design the dewatering method.

The noted groundwater lowering below the bottom of the excavation, 0.6 m, is assuming there will be no granular compaction at the tank bedding or for backfill. No compaction should be carried unless at least there is 1 m between the compaction level and the groundwater table.

Based on the expected amount of groundwater extraction a PTTW is unlikely to be needed and an EASR application may be adequate. The contractor is ultimately responsible to follow regulations.

8.3 Tank Foundation, Subgrade, and backfill

Once dewatering is achieved, a shoring and soil retention system is in place, and excavation is extended to the required depth the subgrade shall be immediately covered with lean concrete upon excavation. Since the compaction is not allowed on the subgrade, lean concrete can provide adequate protection against the excessive subgrade disturbance and it can also function as a level surface for the basal concrete slab on which the tank rests following manufacturers bedding/haunch material criteria. In view of the groundwater levels measured, a tanked chamber is recommended to practically minimize in-service groundwater impact issues on the buried tanks.

The fuel tanks are expectedly plastic and they will be prone to buoyancy when empty. The buoyancy forces can even cause damage to the fuller tanks due to the lower unit weight of petroleum products in comparison to water. Therefore, the fuel tanks should be strapped to a basal concrete slab to resist the tank floating. We recommend housing the tanks in a concrete chamber to isolate the direct buoyancy effects of the groundwater on the tanks. Lateral earth parameters are provided in previous sections.

It is the responsibility of the designer to consider groundwater uplift on the design of the stormwater retention chamber.

The following geotechnical resistance values can be used for the design of the tanks;

At the noted elevations (4.1 m to 4.5 m);

Serviceability Limit State (underground tanks) = 80 kPa

Ultimate Limit State (underground tanks) = 120 kPa

Compaction is not allowed for backfilling of both tanks until at least one meter above the groundwater table at any given state of dewatering.

It should be noted that unshrinkable fill or commonly referred to as u-fill is different than lean concrete. The u-fill tends to degrade over time from its initial strength. Although still suitable to be used as a backfill, it may not be used at foundation subgrade. Instead, lean concrete shall be used. Lean concrete with 10 ± 2 MPa at 28 days compressive strength is adequate for this application.

To emphasize, all dewatering requirements as indicated in section 8.2 shall remain in place until both lean concrete and unshrinkable fill are set.

9.0 CONSTRUCTION CONSIDERATIONS

Any organic material and loose soil of any kind should be removed from the footprint of the building and all structurally load-bearing elements. Site preparation and requirements of engineered fill placement are noted throughout previous sections. Refer to relevant sections for material and compaction requirements.

All backfilling shall comply with the OPSS.MUNI 501 and the City of Ottawa Special Provision General No. D-029 for compaction requirements, unless the design recommendations included in this report exceed provisions of OPSS.MUNI 501 and D-029.

The construction of proposed development as discussed earlier can be carried out by open-cut excavation and sloped or shored excavation based on the construction plan. Excavation and dewatering considerations are discussed in previous sections.

The excavated subgrade must be kept dry at all times to minimize disturbance to the subgrade. If dewatering is needed, it shall be extended to a minimum of 0.6 m to 1 m below the subgrade before compaction and proof rolling. Otherwise, groundwater lowering to 0.3 m below subgrade to allow a ‘dry’ subgrade is adequate.

The silty sand/sandy silt layer is highly prone to disturbance to construction traffic. A minimum 50 mm mud mat say with lean concrete should be placed following inspection and approval of the founding subgrade to preserve the subgrade from construction traffic and exposure to the elements.

Disturbed subgrade shall be excavated and replaced with Granular A compacted to 100% SPMDD or lean concrete.

A geotechnical engineer or technician should attend the site to confirm subgrade excavation, type of fill material and level of compaction.

The native soil is of high frost susceptibility. Care must be taken during winter construction. Footings constructed on frozen soils during wintertime will be subject to major shift and damage once the frost is thawed in springtime.

All earthworks must be protected from freezing. Backfilling materials should not be placed on frozen ground as per OPSS.MUNI 902.

The engineered fill to adjust the grade up to below the pavement structure can be Granular A or Granular B (types 1 through 3) as identified in OPSS.MUNI 1010.

Since installation of underground tanks requires dewatering, it is preferred to schedule for tank installation and associated dewatering before construction of the proposed footings. This is to mitigate risk of unfavorable settlements induced by increase of effective stress below the footings subsequent to dewatering.

A geotechnical engineer or technician should attend the site to confirm the subgrade, type of fill material, and level of compaction. All bearing surfaces should be reviewed by experienced geotechnical personnel before pouring the concrete to ensure that strata having adequate bearing capacity have been reached, and the bearing surfaces have been properly prepared.

10.0 PAVEMENT AND HARD SURFACING

10.1 Flexible Pavement

For the majority pf the site, the pavement structure is most likely to be placed on engineered fill material overlaying the native soil.

SPT sampling has limited capability to retrieve coarser gains normally found in fill material. Bulk samples can be obtained during construction and any fill which complies to OPSS 1010 Granular A or B criteria and compacted to 98% SPMDD can be used as engineered fill below the pavement structure.

The pavement structure is proposed in this design considers the relatively high traffic volume of the gas station and carwash and frequent traffic of tanker trailers and included in Table 10-1.

Table 10-1: Heavy and Light Duty Pavement Structures

Material		Thickness (mm)	
		Heavy duty	Light duty
Surface	Superpave 12.5 mm, PG 58-34	40	50
Binder	Superpave 19 mm, PG 58-34	50	--
Base	OPSS Granular A	150	150
Sub-base	OPSS Granular B Type II	450	450

The base and sub-base materials, i.e., Granular A for base and Granular B Type II for sub-base, shall be in accordance with OPSS.MUNI 1010. Both base and sub-base should be compacted to 100% SPMDD. Asphalt layers should be compacted to comply with OPSS 310.

The light duty pavement is expected to render a lower quality performance and it is only recommended for the carwash access and the drive thru. The rest of the station shall receive the design and denoted heavy-duty. Identification of light-duty traffic needs is beyond the scope of this geotechnical report, and it is the site designers' responsibility.

It is noteworthy that the provided pavement structure is designed based on the expected light and heavy-duty traffic on the prepared subgrade. The proposed thickness of the pavement section shall be preserved across the site.

On occasion when the pavement is passing over the proposed utilities and tanks, each of those services shall be designed to tolerate the weight of large vehicles such as fuel tanks, fire trucks, etc. The designer is responsible to design the tanks accordingly. Asphalt pavement is flexible and deforms under wheel loads.

10.2 Sidewalks and Hard Surfacing

The width and extent of the sidewalks will be defined as per the architectural drawings. The designer shall provision adequate slope, based on applicable codes, to provide appropriate runoff discharge. Sidewalks can be categorized under residential/commercial use.

The City of Ottawa SC4 'Typical Concrete Sidewalk in Boulevard' standard is recommended for the construction of the concrete sidewalk. Expansion, construction, and dummy joints shall be spaced per the City of Ottawa standards SC5 'Sidewalk Construction Joints' and SC14 'Sidewalk Joints'. A minimum of 150 mm bedding of OPSS Granular 'A' compacted to 100% SPMDD is required for the concrete sidewalk panels.

All proposed new curbs shall be constructed as per applicable standards. It is recommended to follow City of Ottawa detail provided in SC1.4 'Concrete Barrier Curb with Sidewalk'. Curbs should receive a minimum of 150 mm Granular 'A' bedding compacted to 100% SPMDD on a proof rolled subgrade that is free from soft, loose, and organic materials.

If interlocking concrete pavers are selected for the design, the concrete pavers should meet the requirements of ASTM C936. The concrete pavers used for walkways (no vehicle traffic) should be placed on a minimum of 25 mm sand bedding which should meet the gradation requirements for concrete sand as described in CSA A23.2-04, Section 4.2.3.3. Below the sand, a minimum of 300 mm OPSS Granular 'A' compacted to 100% SPMDD should be provided.

11.0 SITE SERVICES

At the subject site, the burial depth of water-bearing utility lines is typically 2.4 m below the ground surface. The contractor should retain a professional engineer to provide detailed drawings for excavation and temporary support of the excavation walls during construction.

Clay plugs are recommended in service trenches. They should be placed close to the property boundary and seal the trenches to the highwater table elevation (El. 86.5 m). Clay seals shall terminate below the pavement structure. Clay shall be compacted to a minimum of 96% SPMDD.

All utility pipes shall receive a minimum of 150 mm bedding constructed of Granular A compacted to 100% SPMDD. Pipe covers shall be similarly Granular A, however a lower degree of compaction is acceptable if higher compaction efforts may damage the utility pipes.

Any individual compaction shots may not be less than 96% SPMDD. The mechanical engineer in charge of the gas pipe design shall provide detail pipe installation design and review construction procedure.

If the recommended pipe cover compaction cannot be approved by the gas company, authors of this report shall be informed and consulted to provide recommendation to lower risk of ground subsidence due to potential lack of compaction.

Similarly, other utility designers shall confirm the utility pipes can tolerate compaction loads.

Since the native soil is sensitive and contains fine grained soil, all granular bedding and backfill material shall be separated from the native soil by a layer of filter geotextile such as Class II Non-woven geotextile.

Excavation requirements are discussed in former sections.

12.0 CLOSURE

We trust this geotechnical investigation report meets the requirements of your project. The "Limitations of Report" presented in Appendix A are an integral part of this report. Please contact the undersigned should you have any questions or concerns.

McIntosh Perry Consulting Engineers Ltd.

c. V.

N'eem Tavakkoli, M.Eng., P.Eng.
Senior Geotechnical Engineer

Vasantha Wijeyakulasuriya, M.Eng., P.Eng.
Senior Technical Director, Geotechnical

LICENSED PROFESSIONAL ENGINEER
N. AZIZI TAVAKKOLI
100160911
Mar.28/23
PROVINCE OF ONTARIO

REFERENCES

1) Canadian Geotechnical Society, "Canadian Foundation Engineering Manual", 4th Edition, 2006.

2) Ontario Ministry of Natural Resources (OMNR), Ontario Geological Survey, Special Volume 2, "The Physiography of Southern Ontario", 3rd Edition, 1984.

3) Google Earth, Google, 2015.

4) Government of Canada, National Building Code of Canada (NBCC), "Seismic Hazard Calculation" (online), 2010.

5) Canadian Standards Association (CSA), "Concrete Materials and Methods of Concrete Construction", A23.1, 2009

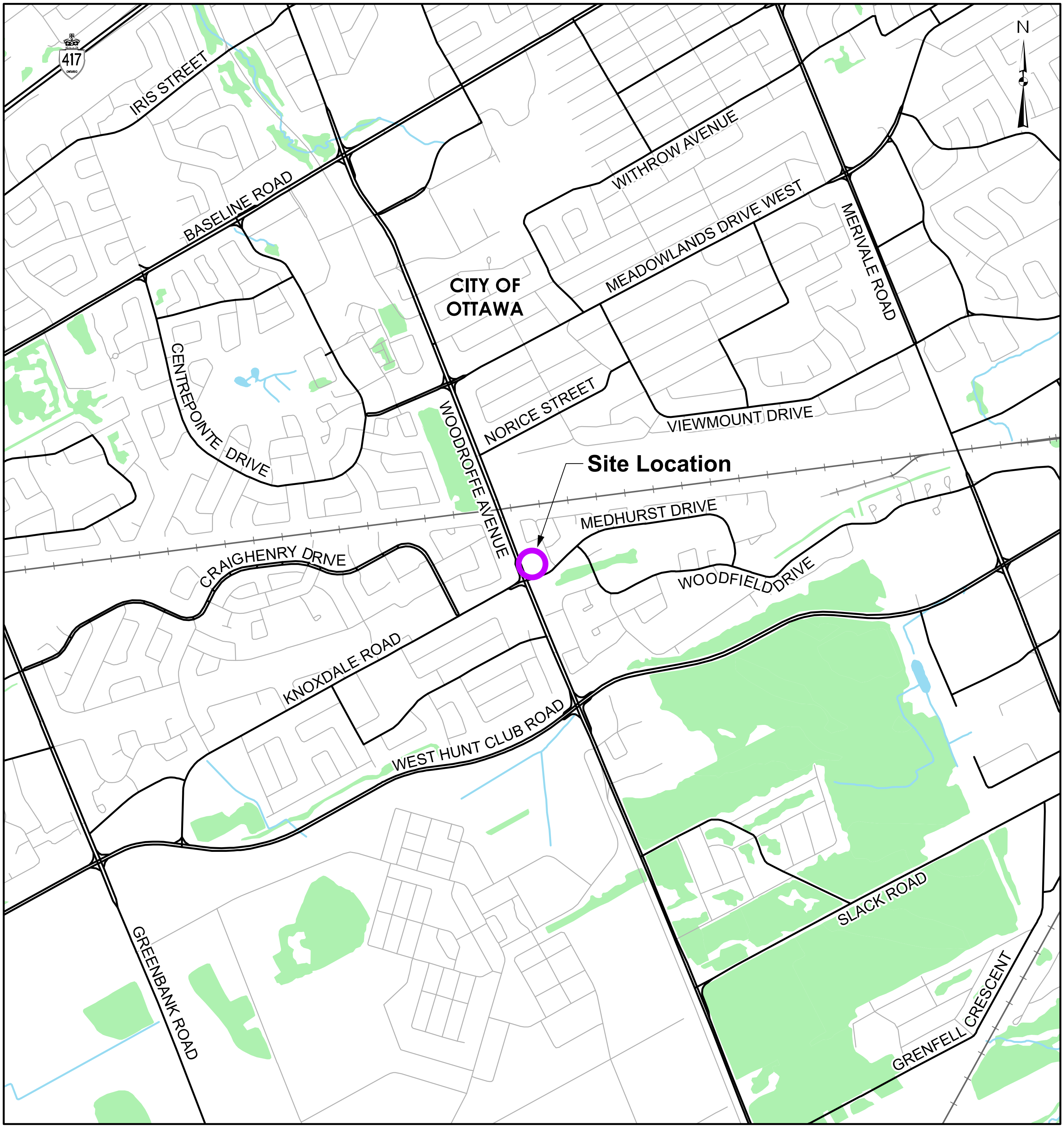
6) Government of Ontario, "Ontario Building Code (OBC)," (online), 2012.

7) MTO – Pavement Design and Rehabilitation Manual, 2013

8) Natural Resources Canada – Seismic Hazard Calculator

CIRCLE K - WOODROFFE – CCO21-2432-06

APPENDIX A
PLANS



600 300 0 600

Scale 1:25,000 Metres

LEGEND

○	Site Location	─┼─	Railroad
─	Local Road	~	Watercourse
━	Major Road	●	Waterbody
		🟩	Wooded Area

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2021.

CLIENT:
CIRCLE K STORES/MAC'S CONVENIENCE LTD.

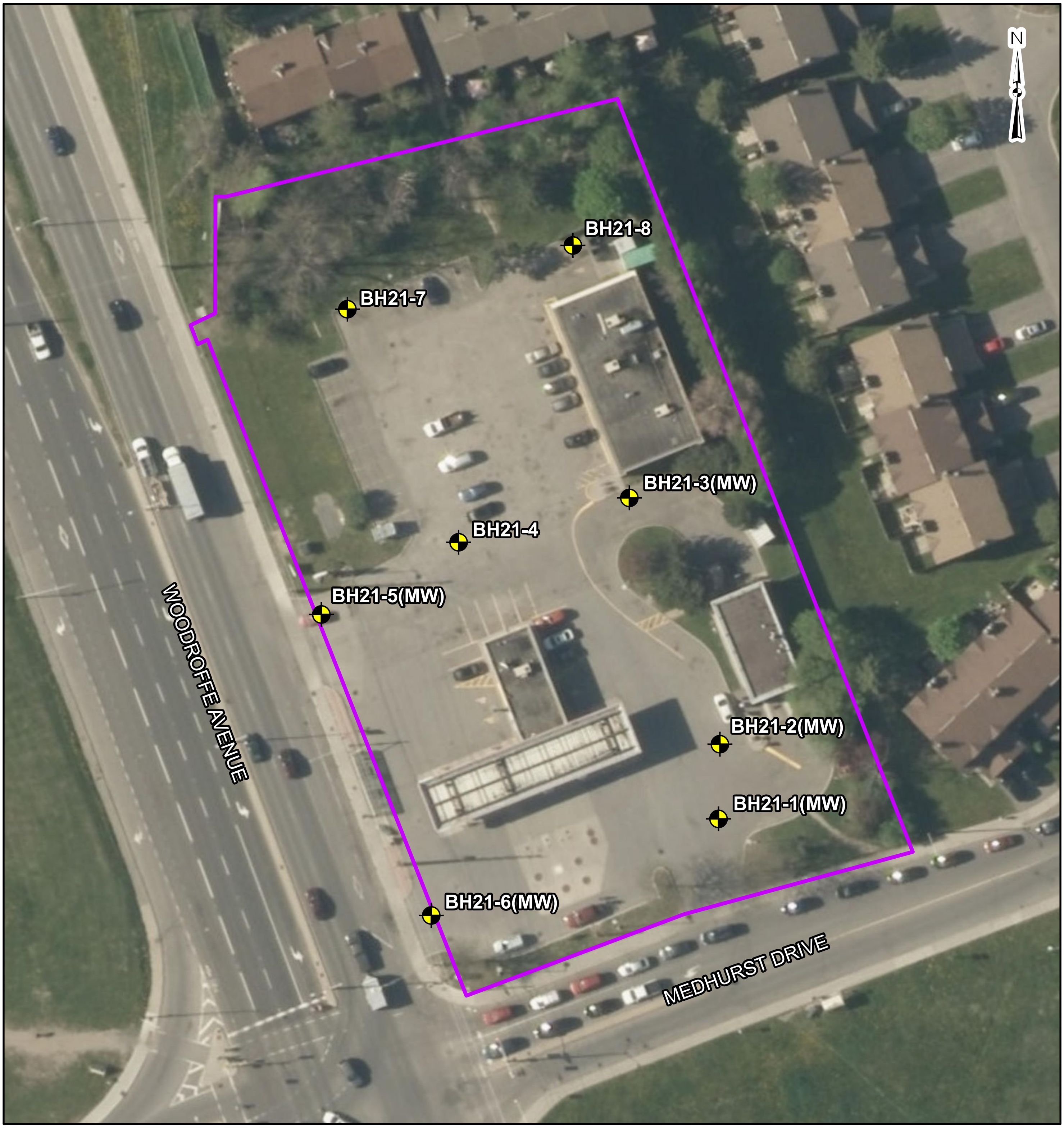
PROJECT: GEOTECHNICAL INVESTIGATION
1545 WOODROFFE AVENUE, OTTAWA, ON

TITLE:
SITE LOCATION

MCINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

PROJECT NO:	CCO-21-2432-06	FIGURE:
Date	Sep., 21, 2021	1
GIS	SK	
Checked By	MH	

C:\Users\s.klassen\McIntosh Perry\GIS - Documents\Projects\2021\CCO\CCO-21-2432-06 Circle K - 1545 Woodroffe Ave\aprx\Geotechnical\CCO-21-2432-06_Geotechnical.aprx



LEGEND

▭ Approximate Site Boundary

⊕ Borehole/Monitoring Well Location

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2021.

CLIENT:
CIRCLE K STORES/MAC'S CONVENIENCE LTD.

PROJECT: GEOTECHNICAL INVESTIGATION
1545 WOODROFFE AVENUE, OTTAWA, ON

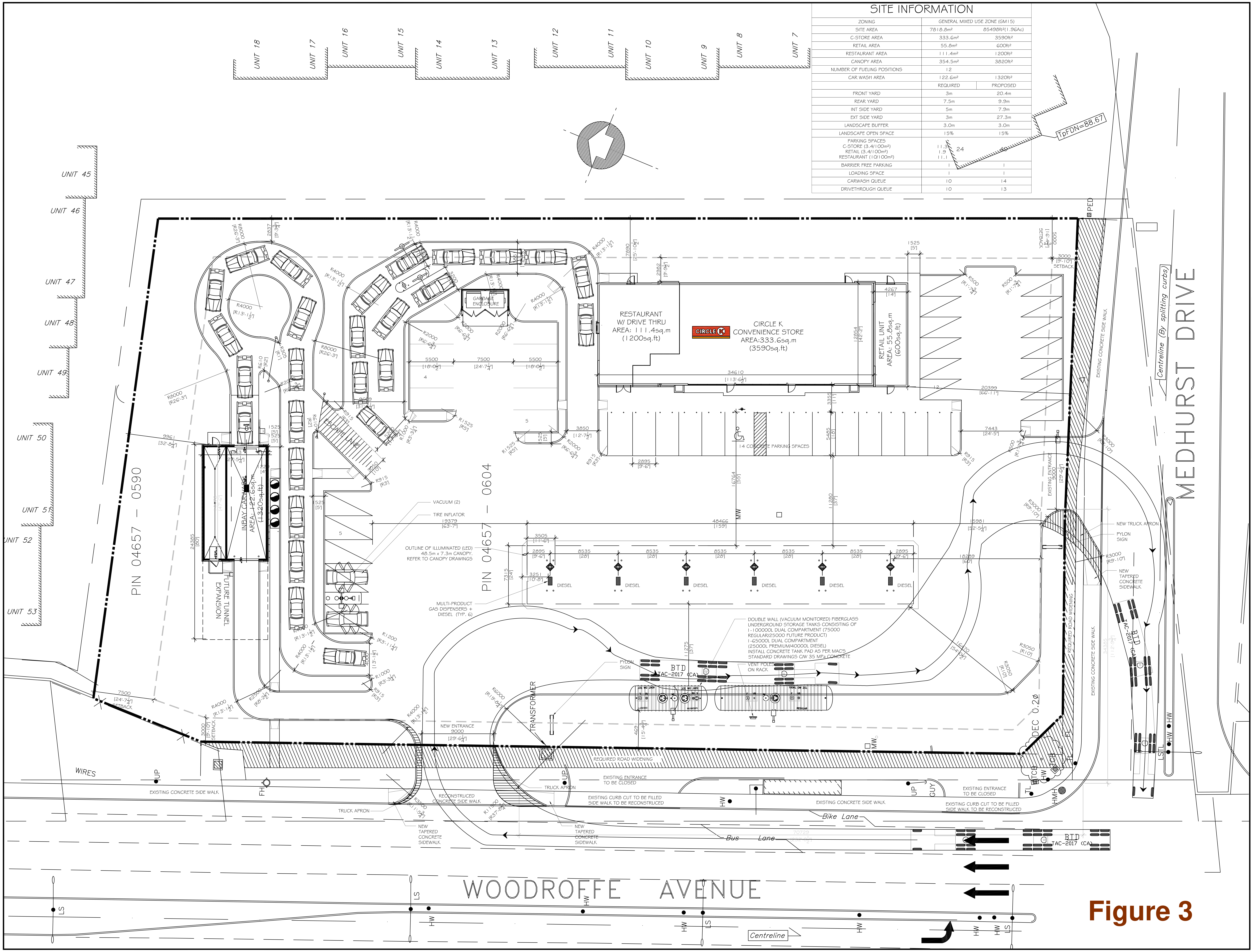
TITLE:
BOREHOLE LOCATIONS

McINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

PROJECT NO:CCO-21-2432-06	
Date	Sep., 21, 2021
GIS	MH
Checked By	SK

FIGURE:
2

C:\Users\s.klassen\McIntosh Perry\GIS - Documents\Projects\2021\CCO\CCO-21-2432-06 Circle K - 1545 Woodroffe Ave\aprx\Geotechnical\CCO-21-2432-06_Geotechnical.aprx



SITE INFORMATION

ZONING	GENERAL MIXED USE ZONE (GM15)	
SITE AREA	7818.8m²	85498ft²(1.96Ac)
C-STORE AREA	333.6m²	3590ft²
RETAIL AREA	55.8m²	600ft²
RESTAURANT AREA	111.4m²	1200ft²
CANOPY AREA	354.5m²	3820ft²
NUMBER OF FUELING POSITIONS	12	
CAR WASH AREA	122.6m²	1320ft²
	REQUIRED	PROPOSED
FRONT YARD	3m	20.4m
REAR YARD	7.5m	9.9m
INT SIDE YARD	5m	7.9m
EXT SIDE YARD	3m	27.3m
LANDSCAPE BUFFER	3.0m	3.0m
LANDSCAPE OPEN SPACE	15%	15%
PARKING SPACES C-STORE (3.4/100m²) RETAIL (3.4/100m²) RESTAURANT (10/100m²)	11.3 1.9 11.1 24	40
BARRIER FREE PARKING	1	1
LOADING SPACE	1	1
CARWASH QUEUE	10	14
DRIVETHROUGH QUEUE	10	13

Figure 3

NOTES:

ARCHITECT/ENGINEER:

STAMPS:

No.	DATE	REVISION\ISSUED	BY

CIRCLE K - CENTRAL CANADA DIVISION

305 MILNER AVE. SUITE 400
TORONTO, ONTARIO
M1B 3V4

TEL: 416.291.4444
FAX: 416.291.4947

DO NOT SCALE DRAWINGS

THE GENERAL CONTRACTOR MUST CHECK AND VERIFY MEASUREMENTS AND REPORT ANY ERRORS TO CONTRUCTION MANAGER BEFORE PROCEEDING WITH WORK.

DATE:
DEC 05, 2016

AREA:
3781 SQFT

PROJECT:
OTTAWA

ADDRESS:
1545 WOODROFFE AVENUE

TITLE:
PROPOSED SITE PLAN

SCALE:
1:200

DRAWN BY:
MS

DWG NO.:
P-1

Friday, February 12, 2021 16:56:02

APPENDIX B
BOREHOLE LOGS

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ'	$-^o$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	$-^o$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
Υ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
Υ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
Υ	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
Υ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L – W_L)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W – W_P)/ I_P$	i	1	HYDAULIC GRADIENT
Υ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L – W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERED SOIL	$e_{,max}$	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
Υ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

LOG OF BOREHOLE BH 21-1 MW

PROJECT: Circle K - 1545 Woodroffe Ave
CLIENT: Circle K
PROJECT LOCATION: 1545 Woodroffe Ave, Ottawa, ON
DATUM: MTM Zone 9
BH LOCATION: N 5021867 E 363474

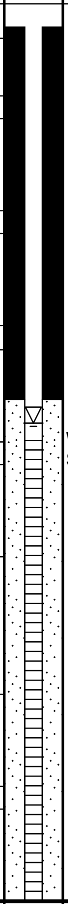
DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200 mm
Date: Aug/17/2021

REF. NO.: CCO-21-2432
ENCL NO.: 1

SOIL PROFILE (m) ELEV / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLS / 0.3 m / RQD	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
88.7	Asphalt								
0.0	**Asphalt**, 90 mm								
88.5 0.2	**Sandy gravel,**trace silt, loose, dark brown to brown, damp, (FILL)	1	GS			44	41	(15)	
					88				
		2	SS	5					
87.1 1.7	**Sandy Silt**, trace gravel, black, soft (Organic)	3	SS	4	87				
86.7 2.0	**Silty Sand**, compact, grey, moist	4	SS	15	86				
85.5 3.2	**Silty Clay**, soft, grey, wet	5	SS	2	85				
84.2 4.6	**Sand**, trace to some silt, compact to loose, grey, wet	6	SS	12	84				
		7	SS	5	83				
82.8 5.9	**End of Borehole** **Monitoring Well Installed**								

STRATA PLOT

GROUND WATER CONDITIONS



W. L. 86.0 m
Sep 03, 2021

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED + FIELD VANE & Sensitivity
● QUICK TRIAXIAL × LAB VANE
50 100 150 200 250

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%)
10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (Mg/m^3)

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

LOG OF BOREHOLE BH 21-2 MW

PROJECT: Circle K - 1545 Woodroffe Ave	**DRILLING DATA**
CLIENT: Circle K	Method: Hollow Stem Augers
PROJECT LOCATION: 1545 Woodroffe Ave, Ottawa, ON	Diameter: 200 mm REF. NO.: CCO-21-2432
DATUM: MTM Zone 9	Date: Aug/17/2021 ENCL NO.: 2
BH LOCATION: N 5021878 E 363474	

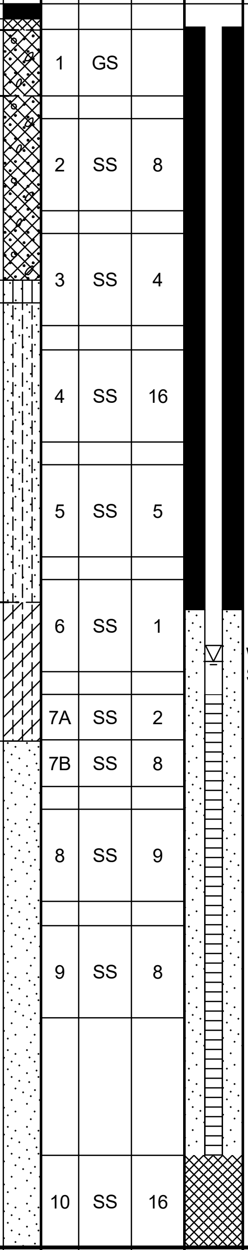
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED + FIELD VANE & Sensitivity
● QUICK TRIAXIAL × LAB VANE
50 100 150 200 250

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

POCKET PEN. (Cu) (kPa) — NATURAL UNIT WT (Mg/m^3)



SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLS / 0.3 m RQD	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
89.0	Asphalt				
88.9 0.1	**Asphalt**, 100 mm				
88.8 0.2	**Sandy gravel**, grey, dry, (Fill) **Gravelly sand**, trace silt (Fill)	1	GS		
88.4 0.6	**Gravelly sand**, trace silt, trace clay, trace organics, loose, dark brown, moist (Fill)	2	SS	8	
87.1 87.0 2.0	**Sandy Silt**, trace clay, loose, grey, moist **Silty Sand**, compact to loose, grey, moist	3	SS	4	
		4	SS	16	0 61 31 8
		5	SS	5	
85.0 4.0	**Silty Clay**, very soft, grey, wet	6	SS	1	
		7A	SS	2	
84.1 4.9	**Sand**, trace silt, loose, brown to grey, wet	7B	SS	8	
		8	SS	9	
		9	SS	8	4 95 (1)
		10	SS	16	
80.7 8.2	**End of Borehole** **Monitoring Well Installed**				

GRAPH NOTES: + ³, × ³: Numbers refer to Sensitivity ○ ³=3% Strain at Failure

LOG OF BOREHOLE BH 21-3 MW

PROJECT: Circle K - 1545 Woodroffe Ave
CLIENT: Circle K
PROJECT LOCATION: 1545 Woodroffe Ave, Ottawa, ON
DATUM: MTM Zone 9
BH LOCATION: N 5021914 E 363460

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200 mm
Date: Aug/17/2021

REF. NO.: CCO-21-2432
ENCL NO.: 3

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%)			
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLS / 0.3 m / RQD			GR	SA	SI	CL
88.9	Asphalt									
88.9 / 0.0	**Asphalt**, 100 mm									
88.8 / 0.1	**Sandy gravel**, dry (Fill)									
88.8 / 0.2	**Gravelly Sand**, trace silt, brown, damp, (Fill)	1	GS							
88.0 / 0.9	**Silty Sand**, some gravel, compact, brown, damp, (Fill)	2	SS	12		88				
87.1 / 1.8	**Silty Sand**, some clay, trace gravel, loose, brown to dark brown, damp, (Fill), tree roots, wood chips, debris, asphalt	3	SS	9		87				
		4	SS	6						
86.2 / 2.7	**Silty Sand**, loose, grey, moist					86				
		5	SS	7			4	46	40	10
85.1 / 3.8	**Silty Clay**, very soft, grey, wet	6	SS	1		85				
		7	SS	WOH	W. L. 84.1 m Sep 03, 2021	84	0	5	67	27
						83				
		8A	SS	WOH						
82.5 / 6.4	**Sand**, trace silt, loose to very loose, grey, moist	8B	SS	7		82				
		9	SS	3						
81.3 / 7.6	**End of Borehole** **Monitoring Well Installed**									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED
+ FIELD VANE & Sensitivity
● QUICK TRIAXIAL
× LAB VANE
50 100 150 200 250

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (Mg/m³)

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity   ○ ^{8}=3% Strain at Failure

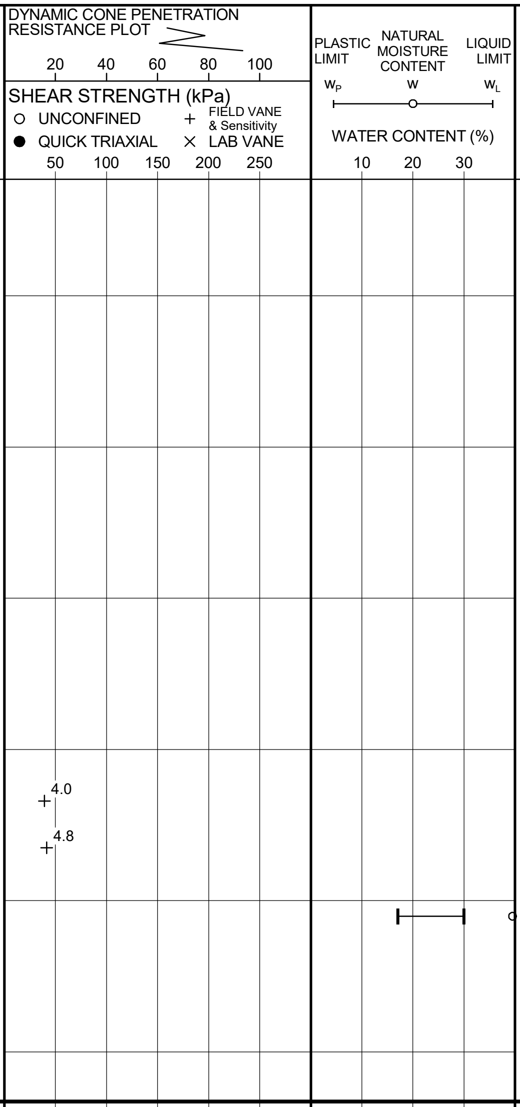
LOG OF BOREHOLE BH 21-4

PROJECT: Circle K - 1545 Woodroffe Ave
CLIENT: Circle K
PROJECT LOCATION: 1545 Woodroffe Ave, Ottawa, ON
DATUM: MTM Zone 9
BH LOCATION: N 5021907 E 363435

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200 mm
Date: Aug/18/2021

REF. NO.: CCO-21-2432
ENCL NO.: 4

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLS / 0.3 m / RQD	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
88.8	Asphalt						
0.0	**Asphalt**, 100 mm						
88.7 0.1	**Sandy gravel**, dry (Fill)						
88.5 0.3	**Gravelly Sand**, some silt, loose, brown, dry (Fill)	1	GRAB				
		2	SS	6		88	
		3	SS	6		87	
86.8 2.0	**Silty Sand**, trace gravel, trace organics, loose, grey, moist						
86.5 2.3	**Silty Sand**, compact, grey, moist	4	SS	14		86	
85.7 3.1	**Silty Clay**, trace sand, soft, grey, wet	5	SS	2		85	
			VANE				
			VANE				
		6	SS	WOH		84	0 2 66 31
83.3 5.5	**Sand**, trace silt, loose, grey, wet	7	SS	8		83	
82.7 6.1	**End of Borehole**						



POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (Mg/m^3)

GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
○ $^{\varepsilon}$=3% Strain at Failure

LOG OF BOREHOLE BH 21-5 MW

PROJECT: Circle K - 1545 Woodroffe Ave
CLIENT: Circle K
PROJECT LOCATION: 1545 Woodroffe Ave, Ottawa, ON
DATUM: MTM Zone 9
BH LOCATION: N 5021896 E 363415

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200 mm
Date: Aug/18/2021

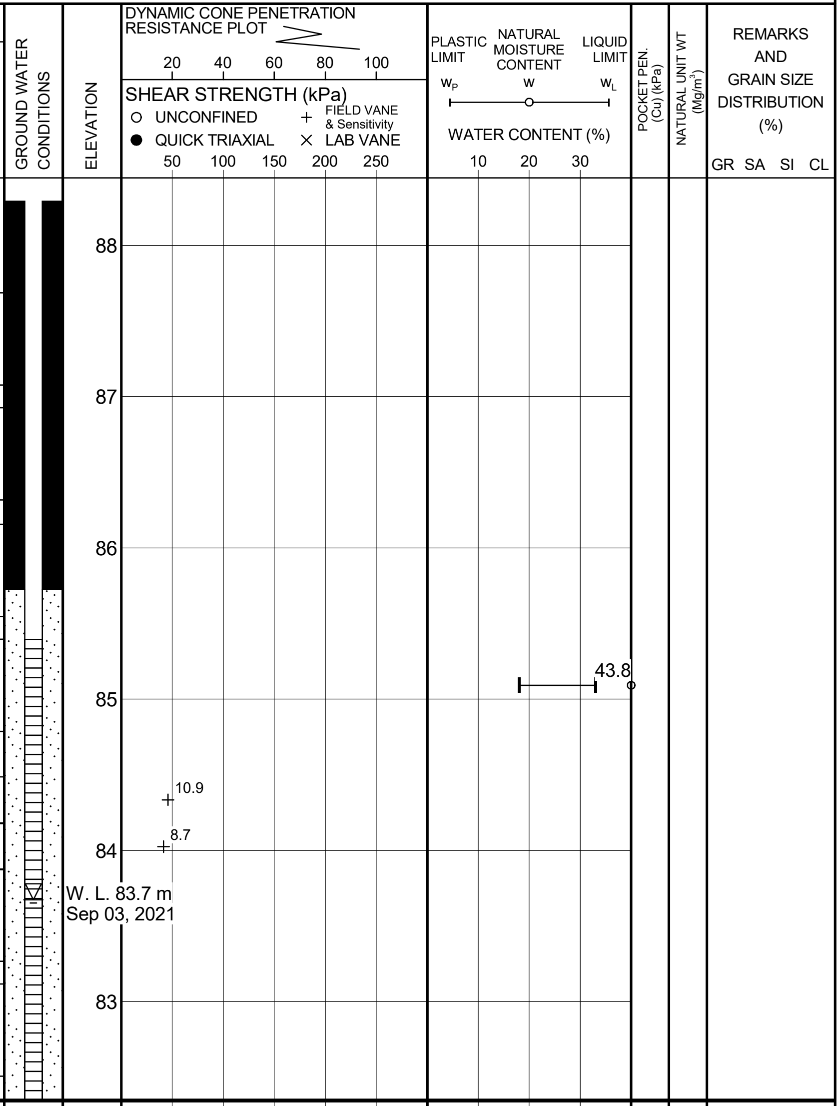
REF. NO.: CCO-21-2432
ENCL NO.: 5

SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION
88.4	Asphalt
88.3 0.0	**Asphalt**, 150 mm
0.2	**Gravelly Sand**, some silt, grey to brown, dry, (Fill)
87.7 0.8	**Sandy Silt**, trace clay, trace organic, loose, grey, damp
86.5 2.0	**Silty Sand**, loose, grey, moist
85.9 2.6	**Silty Clay**, soft to firm, grey, wet
83.7 4.7	**Sand**, trace silt, compact, grey, wet
82.3 6.1	**End of Borehole** **Monitoring Well Insatlled**

SAMPLES

NUMBER	TYPE	"N" BLS / 0.3 m / RQD
1	SS	5
2	SS	7
3	SS	7
4	SS	1
	VANE	
	VANE	
5	SS	12
6	SS	7



GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE BH 21-6 MW

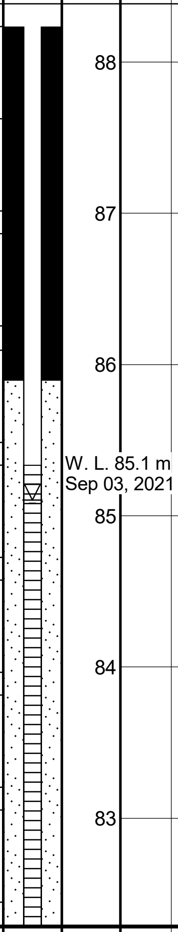
PROJECT: Circle K - 1545 Woodroffe Ave
CLIENT: Circle K
PROJECT LOCATION: 1545 Woodroffe Ave, Ottawa, ON
DATUM: MTM Zone 9
BH LOCATION: N 5021852 E 363432

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200 mm
Date: Aug/18/2021

REF. NO.: CCO-21-2432
ENCL NO.: 6

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLS / 0.3 m / RQD	ELEVATION
88.4	Asphalt				
88.3 0.1	**Asphalt**, 95 mm **Gravelly Sand**, loose to compact, brown, damp, (Fill)	1	SS	9	88
		2	SS	16	87
86.7 1.7	**Silty Sand**, trace to some clay, compact to loose, grey to brown, damp	3	SS	10	
		4	SS	7	86
85.6 2.7	**Clayey silt**, trace silt, very soft to soft, grey, wet	5	SS	1	85
84.4 4.0	**Sand**, trace silt, trace gravel, compact, grey, wet	6	SS	15	84
		7	SS	15	83
		8	SS	11	
82.3 6.1	**End of Borehole** **Monitoring Well Insatlled**				

GROUND WATER CONDITIONS



DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED + FIELD VANE & Sensitivity
● QUICK TRIAXIAL × LAB VANE
50 100 150 200 250

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (Mg/m^3)

REMARKS AND GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ 8=3% Strain at Failure

LOG OF BOREHOLE BH 21-7

PROJECT: Circle K - 1545 Woodroffe Ave
CLIENT: Circle K
PROJECT LOCATION: 1545 Woodroffe Ave, Ottawa, ON
DATUM: MTM Zone 9
BH LOCATION: N 5021941 E 363418

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200 mm
Date: Aug/18/2021

REF. NO.: CCO-21-2432
ENCL NO.: 7

(m) ELEV / DEPTH	DESCRIPTION
88.8	Asphalt
88.8 0.0	**Asphalt**, 38 mm **Sandy gravel**, trace silt, loose, brown, dry, (Fill)
88.4 0.5	**Silty Sand**, some gravel, loose to compact, brown, moist, (Fill)
86.9 2.0	**Sandy Silt**, trace gravel, loose, brown to dark brown, moist
86.6 2.3	**Silty Sand**, loose, brown to grey, moist to wet
85.8 3.1	**Clayey silt**, soft to firm, brown to grey, wet
84.3 4.6	**Silty Clay**, soft to firm, trace sand, dark grey, wet
82.4 6.4	**Sand**, tace silt, compact, grey, wet
82.1 6.7	**End of Borehole**

SAMPLES NUMBER	TYPE	"N" BLS / 0.3 m
1	SS	8
2	SS	10
3	SS	3
4	SS	5
5	SS	3
6	SS	1
7	SS	1
8	SS	WOH

STRATA PLOT | GROUND WATER CONDITIONS | ELEVATION: 88, 87, 86, 85, 84, 83

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED + FIELD VANE & Sensitivity
● QUICK TRIAXIAL × LAB VANE
50 100 150 200 250

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa) | NATURAL UNIT WT (Mg/m^3) | REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH 21-8

PROJECT: Circle K - 1545 Woodroffe Ave
CLIENT: Circle K
PROJECT LOCATION: 1545 Woodroffe Ave, Ottawa, ON
DATUM: MTM Zone 9
BH LOCATION: N 5021951 E 363451

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200 mm
Date: Aug/18/2021

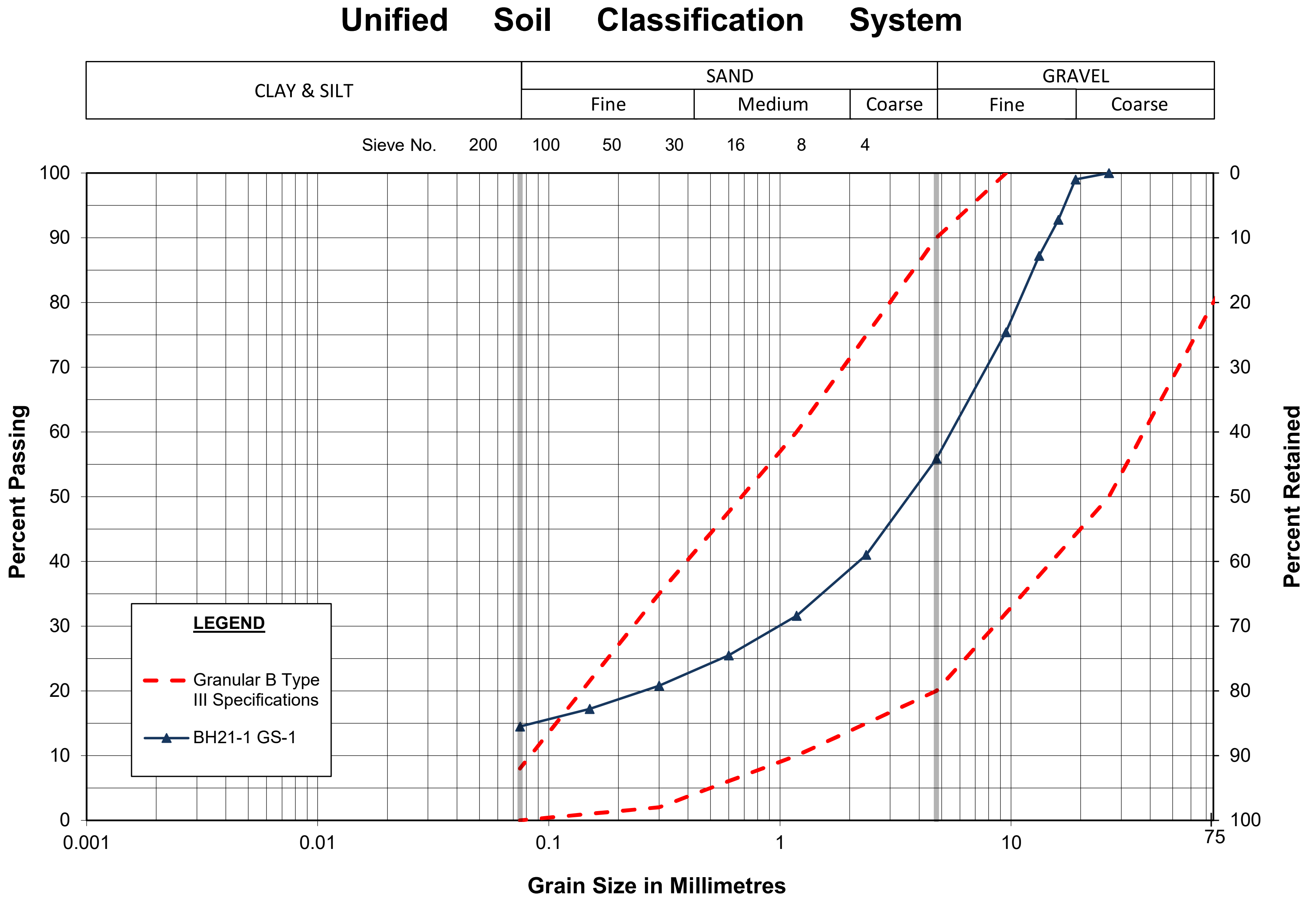
REF. NO.: CCO-21-2432
ENCL NO.: 8

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE: 50 100 150 200 250
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (Mg/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)			
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLS / 0.3 m / RQD					GR	SA	SI	CL
89.0	Asphalt												
88.9 0.1	**Asphalt**, 50 mm **Gravelly Sand**, trace silt, loose, brown, moist, (Fill)												
			1	SS	9		88						
87.5 1.5	**Sandy Silt**, organic, loose, damp												
87.2 1.8	**Silty Sand**, loose to compact, brown, moist to wet		2	SS	5		87						
			3	SS	11		86						
	---------------------------- some clay		4	SS	3					0	25	57	18
85.2 3.8	**Silty Clay**, trace sand, soft to firm, grey, wet		5	SS	2		85						
			6	SS	1		84						
							83						
	---------------------------- some sand		7	SS	1								
82.3 6.7	**End of Borehole**												

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity ○ 3=3% Strain at Failure

APPENDIX C
LABORATORY TEST RESULTS



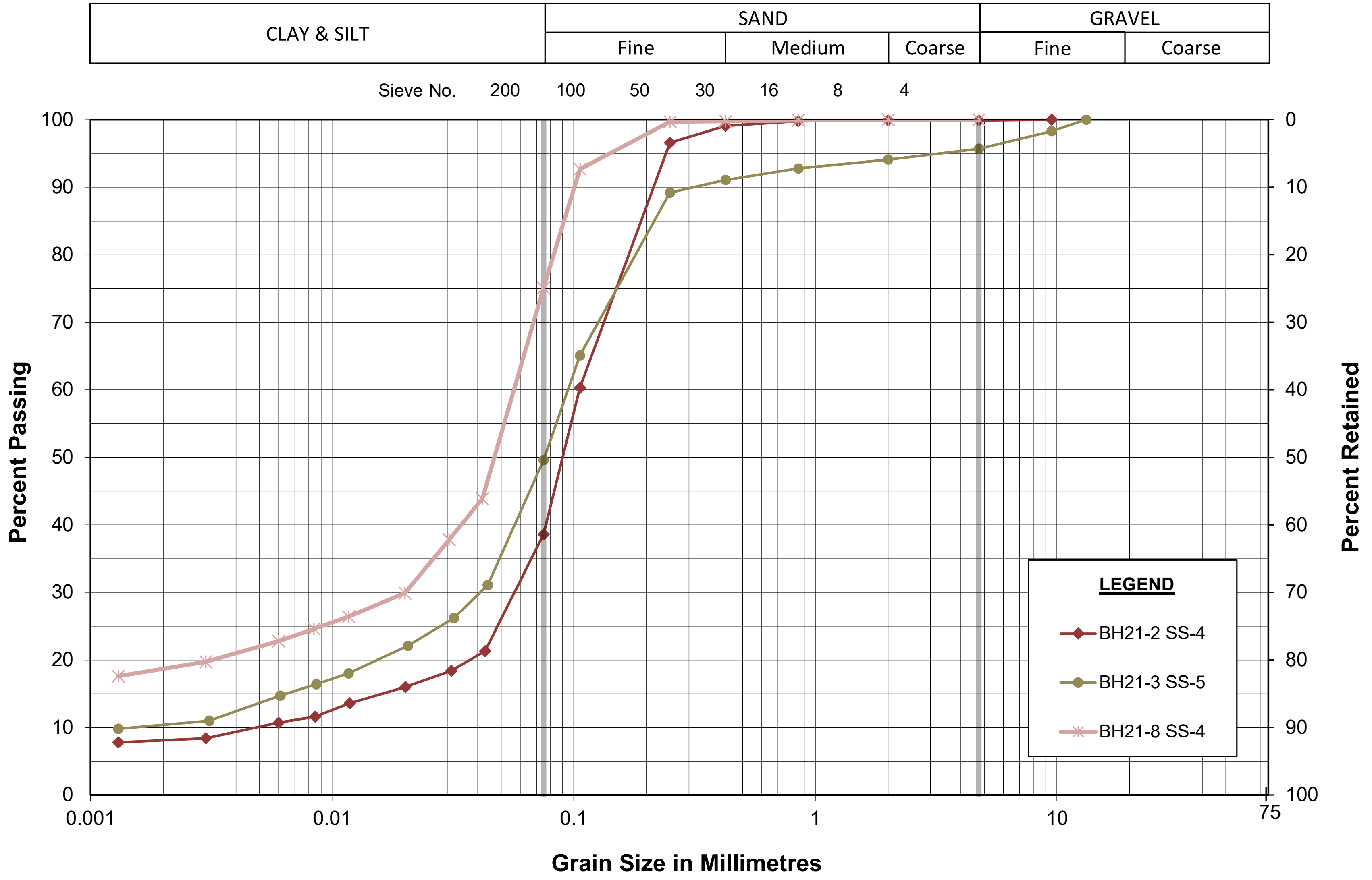
McINTOSH PERRY

Grain Size Distribution
FILL
Circle K - Woodroffe

Figure No. 4

CCO-21-2432-06

Unified Soil Classification System



McINTOSH PERRY

Grain Size Distribution

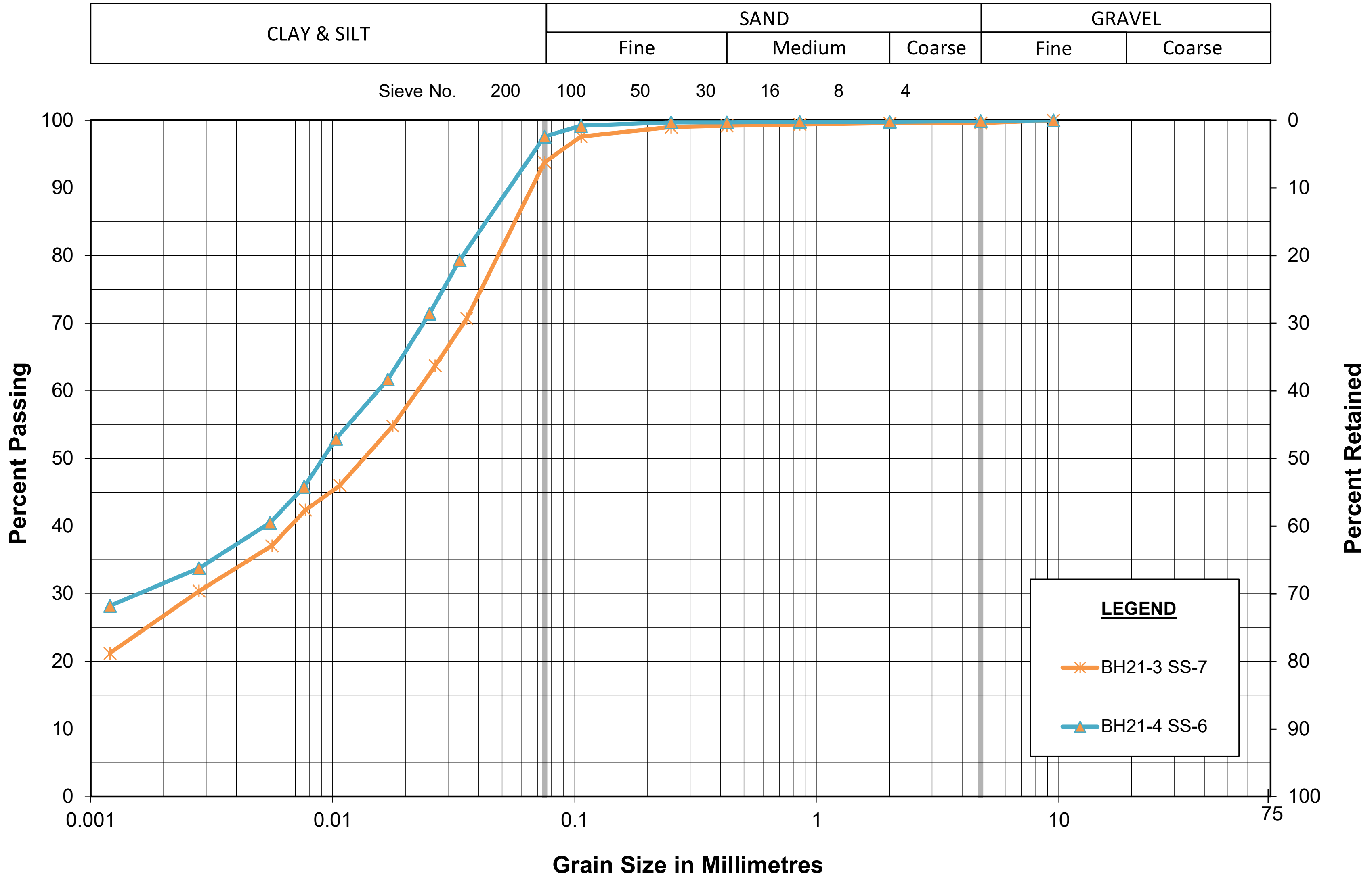
Silty Sand to Sandy Silt

Circle K - Woodroffe

Figure No. 5

CCO-21-2432-06

Unified Soil Classification System

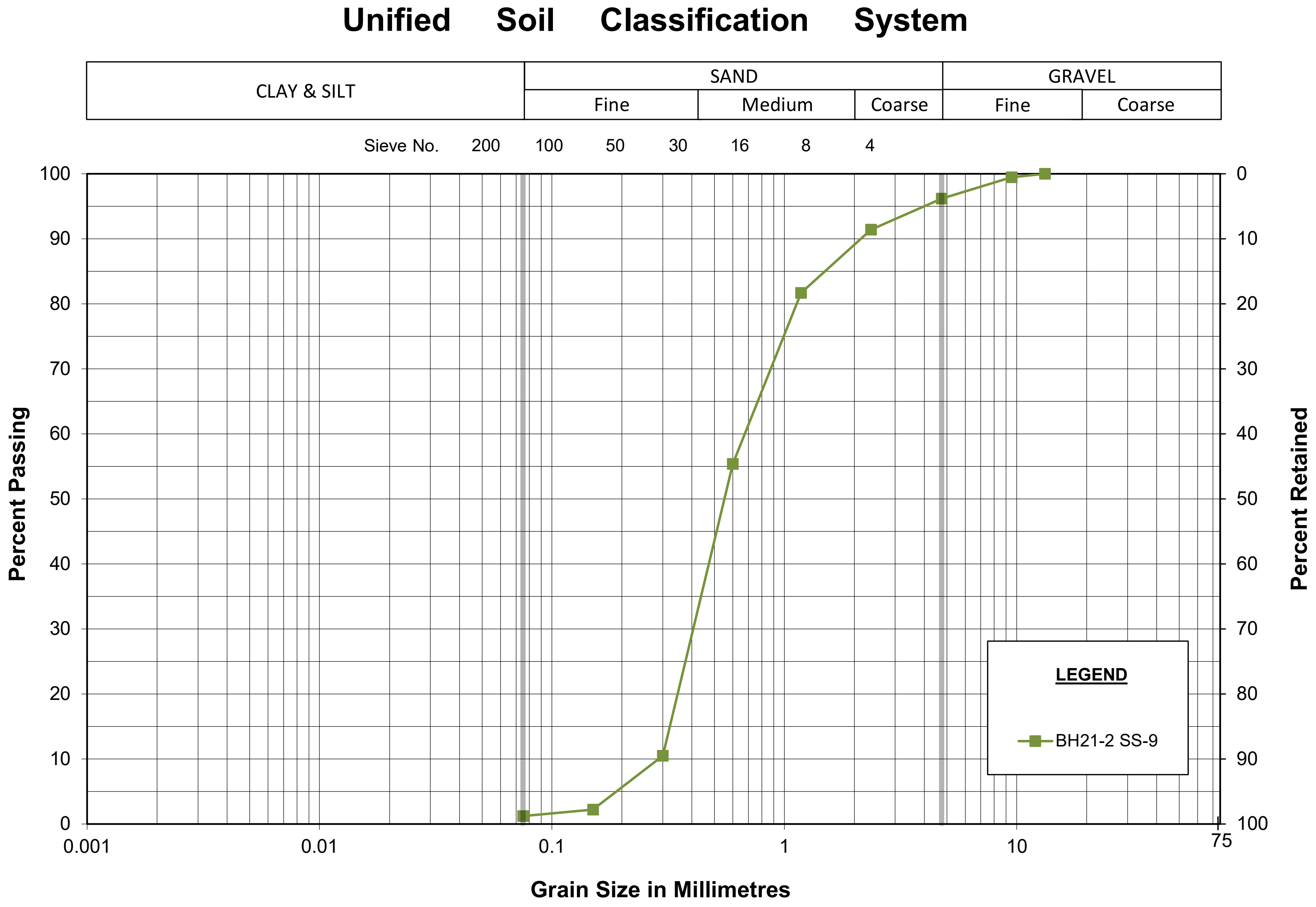


McINTOSH PERRY

Grain Size Distribution
Clayey Silt
Circle K - Woodroffe

Figure No. 6

CCO-21-2432-06



McINTOSH PERRY

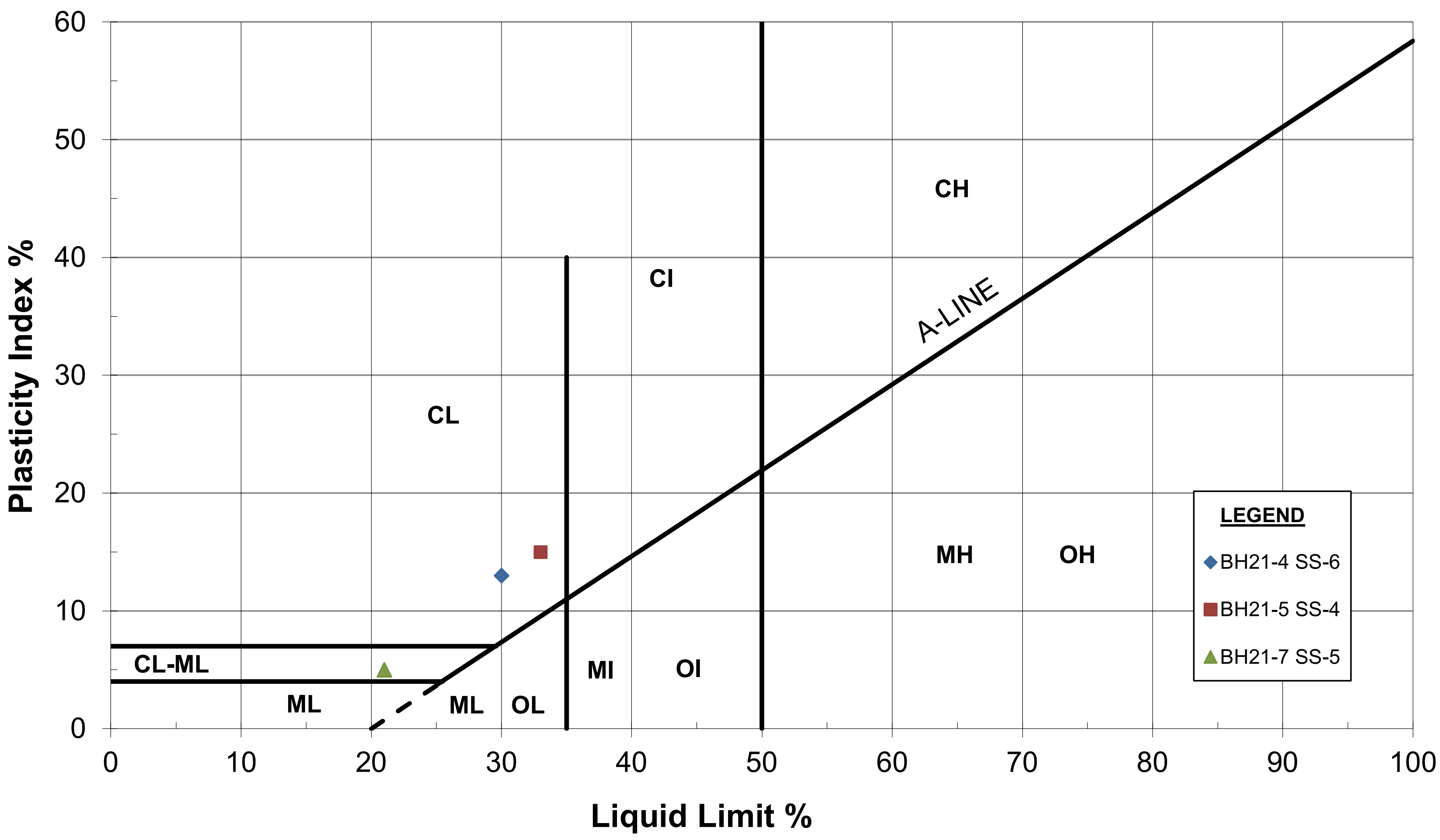
Grain Size Distribution

Sand

Circle K - Woodroffe

Figure No. 7

CCO-21-2432-06



McINTOSH PERRY

PLASTICITY CHART

Clayey Silt to Silty Clay

Figure No. 8

CCO-21-2432-06

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

Certificate of Analysis

McIntosh Perry Consulting Eng. (Nepean)

215 Menten Place, Unit 104
Nepean, ON K2H 9C1
Attn: Jason Hopwood-Jones

Client PO: CCO 21-2432-06-09
Project: 1545 Woodroffe, Circle K
Custody: 128756

Report Date: 14-Sep-2021
Order Date: 9-Sep-2021

Order #: 2137396

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2137396-01	BH21-1 SS-4
2137396-02	BH21-5 SS-4

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	13-Sep-21	13-Sep-21
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	10-Sep-21	10-Sep-21
Resistivity	EPA 120.1 - probe, water extraction	10-Sep-21	11-Sep-21
Solids, %	Gravimetric, calculation	13-Sep-21	13-Sep-21

Certificate of Analysis

Client: **McIntosh Perry Consulting Eng. (Nepean)**

Client PO: CCO 21-2432-06-09

Report Date: 14-Sep-2021

Order Date: 9-Sep-2021

Project Description: 1545 Woodroffe, Circle K

	Client ID:	BH21-1 SS-4	BH21-5 SS-4	-	-
	Sample Date:	17-Aug-21 09:00	17-Aug-21 09:00	-	-
	Sample ID:	2137396-01	2137396-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.5	71.6	-	-
General Inorganics					
pH	0.05 pH Units	7.76	7.68	-	-
Resistivity	0.10 Ohm.m	24.7	7.81	-	-
Anions					
Chloride	5 ug/g dry	156	492	-	-
Sulphate	5 ug/g dry	59	206	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g dry	ND			NC	20	
Sulphate	631	5	ug/g dry	559			12.1	20	
General Inorganics									
pH	7.68	0.05	pH Units	7.65			0.4	2.3	
Resistivity	92.0	0.10	Ohm.m	91.1			1.0	20	
Physical Characteristics									
% Solids	81.6	0.1	% by Wt.	81.7			0.1	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	90.8	5	ug/g	ND	90.8	82-118			
Sulphate	641	5	ug/g	559	82.0	80-120			

Qualifier Notes:

Login Qualifiers :

Container and COC sample IDs don't match - Sample bag reads: "BH21-2 SS-4"

Applies to samples: BH21-1 SS-4

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

APPENDIX D
SEISMIC HAZARD CALCULATIONS

2015 National Building Code Seismic Hazard Calculation

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

Site: 45.335N 75.752W

2021-09-24 19:28 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.438	0.239	0.143	0.042
Sa (0.1)	0.513	0.291	0.180	0.058
Sa (0.2)	0.430	0.248	0.156	0.053
Sa (0.3)	0.327	0.190	0.121	0.042
Sa (0.5)	0.232	0.135	0.086	0.030
Sa (1.0)	0.116	0.068	0.044	0.015
Sa (2.0)	0.055	0.032	0.020	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.275	0.158	0.098	0.031
PGV (m/s)	0.193	0.108	0.066	0.021

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

References

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

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

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

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

Natural Resources Canada Ressources naturelles Canada

Canada

CIRCLE K - WOODROFFE – CCO21-2432-06

APPENDIX E
LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

McIntosh Perry Consulting Engineers Ltd. (McIntosh Perry) carried out the field work and prepared the report. This document is an integral part of the Foundation Investigation and Design report presented.

The conclusions and recommendations provided in this report are based on the information obtained at the borehole locations where the tests were conducted. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations where tests were conducted and conditions may become apparent during construction, which were not detected and could not be anticipated at the time of the site investigation. The benchmark level used and borehole elevations presented in this report are primarily to establish relative differenced in elevations between the borehole locations and should not be used for other purposes such as to establish elevations for grading, depth of excavations or for planning construction.

The recommendations presented in this report for design are applicable only to the intended structure and the project described in the scope of the work, and if constructed in accordance with the details outlined in the report. Unless otherwise noted, the information contained in this report does not reflect on any environmental aspects of either the site or the subsurface conditions.

The comments or recommendation provided in this report on potential construction problems and possible construction methods are intended only to guide the designer. The number of boreholes advanced at this site may not be sufficient or adequate to reveal all the subsurface information or factors that may affect the method and cost of construction. The contractors who are undertaking the construction shall make their own interpretation of the factual data presented in this report and make their conclusions, as to how the subsurface conditions of the site may affect their construction work.

The boundaries between soil strata presented in the report are based on information obtained at the borehole locations. The boundaries of the soil strata between borehole locations are assumed from geological evidences. If differing site conditions are encountered, or if the Client becomes aware of any additional information that differs from or is relevant to the McIntosh Perry findings, the Client agrees to immediately advise McIntosh Perry so that the conclusions presented in this report may be re-evaluated.

Under no circumstances shall the liability of McIntosh Perry for any claim in contract or in tort, related to the services provided and/or the content and recommendations in this report, exceed the extent that such liability is covered by such professional liability insurance from time to time in effect including the deductible therein, and which is available to indemnify McIntosh Perry. Such errors and omissions policies are available for inspection by the Client at all times upon request, and if the Client desires to obtain further insurance to protect it against any risks beyond the coverage provided by such policies, McIntosh Perry will co-operate with the Client to obtain such insurance.

McIntosh Perry prepared this report for the exclusive use of the Client. Any use which a third party makes of this report, or any reliance on or decision to be made based on it, are the responsibility of such third parties. McIntosh Perry accepts no responsibility and will not be liable for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.