GEOTECHNICAL INVESTIGATION AND DESIGN RECOMMENDATION REPORT
1083 AND 1095 MERIVALE ROAD, OTTAWA, ONTARIO

1083 and 1095 Merivale Road, Ottawa, Ontario

Project No.: CCO-22-3530

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1.0 INTRODUCTION

McIntosh Perry Consulting Engineers Ltd. (McIntosh Perry) was retained by CSV Architects (Client) on behalf of the Shepherds of Good Hope (SOGH, Owner) to undertake a geotechnical investigation and provide design recommendations for the proposed construction of a four (4) storey building with a basement (Project). The site is located at 1083 and 1095 Merivale Road (Site), also known as Ottawa Road #63 in Ottawa, Ontario.

The geotechnical investigation and design recommendations services for the proposed building are provided at the request of CSV Architects. A proposal was submitted to the Client on December 11, 2021 and was accepted by the Client through email dated November 15, 2022.

Additional work was requested by the Client that included evaluating the global stability of the proposed retaining wall at the north of 1095 Merivale Road. A proposal for a scope change was submitted to the Client on November 16, 2022. The scope change was accepted by the Client and the Owner by means of an authorization email dated January 06, 2023.

This report presents the factual findings obtained from the geotechnical engineering investigation at the Site. The fieldwork was carried out between January 10 and 11, 2023 and comprised of five (5) boreholes in total.

The purpose of the investigation was to explore the subsurface conditions at this site and to provide borehole location plans, record of borehole logs, and laboratory test results. This report provides anticipated geotechnical conditions influencing the design and construction of the proposed building.

This report is prepared for the sole use of the Client. The use of this report, or any reliance on it by any third party, is the responsibility of such third party. This report is subject to the limitations shown in Appendix A. It is understood that the Project will be designed and constructed in accordance with all applicable codes and standards present within its jurisdiction.

2.0 PROJECT UNDERSTANDING

McIntosh Perry's understanding of the Project is based on communication with the Client and based on the preliminary architectural drawings dated July 22, 2021 and November 10, 2022, and survey drawings dated August 09, 2021 received from the Client.

It is understood that the Project consists of the construction of a four (4) storey building with one level basement located at 1083 Merivale Road and partially on 1095 Merivale Road, and other site improvements across the combined site for 1083 and 1095 Merivale Road, Ottawa, Ontario. No information was provided regarding the proposed final floor elevation. It was assumed that the proposed building will be supported on shallow foundation founded on sound bedrock on approximate elevation (El.) of 84.0 ± 0.5 meter above sea level (masl).

It is understood that a retaining wall is proposed on the north boundary of the 1095 Merivale Road property. At the time of preparing this report, no information was provided to the authors of this report regarding the type, size, or geometry of the proposed retaining wall.

The discussions and recommendations provided in this report address only those aspects related to the proposed building and improvements on 1083 and 1095 Merivale Road properties and the proposed retaining wall on the north boundary of the 1095 Merivale Road property. Discussions or recommendations regarding the existing development and paved areas on 1095 Merivale Road property is out of the scope of this report.

3.0 SITE DESCRIPTION

3.1 Existing Site Conditions

The existing site is occupied by a single-storey dwelling at 1083 Merivale Road and a three-storey apartment building at 1095 Merivale Road. It is understood that the single-storey building will be demolished to accommodate the new proposed apartment building. The existing building at 1095 Merivale Road is to remain. A concrete block retaining wall exists at the north boundary of the 1095 Merivale Road . The existing retaining wall observed to be cracked and damaged.

The Site is bounded by Merivale Road from the west side. The Site is surrounded by a mix of multi-story residential buildings and single-family dwellings to the north, east and south of the site. Topography observed to be relatively flat within the Site proximity.

3.2 Site Geology

Based on published geological 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 Site is located within till comprising of stone-poor, sandy silt to silty sand textured till on Paleozoic terrain and bounded by Paleozoic bedrock formation from the east. The bedrock within the area is identified to comprise of Limestone of the Shadow Lake Formation.

4.0 FIELD INVESTIGATION AND TESTING

McIntosh Perry cleared the Site before the commencement of any geotechnical drilling. Utility clearance requisitions were submitted to Ontario One Call (ON1Call) to obtain public utility locates. Private locates were submitted to GFL Environmental. Public utility owners were informed, and all utility clearance documents were obtained before the commencement of drilling work.

The field work was completed between January 10 and 11th 2023. The boreholes were drilled using a CME-850 truck-mounted drilling rigs, outfitted with hollow stem augers. The equipment used for drilling was owned and operated by Downing Drilling of Hawkesbury, Ontario. 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. The bedrock was cored and sampled to approximately 3.0 m depth from the top of the encountered bedrock surface in boreholes 22-2, 22-3, 22-5 and 22-6. NQ size rock cores were obtained using diamond drilling and wireline tooling. Rock cores were retrieved in double-walled NQ coring methods. Three 51 mm diameter standpipe monitoring wells were installed in BH22-2 and 22-6 with screen installed in the bedrock and overburden and in BH22-5 with screen installed in the bedrock. The wells were protected in flush-mount caps. Details and location information of the wells are provided in Section 6.4 and summarized in Tables 6-3.

The bedrock core holes were sealed with bentonite holeplug and the boreholes were backfilled with auger cuttings and holeplug and restored to the original ground surface. The boreholes were surveyed with a GPS unit to record their locations and elevations. Borehole locations are shown in Figure 2, included in Appendix B.

Table 4-1: Borehole Designations, Locations, and Depth

BH No.	Drilling Date	Coordinates			Borehole Termination	
		Latitude	Longitude	Surface El. (masl)	Depth (mbgs)	El. (masl)
22-1	January 10, 2023	45.376983	-75.732891	87.2	1.9	85.3
22-2 MW	January 10, 2023	45.377063	-75.732931	86.9	5.6	81.3
22-3	January 10, 2023	45.377118	-75.732765	86.9	5.6	81.3
22-4	BH22-4 was a shallow pavement borehole and was not drilled due to space restrictions.					
22-5 MW	January 10, 2023	45.377281	-75.732742	86.8	7.1	79.7
22-6 MW	January 11, 2023	45.377445	-75.732323	85.7	5.7	80.0

Field investigation, including drilling and sampling, were supervised on a full-time basis by McIntosh Perry. All boreholes were logged during the drilling process. All samples were labelled by waterproof paper one by one as they were retrieved. All soil samples were preserved in double plastic bags to mitigate the risk of moisture loss during transportation to the geotechnical laboratory. Rock cores were laid and labelled in specialty boxes made for rock core transferring. The Rock Quality Designation was measured for the first time in the field immediately after drilling to reduce the measurement errors caused by transportation induced damages to the rock cores.

5.0 LABORATORY TESTING

All soil and rock samples obtained during the investigation were transported to McIntosh Perry's geotechnical laboratory in Nepean, Ontario. McIntosh Perry's Geotechnical laboratory is certified by the Ministry of Transportation Ontario (MTO) under the RAQS program at Medium Complexity level for Soil and Rock Testing, including Testing for Foundation Engineering.

Geotechnical laboratory testing was performed on representative soil samples to determine soil index properties including grain-size analysis tests. The laboratory tests were performed in accordance with the Ministry of Transportation Ontario (MTO) test procedures, which follow the American Society for Testing Materials (ASTM) test procedures.

Paracel Laboratories Ltd. in Ottawa carried out chemical tests on four (4) representative soil samples and consisted of pH, chloride, sulphate, resistivity, Sulphide and RedOx. Laboratory test results are included in Appendix D.

The rest of the soil samples recovered will be stored in McIntosh Perry storage facility for a period of three (3) months after submission of the final report. Samples will be disposed after this period unless otherwise requested in writing by the Client.

6.0 SUBSURFACE CONDITIONS

This section shall be read in conjunction with the Site Description section as well as the borehole logs included in Appendix C and laboratory test results included in Appendix D. The general description of stratigraphy is provided in the following sections. However, it should be noted that in case of any discrepancy between the description and the borehole logs, borehole records govern the description of subsurface conditions.

In general, the site stratigraphy for BH22-1, 22-3 consists of fill underlain by bedrock. The site stratigraphy for BH22-2, 22-5 and 22-6 consists of fill underlain by native till soil with bedrock encountered below the native soil. The bedrock was cored and sampled in all boreholes, except in BH22-1 which was a pavement borehole. For classification purposes, the fill, soil and bedrock encountered at this site can be divided into three distinguishable zones;

a) Fill

b) Till

c) Bedrock

The fill and soils encountered during the course of the investigation, together with the field and laboratory test results are shown on the borehole records included in Appendix C. Laboratory test results are included in Appendix D. Description of the strata encountered are given below.

6.1 Fill

A fill layer was encountered in all boreholes at the surface and extend to depths ranging between 1.5 mbgs (El. 85.3 masl) in BH22-5 to 2.5 mbgs (El. 84.4 masl) in BH22-3. The fill was observed to compose mainly sand and gravel, with some silt and trace of clay, and was observed to be brown to dark brown with moisture content of dry to moist.

Three (3) samples from the fill underwent grain size analysis testing and the layer was observed to contain 17 to 52% gravel, 34 to 50% sand, and 14 to 35% fines. The test results are summarized in Table 6-1. Test results of grain size analysis are shown in Figure 4, included in Appendix D.

Table 6-1: Grain Size Analysis Test Results Summary – Fill

Sample ID	Depth (mbgs)	Elve. (masl)	Size Fraction (%)			Description / Remarks
			Gravel	Sand	Fines	
BH22-3 / SS-2	0.8 to 1.4	86.1 to 85.5	52	34	14	Sandy gravel with some silt
BH22-5 / SS-2	0.8 to 1.4	86.0 to 85.4	21	50	29	Gravely silty sand
BH22-6 / SS-2	0.8 to 1.4	84.9 to 84.3	17	47	35	Silt and Sand with some gravel

The recorded SPT 'N' value within the fill in BH22-1 ranged between 15 and 52 blows/300 mm, indicating that the sand is compact to dense. Occasional refusal was encountered within the fill probably on gravel or cobble pieces.

6.2 Till

Below the fill in BH22-2, 22-5, and 22-6, a thin layer of till was encountered at depths ranging between 1.5 mbgs (El. 85.3 masl) in BH22-5 and 2.0 mbgs (El. 83.7 masl) in BH22-6. The till was observed to be mainly composed of sandy silt with trace gravel. Occasional fragments of limestone rock observed in the retrieved soil samples. The till was observed brown with the moisture content of moist to wet. The till was observed to be compact to dense with recorded SPT N-value ranged between 14 and 52 blows/300 mm. Refusal was encountered within the till in BH22-6 probably on a cobble or a rock fragment.

6.3 Bedrock/Refusal

Refusal was encountered in BH22-1, which is a pavement borehole, on inferred bedrock/boulder at depth approximately 1.9 (El. 85.3 masl). The bedrock was encountered and cored in the foundation boreholes below the fill and till between 2.3 mbgs (El. 84.6 masl) in BH22-2 and 3.2 mbgs (El. 83.7 masl) in BH22-5. The Bedrock was also encountered at depth of approximately 2.7 mbgs (El. 83.0 masl) at the proposed retaining wall location. The bedrock was cored and sampled down to approximately 3.0 m in the bedrock using diamond core NQ size.

During the core drilling, measurements including Total Core Recovery (TCR) and Rock Quality Designation (RQD) were carried out as part of the rock quality classification. TCR is defined as the sum of all recovered rock core pieces from a core run expressed as a percent of the total length of the core run. The RQD is defined as a percentage of the sum of the intact core pieces over 100 mm divided by the total length of core run. The TCR and RQD for the rock cores are presented in the borehole log records in Appendix C.

Based on the retrieved rock cores from boreholes within the proposed building footprint, the bedrock was identified as limestone with shale parting and was observed to be moderately weathered to weathered with closely spaced, horizontal joint discontinuities. The limestone was observed to be strong, grey, thinly bedded, and has poor to fair quality based on RQD values (41% to 67%). Mud seams interbeds in the rock joints.

At the proposed retaining wall location, the bedrock was identified as limestone with shale parting and was observed to be very weathered, very intensely fractured with very closely spaced, horizontal joint discontinuities. The limestone was observed to be moderately strong, grey to dark grey, thinly bedded, and has very poor quality based on RQD values (6% to 16%).

Rock cores photos are presented in Appendix C. A summary of bedrock observations is provided in Table 6-2.

Table 6-2: Summary of Bedrock Observations

Borehole	Ground Surface El. (masl)	Bedrock Surface El. (masl)	Weathered Bedrock El. (masl)	Sound Bedrock El. (masl)	Remarks
BH22-2	86.9	84.6	--	84.6 to 81.3	- Rock core ~ 3.3 m
BH22-3	86.9	84.4	84.4 to 83.6	83.6 to 81.3	- Rock core ~ 3.1 m
BH22-5	86.8	83.7	--	83.7 to 79.7	- Rock core ~ 4.0 m
BH22-6	85.7	83.0	83.0 to 80.0	--	- Rock core ~ 3.0 m

6.4 Groundwater Conditions

Groundwater was not observed during the site of investigation in open borehole. Three monitoring wells were installed in BH22-2, 22-5 and 22-6. These boreholes were denoted with “MW”. The groundwater level was measured in the monitoring wells on February 13, 2023. The observed groundwater levels in the monitoring wells with monitoring well information are presented in Table 6-3.

Groundwater levels are expected to fluctuate due to extreme weather events and seasonal changes.

Table 6-3: Monitoring Wells Summary

BH/MW ID	Screen Interval (masl)	Water Level Observation				Remarks
		Installation Date	Measurement Date	Depth (mbgs)	Elev. (masl)	
BH22-2 MW	82.3 – 85.1	Nov. 23, 2022	Feb. 13, 2023	5.0	81.9	Screen in the overburden and bedrock
BH22-5 MW	83.0 – 79.7	Jan. 10, 2022	Feb. 13, 2023	5.8	81.0	Screen in the bedrock
BH22-6 MW	84.8 – 81.4	Jan. 11, 2022	Feb. 13, 2023	2.3	83.4	Screen in the overburden and bedrock

6.5 Chemical Test Results

Chemical analyses were conducted by Paracel Laboratories in Ottawa, ON, to determine the resistivity, pH, sulphate and chloride content of four (4) representative soil samples collected from the boreholes. The soil samples were chosen from within the estimated foundation depth. The laboratory results for the chemical analysis are shown in Table 6-4 and included in Appendix D.

Table 6-4: Soil Chemical Analysis Results

Borehole	Sample	Depth (mbgs)	pH	Sulphate (%)	Chloride (%)	Resistivity (Ohm-cm)	RedOx (mV)	Sulphide (%)
BH22-2	SS-2	0.8 – 1.4	7.59	0.0085	0.0029	3030	424	0.04
BH22-2	SS-3	1.5 – 2.1	7.54	0.0087	0.0017	3800	434	<0.04
BH22-5	SS-3	1.5 – 2.1	7.66	0.0041	0.0112	2230	410	<0.04
BH22-5	SS-4	2.3 – 2.9	7.64	0.0022	0.0013	4690	438	<0.04

7.0 DISCUSSIONS AND RECOMMENDATIONS

7.1 General

Based on the results of the geotechnical field and laboratory investigation performed, the following discussion is provided to assist the Client and the Designer with the proposed new multi-storey building at 1083 and 1095 Merivale Road Project. The recommendations provided within this report are based on our understanding of the proposed Project which is summarized above in "Section 2" and through the interpretation of factual information obtained from the boreholes advanced during this subsurface investigation. If any of these understandings change, McIntosh Perry should be contacted to assess the implications of those changes on the recommendations provided herein.

Based on the soil conditions observed in the boreholes, and assuming they are representative of soil condition across the Site, the most important geotechnical considerations for the design of the culvert and installation of the proposed culvert replacement are expected to be the following:

- **Foundation on Bedrock and Subgrade Preparation:** the building will be supported on shallow foundation system founded on sound bedrock at approximate elevations of 84.0 ± 0.5 masl. The bedrock subgrade should be cleaned of any loose or unstable rock pieces from the footing influence zone. Lean mixed concrete should be used for levelling the sound bedrock. If lean mixed concrete is used below any footings it must extend a minimum of 0.3 m beyond the edge of the footing and then downward at a 1H:1V. The lean mix concrete shall have a minimum compressive strength of 25 MPa. The underlying subgrade has to be approved by the Geotechnical Engineer.

- **Seismic Site Classification:** The proposed building will be designed in accordance with Part Four of OBC-2012. Part Four of the Code requires that all buildings to be designed to resist earthquake forces. Based upon the results of the site investigation, the proposed building can be designed to "Site Class C" in accordance with Table 4.1.8.4.A of the OBC-2012, and subject to the limitations of the code.

- **Temporary Construction Dewatering:** Excavation for the proposed multi-storey building will proceed through the fill and till into the bedrock. Groundwater was observed in all monitoring wells. In in BH22-2 and 22-5 which were drilled within the proposed building footprint, the groundwater was observed within the bedrock. The groundwater in BH22-6 at the proposed retaining wall was observed within the overburden. The observed groundwater is below the proposed foundation level. However, groundwater and surface runoff water may infiltrate and accumulate at the bottom of the excavations due to seasonal changes and extreme weather events. It is expected that dewatering will be required during the construction stage for this Site to keep the excavation reasonably dry. Dewatering may be achievable with traditional sump and pump dewatering method. Application for Permit to Take Water is not anticipated for this Site based on the observed level of water in the monitoring well. However, if groundwater encountered during construction, the contractor should decide on an adequate permit as per applicable regulations. Due to predicted seasonal groundwater fluctuations application for Environmental Activity and Sector Registry (EASR) is recommended.

- **Permanent Drainage and Waterproofing:** The proposed building includes a basement level below the grade with shallow foundation founded on bedrock. The excavations for the basement and the foundation will extend below the existing bedrock surface resulting in water pooling at the proposed floor slab level. Therefore, permanent under-floor drainage and waterproofing are required. Exterior perimeter drains are not recommended for this Site. Full water proofing membranes such as a WR Meadows Mel-ROL PRECON, or an equivalent type product for walls and under-slab will be required. Water stops should be installed at cold joints in the foundation walls and floor-wall joints. We also recommend that considerations should be given to the design of building basement as a fully waterproof 'bath-tub' design (without external perimeter drains) to avoid potential adverse impacts due to moisture movements in the immediate areas surrounding the proposed building footprint.

It is important to emphasize that at the time of writing this report, limited information was communicated to the authors of this report with respect to the proposed building design loads, and the proposed retaining wall type, size, or geometry.

McIntosh Perry geotechnical team shall be retained to review the proposed foundation and retaining wall designs once they become available and provide comments to ensure conformance with the general recommendations provided within this report.

The comments made regarding the construction of the proposed building are intended to highlight those aspects which could impact or affect the detail design of the proposed structure, for which special provisions may be required in the Contract Documents. Comments related to construction aspects are not intended to dictate construction equipment or methods. Relevant parties should make their own interpretation of the factual data presented in the report. Interpretation of the data presented may affect equipment selection, proposed construction methods, and scheduling of construction activities.

7.2 Site Preparation and Grading

The construction of the proposed building will include demolishing the existing building on the premises of the 1083 Merivale Road property and removing all concrete foundation and buried elements. It will include also removing the existing retaining wall at the north boundary of the 1095 Merivale Road property. Excavation to remove the existing footings will proceed through asphalt, fill, native soil.

The Site should be graded in the early stages of construction to provide for positive control of surface water and directing it away from excavations and subgrades. The Contractor should take appropriate measurements for collection and disposal of surface and groundwater and runoff including an adequate pumping system.

7.2.1 Buried Services

Public and private utility owners should be notified prior to the commencement of any construction activities. Existing underground utilities in the vicinity of the proposed excavation should be reviewed before commencing any excavation works to identify potential damage hazards due to the proposed excavation. Existing utilities that are excavated or exposed as part of the construction will need to be supported and rerouted during the construction. Even with a shoring system, some inward movement of shoring is inevitable. This may cause slight ground settlement which may have an adverse effect on the existing buried utilities. The contractor shall inform owners of all existing utilities before proceeding with excavation. The utility owners may provide the permissible deformation that a particular utility may tolerate. Shoring shop drawings should be stamped by a professional engineer.

7.2.2 Excavation Impact on Adjacent Structures

The designer and contractor should account for the influence of the excavation on nearby commercial, residential properties and roadways. If the foundation influence zone intersects with the excavation, a case-specific review shall be provided by a structural engineer. The influence zone is defined by a 1H:1V outward and downward from the bottom of the footings. If any adjacent load bearing element is subject to undermining, then an Engineered Shoring system and/or underpinning program will need to be considered.

7.2.3 Existing Topsoil and Fill Soils

All fill soils shall be removed from within the footprint of the proposed buildings, to expose a native undisturbed subgrade. Any over excavation shall be leveled by lean concrete or a concrete mix of the same strength as the foundation system.

The excavated materials and any corresponding excess soils should be disposed of in accordance with all applicable environmental legislation. Excess soils management and evaluation of the environmental quality of subsoils is not within the scope of this geotechnical investigation.

7.3 Excavation

7.3.1 Overburden Excavation

It is understood that the excavations for the building will extend to a depth range approximately between 2.3 to 3.2 mbgs (El. 84.6 to 83.6 m). At the time of writing this report, information regarding the proposed final floor elevation was not provided and it was assumed that the proposed building will be founded on shallow foundation founded on sound bedrock at approximate El. 84.0 ± 0.5 masl.

Also, no information was available regarding the existing retaining wall foundation depth or the proposed foundation depth for the new retaining wall. However, it is expected that the new retaining wall will be supported on a strip footing founded on bedrock at depth ranges between 2.7 to 3.0 mbgs (El. 83.0 ± 0.3 masl).

The excavations for the proposed building and retaining wall will extend through the fill and native till to the bedrock. All excavations must be undertaken in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), Regulations for Construction O.Reg. 213/91, with specific reference to acceptable size slopes and stabilization requirements. The general stratigraphy outlined herein can be considered an OHSA Type 3 Soil above groundwater and Type 4 Soil below groundwater. For excavations through multiple soil types, the side slope geometry is governed by the soil with the highest number designation. The excavation for proposed building and retaining wall should be conducted through a minimum 1H:1V or a flatter slope above groundwater and 3H:1V below groundwater. Where excavation advanced through multiple soil types, the highest soil type shall be considered as per OHSA. If the minimum slope requirement cannot be achieved, Engineered Shoring should be used.

Excavation through the soil below the groundwater level are anticipated to be more problematic. The coarse-grained fill and till are anticipated to seep into the excavation. In addition, space restrictions are expected for open excavation due to proximity to Merivale Road and existing multi-story buildings to the north and east. Therefore, it is recommended that the excavations be undertaken within the confines of an Engineered Shoring designed and installed in accordance with OHSA. The shoring will need to support the excavation sidewalls and act as a barrier against groundwater flow into the excavation. However, the removal of water within the shored excavation will still be required. Recommendations for appropriate dewatering measures beyond conventional sump and pump techniques such as a positive dewatering system to effectively lower the static groundwater level shall be provided by a specialized dewatering contractor.

The stability of the excavation side slopes is highly dependent on the Contractor's methodology and layout. The excavations of the fill and overburden soils are expected to be performed using conventional hydraulic excavation equipment. Cobbles, and boulders may be encountered during the excavations. Boulders larger than 0.3 meters in diameter should be removed from the excavation side slopes for workers' safety. No surface surcharges should be placed closer to the edge of the excavation than a distance equal to twice the depth of the excavation, unless an excavation support system has been designed to accommodate such a surcharge.

7.3.2 Bedrock Excavation

For excavations into bedrock, the upper weathered rock zone will require back sloping depending on the degree of weathering. The bedrock quality and Site-specific requirements need to be assessed during construction by the geotechnical engineer.

For planning purposes, a weathered bedrock is recommended to be treated as a Type 2 Soil. Sound rock would generally be self-supporting, however, as a precautionary measure, it should be back-sloped at 10V:1H. All rock excavations should be scaled, to remove loose rock fragments to ensure safe working conditions. All rock faces should be reviewed by a geotechnical engineer to look for loose pieces and wedge failures. Rock bolting for worker safety may be necessary depending on the layout and field condition at that time.

Bedrock excavation will require pneumatic or hydraulic breakers such as hoe-rams or heavy rock excavation equipment capable of breaking and ripping sound limestone bedrock. Alternatively, controlled blasting techniques may need to be used, subject to the bylaws and blasting restrictions that are in effect for the area. Designers are referred to the OPSS.MUNI 120 specifications for the use of explosives. In general, these documents require a blasting plan to be prepared by a blasting engineer. They also require conducting pre-blast surveys on nearby buildings, utilities, structures, water wells, and facilities likely to be affected by the blast. Vibration monitoring during the blasting in nearby structures or infrastructure is required. The structural engineer shall indicate the maximum allowable PPV tolerance for the adjacent buildings, and this information shall be included in the contract drawings.

7.3.3 Engineered Shoring

Engineered Shoring system is required during both excavation and construction stages to protect the adjacent roadway, properties, utilities, and to protect the public safety. Engineered Shoring systems such as soldier piles and lagging, interlocking sheet piles, secant and/or tangent walls, and permanent diaphragm walls are often used to support excavations through soil. The design of the Engineered Shoring system is the responsibility of the contractor. The contractor should hire an experienced professional geostructural engineer to provide a detailed design for the Engineered Shoring system considering the space restrictions, estimated costs, and availability of materials. The Engineered Shoring designer must take into consideration the loads from any adjacent structures or infrastructure being retained, lateral earth pressures, groundwater pressure, seismic loading, construction surcharge loads, and pre-stressing loads or post tensioning loads on tiebacks. Also, it should consider the freeze-thaw action on the face of excavations, expansion and contraction of shoring elements, construction vibrations and compatibility with the design of proposed waterproofing and drainage systems for the sub-surface levels.

Any temporary retaining system will most likely receive tiebacks to harness the retaining wall movements at the top. Upon completion of construction, the upper tiebacks shall not be cut until the gap between the foundation wall and the retaining system is backfilled with OPSS approved granular material and compacted to the specified target densities. Cutting the tiebacks will cause a sudden relief of the fill supporting the existing buildings and can cause damages to the existing adjacent properties.

A preconstruction survey should be carried out at the outset of the Project. The magnitude of ground movements adjacent to the excavation should be monitored throughout the construction. The threshold alert level for movement adjacent to the existing building should be determined by the shoring designer. Stockpiling of soil beside the excavations should be avoided. The weight of the stockpiled soil could lead to overstressing the shoring system.

It is recommended that the Client retain contractors and designers who have significant experience with excavations performed under similar soil conditions. Shop drawings should be submitted to the designers and reviewed by the geotechnical engineer well in advance of mobilization.

The preliminary lateral earth pressure parameters to assist designers and Contractors with shoring designs through soil are discussed in Section 7.10 below.

7.3.4 Soil Anchor (Tieback) Design Parameters

If tiebacks are needed, the following design parameters per the Limit States Design (LSD) method can be used. The soil anchors or tiebacks can be designed based on frictional stress between the grout and sandy gravel/gravely sand fill and native sandy silt till. The Ultimate Limit States (ULS) and Serviceability Limit States (SLS) bond stress values must be based on both performance and structural criteria. However, based upon typical published values, the unfactored ULS bond stress for straight shaft pressure-grouted anchors along the anchor bond zone values may be taking approximately 250 kPa for sandy silt till, and 200 kPa for sandy gravel/gravelly sand fill as per Ground Anchors and Anchored System (FHWA-IF-99-015).

The existing sandy gravel/gravelly sand fill is coarse-grained material with high hydraulic permeability. Grout through such materials could be challenging as it may seep through the soil pores. The contractors and designers of shoring system should count for such influence and make a proper adjustment in their design.

CFEM (2006) recommends a geotechnical resistance factor of 0.3 be applied to the empirical unfactored ULS values. Higher stress values may be used if performance load testing in the field is conducted to prove the capacities. If performance testing is carried out at the outset of the Project, then a resistance factor of Φ=0.4 could be applied.

7.3.5 Subgrade Preparation

The excavations for the foundations of the proposed four-storey with a basement level structure are generally expected to extend down to sound bedrock. Based on the recent boreholes the sound bedrock is expected to be encountered at approximate depths between 2.3 to 3.2 mbgs which is corresponding to approximate El. 84.6 to 83.6 masl. The footings of the proposed building are expected to be founded at an approximate elevation of El. 84.0 ± 0.5 masl and for the proposed retaining wall at an approximate elevation of 83.0 ± 0.3 masl. Therefore, moderate bedrock excavation will be required to achieve the desired elevations which is expected to generate a manageable amount of excavated rock materials.

Subgrade preparation for footings founded on rock will involve the removal of all soils and weathered bedrock to expose a sound limestone bedrock. Any pieces of rock that can be manipulated by conventional excavation equipment should be removed, and as directed by the geotechnical engineer. Final subgrade surfaces should be brushed and/or air blown clean, and dry. The exposed bedrock surface should be examined and approved by the geotechnical engineer to confirm the competency to support the design bearing pressures.

Confirmation of bedrock quality during construction will require the contractor to perform probing of the bedrock using 50 mm diameter drill holes drilled to a depth of 1.5 m within the footings. These holes will need to be reviewed by the Geotechnical Engineer to confirm that no significant mud seams or voids exist at the footing location. If mud seams are found, localized areas of the footings may need to be lowered below the mud seam, or footing sizes increased to lower design bearing pressures accordingly. The locations of these probe holes should be selected under the direction of the geotechnical engineer during construction. Contractors should plan for one probe per pad footing and a minimum or 1 probe every 6 m in strip footings.

7.3.6 Temporary Construction Dewatering

Groundwater was observed in all monitoring wells. In in BH22-2 and 22-5 which were drilled within the proposed building footprint, the groundwater was observed within the bedrock. The groundwater in BH22-6 at the proposed retaining wall was observed within the overburden. The observed groundwater is below the proposed foundation level. However, groundwater and surface runoff water may seep and accumulate at the bottom of the excavations due to seasonal changes and extreme weather events. Water quantities will depend on seasonal conditions, the depths of excavations, and the duration that excavation is left open. Furthermore, if excavations intercept existing or former service trenches, then the backfill in these trenches could act as a drain supplying unexpected offsite water into excavations. These trenches should be plugged at the outset of construction in an attempt to mitigate this possibility. Therefore, it is expected that more intense dewatering will be required during the construction stage for this Site to keep the excavation reasonably dry.

Contractors should be prepared to handle any surface or groundwater infiltration by ditching, pumping and/or other methods in order to maintain dry working conditions. Recommendations for appropriate dewatering measures beyond conventional sump pump techniques or other more intensive dewatering systems (e.g., well points or other specialized methods) to effectively lower the static groundwater level shall be provided by a specialized dewatering contractor. A Permit to Take Water (PTTW) from the Ontario Ministry of the Environment, Conservation and Parks (MECP) will be required if the quantity of water to be pumped from the Site exceeds 400,000 L/day. For expected groundwater extraction between 50,000 and 400,000 L/day, an Environmental Activity and Sector Registry (EASR) permit is adequate. Application for Permit to Take Water is not anticipated for this Site based on the observed level of water in the monitoring well. However, if groundwater encountered during construction, the contractor should decide on an adequate permit as per applicable regulations. Due to predicted seasonal groundwater fluctuations, application for Environmental Activity and Sector Registry (EASR) is recommended.

The hydraulic conductivity values for the sandy gravel/gravelly sand fill is expected to be $\geq 1x10^{-3}$ m/s, and for the sandy silt till is expected to be approximately $1x10^{-5}$ to $1x10^{-6}$ m/s. These hydraulic conductivity values are estimated based on soil gradation analysis. In-situ percolation tests were not performed as part of this investigation. The provided hydraulic conductivity values can be used for the selection of the pump capacity for dewatering.

It is noteworthy that dewatering can result in ground settlement that extends beyond the immediate area of dewatering. It is recommended that the contractor evaluates the possible impact of dewatering on nearby structures, buried services, and roadways, and uses methods that will control such an adverse impact. A pre-construction survey documenting the conditions of nearby settlement-sensitive facilities and infrastructure should be completed prior to construction.

7.4 Foundations

It is important to emphasize that at the time of preparing this report, McIntosh Perry has not been provided with the proposed service loads or foundation details for the proposed building or the retaining wall. Based on observation during the site investigation, we recommend that the proposed building to be supported on a shallow foundation system founded on sound bedrock at an approximate elevation of El. 84.0 ± 0.5 masl. Also, we recommend that the proposed retaining wall to be supported on shallow foundation founded on bedrock at an approximate elevation of 83.0 ± 0.3 masl.

7.4.1 Geotechnical Bearing Resistance for the Proposed Building

Provided there are no continuous soil-filled seams or mud seams present at shallow depth in the bedrock below the founding level, conventional pad and strip footings founded on the sound limestone bedrock, a factored bearing resistance of 1,000 kPa under Ultimate Limit States (ULS) conditions is recommended for the proposed building. This includes for a geotechnical resistance factor of $\Phi = 0.5$. The factored ULS bearing resistance was estimated using the Rock Mass Rating (RMR) method by Bieniawski (1989).

The size of the selected footing shall be determined by structural engineer. The selected size of the footing shall have adequate compressive strength to provide resistance to the structural loads from the building and to avoid failure in concrete material under the applied pressure. Designers should keep footing dimensions to a minimum of 1.0 m for pad footings, and 0.75 m for strip footings regardless of the bearing pressure being used.

Provided the bedrock surface is properly cleaned of soil and weathered material at the time of construction, settlements under the ULS condition is expected to be negligible. Therefore, there is no corresponding design bearing pressure recommended under Serviceability Limit State (SLS) conditions for bedrock.

Subgrade preparation for footings founded on bedrock will involve the removal of all soils and weathered rock to expose sound bedrock. Any pieces of rock that can be manipulated by conventional excavation equipment should be removed, as directed by the geotechnical engineer. Final subgrade surfaces should be brushed and/or air blown clean, and dry. The exposed surface should be examined by the geotechnical engineer to assess its competency.

If the bedrock surface needs to be leveled or the grade is required to be raised between the approved sound bedrock subgrade and the design footing elevation, it is recommended to use a lean mix concrete of compressive strength 25 MPa or greater, as opposed to with granular fill soils to avoid differential behavior. If lean mixed concrete is used below any footings, it must extend a minimum of 0.3 m beyond the edge of the footing and then downward at a 1H:1V. Recommended design bearing pressures on lean mix concrete would be the same as those for the bedrock, provided that the underlying subgrade has been approved by the geotechnical engineer.

Confirmation of bedrock quality during construction will require probing of the bedrock at footing locations using 50 mm diameter holes drilled to a depth of 1.5 m within the footprint of footings. The locations of these probe holes should be provided under the direction of the geotechnical engineer during construction. These holes will need to be reviewed by the geotechnical engineer to confirm that no significant mud seams or voids exist. If mud seams are found, localized areas of the footings may need to be lowered below the mud seam, or footing sizes increased to lower design bearing pressures accordingly.

7.4.2 Geotechnical Bearing Resistance for the Proposed Retaining Wall

As discussed, no information regarding the proposed retaining wall foundation depth or type, or the proposed retaining wall type (i.e., cast in place concrete, concrete blocks, armor stone) was communicated to the author of this report. It was assumed that the retaining wall be supported on a concrete strip footing at an approximate elevation of 83.0 ± 0.3 masl.

The limestone bedrock quality within proximity of the proposed retaining wall was observed to be very poor based on recorded RQD values of the retrieved rock cores from BH22-6. For such rock quality with very closely spaced discontinuities, CFEM 2006 recommends foundation design to be undertaking based on soil mechanics approach.

Based on the subsurface investigation results, the encountered bedrock maybe classified as low frost susceptibility. However, due to the poor quality of the retrieved rock core, the rock may be classified as moderately frost susceptible. The frost penetration depth for the subject Site is 1.8 m below the surface. Frost penetration depth is estimated based on the OPSD 3090.101, Foundation Frost Penetration Depths for Southern Ontario. Therefore, foundation of the retaining wall should be provided with a minimum of 1.8 m of earth cover or equivalent thermal rigid insulation for frost protection purposes.

Provided there are no continuous soil-filled seams or mud seams present at shallow depth in the bedrock below the founding level, conventional strip footing of a minimum dimension of 0.75 m founded on the limestone bedrock can be designed with a factored ULS bearing resistance of 200 kPa for the proposed retaining wall. This includes for a geotechnical resistance factor of $\Phi = 0.5$. The factored ULS bearing resistance was estimated using the general bearing capacity equation with estimated effective shear angle (ϕ') of 35° and effective cohesion of 20 kPa. The SLS bearing resistance may be taken as 100 kPa for the retaining wall.

Subgrade preparation for footings founded on bedrock will involve the removal of all soils and any pieces of the weather rock that can be manipulated by conventional excavation equipment, as directed by the geotechnical engineer.

If the bedrock surface needs to be leveled or the grade is required to be raised between the approved subgrade and the design footing elevation, it is recommended to use a lean mix concrete of compressive strength 15 MPa or greater. Engineered Fill may also be used for leveling below the retaining wall foundation only. If lean mixed concrete or Engineered Fill is used below retaining wall foundation, it must extend a minimum of 0.3 m beyond the edge of the footing and then downward at a 1H:1V. Recommended design bearing pressures on lean mix concrete or Engineered Fill would be the same as those for the bedrock, provided that the underlying subgrade has been approved by the geotechnical engineer. If mud seams are found, localized areas of the footing may need to be lowered below the mud seam, or footing sizes increased to lower design bearing pressures accordingly.

7.4.3 Lateral Resistance of the Retaining Wall Foundation

The factored ultimate resistance of the footings to lateral loading 'shear resistance for sliding' across the interface between the footing, and the bedrock may be calculated using Mohr-Coulomb criterion below with load and resistance factors given in Table 7-1.

$$\tau = f_c c' + (\sigma - f_U U) f_\phi tan\phi'$$

where c' is cohesion, ϕ' is shearing angle, U is water pressure, and σ is the normal stress on the sliding surface.

Table 7-1: Minimum Lateral Load and Resistance Factors after Meyerhof (1984) (Wyllie 2009)

Category	Item	Load Factor	Resistance Factor
Loads	Dead Loads, (f_{DL})	1.25 **(0.8)***	--
	Live Loads, Wind, earthquake, (f_{LL})	1.5	--
	Water Pressure, (f_U)	1.25 **(0.8)***	--
Shear strength	Cohesion "c" - stability, earth pressure, (f_c)	--	0.65
	Cohesion "c" – Foundation, (f_c)	--	0.5
	Friction angle "ϕ", (f_ϕ)	--	0.8

*** The values given in the parenthesis apply to beneficial loading conditions such as dead loads resist overturning or up lift**.

To increase the lateral resistance against sliding, the strip footing shall be anchored to the bedrock by means of rock anchors (i.e., dowels or rebars). Since the bedrock surface is weathered and possibly irregular, the anchors shall be installed at a minimum embedment depth of 1.5 m into the bedrock. The number and interval of the anchors, the embedment length of anchors in concrete shall be designed by a structural engineer.

7.4.4 Uplift and Overturning Resistance

The dead load of the building and backfill soil can provide resistance to uplift and overturning forces that the proposed building foundation may experience. Additional resistance can be provided by increasing the dead weight of the structure using additional concrete elements or by using rock anchors.

Like soil anchors, grouted rock anchors may be designed based on frictional stress between the grout and intact bedrock. The bond zone must be entirely within sound bedrock. The design of rock anchors can be be performed extending the LSD method. The Ultimate Limit States (ULS) and Serviceability Limit States (SLS) bond stress values must be based on both performance and structural criteria. However, based upon typical published values, the unfactored ULS bond stress values for limestone bedded with shale may be approximately 800 kPa to more than 1,400 kPa as per Ground Anchors and Anchored System (FHWA-IF-99-015).

CFEM (2006) recommends a geotechnical resistance factor of 0.3 be applied to the empirical unfactored ULS values. Due to poor rock quality, performance testing shall be carried out at the outset of the Project to verify the the anchor capacities. Performance tests shall be performed on the first three production anchors installed and thereafter on a minimum of 2% of the remaining production anchors. The resistance factor can be increased to Φ=0.4 based on the performance testing results. Designers may take the approach that working stress value is approximately equivalent to the SLS value. We recommend that a conservative allowable working stress value of 240 kPa be used to calculate the length of the required bond zone. The bond zone must be entirely within sound bedrock.

In order to mobilize the shear stress in the rock, the load at the top of the anchor must be properly transferred through the upper bedrock to the bond zone to prevent progressive grout fail and ensure proper performance. Therefore, a "free length" is required through the foundation element, the weathered rock zone, and down to the bond zone.

The mass of rock mobilized by a rock anchor may be assumed to be based upon a 60°cone drawn upward from a point located at the lower one-third point of the bond zone and spaced such that the theoretical cones do not overlap. Designers should review the spacing of anchors and take into account of any overlapping cones (i.e., avoid doubling-up on rock mass calculations for overlapping cones). The bulk unit weight of bedrock may be assumed to be approximately 26 kN/m^3. The corresponding buoyant unit weight would be approximately 16 kN/m^3. It is recommended that designers consider the water level to be near the surface, and therefore, use submerged unit weights for the rock mass calculations.

7.5 Slab-on-Grade

A typical floor slab loading for a lightly loaded slab on grade would involve a maximum pressure of 20 kPa. If this is not the case, then McIntosh Perry should be retained to perform additional consulting in regard to the design of the floor slab. For design purposes and based upon a properly prepared native subgrade surface covered with 200 mm of Ontario Provincial Standard Specification (OPSS) 1010 Granular A, a typical preliminary modulus of subgrade reaction appropriate for the slab design would be approximately 25 MPa/m on Engineered Fill compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD). Alternative values would require additional analysis and testing.

For the unheated portions of the buildings or slabs that are exposed to cold temperatures, the slab subgrade shall be insulated. The insulation shall be load-bearing and spread below the slab for the entire width. It is the designers' responsibility to determine the thickness of insulation based on the required R-value, equivalent to 1.8 m of earth material insulation value. The insulation shall wrap around the slab thickenings.

A capillary moisture barrier consisting of a layer of either 19 mm clear stone or an OPSS 1010 Granular A at least 200 mm thick should underlie the slab. This layer should be compacted to 100%of its SPMDD and placed on approved subgrade surfaces.

Subgrade preparation below floor slabs will involve the removal of all soils and weathered bedrock to expose the sound limestone bedrock. Any pieces of rock that can be easily manipulated by conventional excavation equipment should be removed. Final subgrade surfaces should be brushed and/or air blown clean, and dry. The exposed bedrock surface should be examined and approved by the geotechnical engineer. Any new fill used to raise the grade between the approved bedrock subgrade and the floor slab should be considered as Engineered Fill.

7.6 Grade Raise

No information was provided regarding any proposed grade raise for the Site. However, grade-raise is not expected to result in long-term settlement. Therefore, a grade raise up to 1.0 m is allowed for this Site.

7.7 Frost Protection

Frost penetration depth is 1.8 m below the surface for the subject Site. Frost penetration depth is estimated based on the OPSD 3090.101. For protection against frost effects, earth cover of 1.8 m must be provided for all footings in unheated or isolated structures in the Ottawa area. All footings for heated structures must be provided with a minimum of 1.5 m of earth cover. In the absence of adequate soil cover, equivalent synthetic insulation material can be used.

Should construction take place during winter, surfaces that support foundations or Engineered Fill must be protected by contractors against freezing for the entire duration of construction or until adequate soil cover is in place. Backfill soils should not be placed in a frozen condition or placed on frozen subgrades.

7.8 Site Classification for Seismic Site Response

Selected spectral responses in the general vicinity of the site for a 2% chance of exceedance in 50 years (2500 years return period) are as indicated in Table 7-2, based on the National Building Code Seismic Hazard published by Natural Resources Canada 2010.

Table 7-2: Selected Seismic Spectral Responses (2% in 50 Yrs)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.634	0.307	0.137	0.046	0.323

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. Bedrock is sampled and confirmed by diamond coring in some three boreholes within the footprint of the proposed building and one borehole within proximity of the proposed retaining wall.

Given the relatively shallow bedrock across the site, the foundation design option considered for this project is for all footings to be bearing on bedrock. Otherwise, differential settlements will be inevitable. With the assumption of all column loads and lateral resistance systems loads transferred to the bedrock, the site can be classified as Seismic Site Class (C).

7.8.1 Liquefaction Analysis

Liquefaction susceptibility of the fill and sandy silt layers was evaluated using SPT test results following recommendations of the Canadian Foundation Engineering Manual (CEEM).

Based on the calculated CSR and corrected SPT values, Figure 6.7 from CFEM can be used to evaluate the fill and the sandy silt till layers susceptibility to liquefaction. Both layers and, therefore, the site found to be non-susceptible to liquefaction.

7.9 Global Stability of the Retaining Wall

It is important to emphasize that at the time of preparing this report McIntosh Perry was not provided any information or whatsoever with respect to the geometry, type (i.e., cast in place concrete, concrete blocks, armor stone), or foundation level of the proposed retaining. Based on the site investigation results, we recommend that the proposed retaining wall to be supported on a shallow foundation founded on bedrock at an approximate elevation of 83.0 ± 0.3 masl.

Typically, accepted factor of Safety (FS) values against global stability in practice for MSE system are 1.5 under static conditions and 1.1 under seismic "pseudo-static" conditions.

Global slope stability analyses were performed to assist the global stability of the proposed retaining wall using SLOPE/W 2021 software. The analyses were performed using the Limit Equilibrium method, using entry and exit method to define the critical failure mode. In general, this approach calculates a factor of safety that represents the ratio of resisting forces (i.e., shear strength, friction, etc.) to the driving forces (weight, external loadings, acceleration, etc). Both static and seismic conditions were simulated using soil parameters provided in Table 7-3.

7.9.1 Soil Model

The field and laboratory test results were used to estimate the mechanical properties of the encountered fill and soil layers and are presented in Table 7-3. The geometry was inferred based on the survey drawings dated August 09, 2021 conducted by Fairhall Moffatt and Woodland Limited and provided to us by the Client. The retaining wall was assumed to be 2.8 m high supported on weathered bedrock at an approximate elevation 83.0 ± 0.3 m and retaining 3 m of soil/backfill back sloped at 3H:1.2V. Immediately behind the wall, the backfill is composed of Engineered Fill sloped at 1:1 at minimum with weeping tile for drainage installed at the bottom of the retaining wall. The weathered bedrock was conservatively simulated as a soil using Mohr-Coulomb model. The foundation of the retaining wall was assumed to be reinforced with dowels or rock anchors to provide additional resistance against base sliding and seismic loadings.

The analyses were carried out using the Morgenstern-Price method considering circular slip surface for static conditions (long-term stability) and Pseudo-static conditions for seismic stability with seismic load equals 50% of the Peak Ground Acceleration (PGA).

The groundwater level was assumed at 83.7 masl for static analyses and at the surface for seismic loading conditions.

Table 7-3: Geotechnical Model Parameters

Soil Type	Total Unit Weight (kN/m^3)	Pseudo-Static Conditions (Seismic)		Static Conditions (Long Term)	
		Undrained Shear Strength, S_u (kPa)	Friction Angle, ϕ	Drained Cohesion, c' (kPa)	Friction Angle, ϕ'
Retained Soil (Backfill and Till)	18.5	0	30	0	30
Engineered Fill	20.0	0	30	0	30
Weathered bedrock	25.0	15	35	15	35
Retaining Wall	24	High Strength Material			

7.9.2 Slope Stability Evaluation

The results of the global slope stability analyses under static and seismic loading are presented in Table 7-4 and in Appendix F. The Factor of Safety under both loading conditions meet the minimum factor of safety requirements as per the City of Ottawa Slope Stability Guidelines for Development Applications (2012).

Table 7-4: Global Stability - Factor of Safety

Analysis	Factor of Safety	
	Static	Seismic
Circular Stability	≥ 2.4	≥ 1.4

7.9.3 Design Recommendations for the Retaining Wall

The following are recommendations for designing the proposed retaining wall:

- The retaining wall can be constructed with reinforced concrete supported on a strip footing as discussed in Sections 7.4.2 and 7.4.3.
- To increase the lateral resistance against sliding, the strip footing shall be anchored to the bedrock by means of rock anchors (dowels or rebars). Since the bedrock surface is weathered, the anchors shall be installed at a minimum embedment depth of 1.5 m in the sound bedrock. The number and interval of the anchors, and the embedment length of anchors in concrete shall be designed by a structural engineer.
- Free draining materials conforming to OPSS 1010 should be used for backfill behind the wall. Engineered Fill as per Section 7.12.1 should be used immediately behind the wall.
- Weeping tile should be installed behind the wall at the base of the retaining wall to prevent static pore water pressure building up behind the wall.
- It is recommended to limit any vegetation behind the wall to small bushes and shrubs and avoid planting trees behind the wall as mature trees' roots may grow against the wall adding additional lateral loads on the wall.

- Surface loads or surcharges should be limited to a distance equal to twice the height of the wall from the crust of the retained soil.
- The foundation minimum frost protection requirements are discussed in Section 7.7.

7.10 Lateral Earth Pressure

Active earth pressure is the minimum value of the lateral earth pressure, which a soil mass can apply against an unrestrained structure. On the other hand, passive earth resistance is the maximum value of lateral pressure, which can be mobilized in the soil by the structure moving toward the soil mass.

This report provides coefficients of lateral earth pressure. Static lateral pressure can be calculated by using the following equation:

$$P_h = K \times (\gamma h + q)$$

In this equation, the provided unit weight of the soil, γ, is for a moist soil above the groundwater table. Pseudo-dynamic effects of seismic activities are considered based on Mononobe-Okabe method.

The backfill material shall be 'free draining' and to follow OPSS 1010 recommendation for grain size distribution. However, if there is a chance of hydrostatic pressure build-up behind the wall, the designer shall consider the fluid pressure in the analysis of retaining wall pressure.

Calculation of all live load and dead load surcharges are the responsibility of the bridge designer.

The PGA for this Site is 0.323 based on Site Class C and probability of exceedance per annum of 0.000404.

Table 7-5: Lateral Earth Pressure Design Parameters for Backfill and Native Soil

Design Parameters	Material		
	Granular A	Granular B	Native Sandy Silty Clay/Silt and Clay
Unit Weight, γ (kN/m^3)	21	20	18
Internal Friction Angle, ϕ (°)	32	30	28
Static at-rest pressure, K_o	0.47	0.50	0.53
Static active pressure, K_a	0.31	0.33	0.36
Static passive pressure, K_p	3.25	3.00	2.77
Dynamic active pressure, K_{AE}	0.66	0.72	0.78
Dynamic passive pressure, K_{PE}	1.29	1.29	1.29

The above noted lateral pressure coefficients are calculated assuming the wall back angel is vertical and the backslope of the retained soil is horizontal. The wall-soil interaction angle is assumed to equal to 0.5ϕ as per CFEM. If Engineered Shoring is used, then designers should refer to CFEM for design assistance and a geotechnical engineer should be retained to perform the shoring design review.

7.11 Waterproofing and Permanent Drainage

The building basement can be designed as a fully waterproof 'bath-tub' design (without external perimeter drains to avoid potential adverse impacts due to moisture movements in the immediate areas around the proposed building footprint.

If the Project designers consider a drained basement design, the options for a perimeter drainage system are to use a conventional drainage tile. If a traditional perimeter drain system is installed, it may be constructed with 100 mm diameter weeping tiles placed on a 150 mm bed of 19 mm clear stone and then covered with 150 mm of the same stone. The stone and weeping tile should be wrapped with a non-woven geotextile filter cloth. The perimeter drainage system should be placed at the footing level and be connected to a frost-free outlet, such as a sump or storm sewer. Design drawings shall provision drainage outlet.

Full waterproofing membranes such as a WR Meadows Mel-ROL PRECON or equivalent type product for walls and under-slab will be required. These types of membranes adhere to the concrete and provide a waterproof seal between the membrane and poured concrete. Their installation would require that excavations be planned large enough for safe worker accesses on the exterior of the foundation wall to allow installation. Water stops should be installed at cold joints in the foundation walls and floor-wall joints.

Under floor drainage is recommended for this structure based on expected groundwater level fluctuation due to seasonal changes. Under floor drainage systems should be placed at a minimum 4.5 m spacing between drains, running in one direction, and set at a minimum of 0.45 m below the underside of floor slabs.

7.12 Backfill

7.12.1 Engineered Fill

All new fill soils that underlie footings, or other structural applications, behind retaining wall is considered as Engineered Fill. Engineered Fill may be required to raise the grade above the approved subgrade. Engineered Fill must meet the strict requirements as shown below:

- Typically, a crushed well-graded material such as an OPSS 1010 Granular A or Granular B Type II is suitable. However, other suitable granular materials may be proposed and considered depending on the Site-specific conditions;
- Prior to placing any Engineered Fill, all unsuitable fill materials must be removed, and the subgrade approved by a geotechnical engineer. Any deficient areas should be repaired prior to placement;
- Engineered Fill shall be placed in maximum loose lifts of 300 mm and adequately compacted to achieve 100% of its SPMDD. Engineered Fill must have full-time compaction testing by a geotechnical personnel; and

- At a minimum, the Engineered Fill beneath foundations should extend laterally a distance of 0.3 m beyond the edge of the footings and then be sloped downward and outward at 1H:1V slope.

7.12.2 Exterior Foundation Wall Backfill

The backfill placed against exterior foundations shall be free draining granular material meeting the grading requirements of an OPSS 1010 Granular B Type I or equivalent granular material.

The exterior backfill should be placed and compacted as outlined below:

- Backfill should not be placed in frozen condition, or placed on a frozen subgrade;
- Backfill should be placed and compacted in maximum loose lift thickness compatible with the selected construction equipment, but not thicker than 0.3 m. Each lift should be uniformly compacted to achieve 98% of its SPMDD.
- In landscaped areas the upper 0.3 m of backfill below landscape details should be a low permeable soil to reduce surface water infiltration;
- Backfill should be placed uniformly on both sides of the foundation walls to avoid build-up of unbalanced lateral pressures, or alternatively wait until the basement walls are tied together with the floor above before backfilling the exterior foundation wall;
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to achieve 98% of its SPMDD;
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95% of its SPMDD;
- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall;
- Entrance slabs should be founded on frost walls or alternatively have insulation details developed to prevent frost heaving at the building entrances; and
- In areas where the building backfill underlies pavement, sidewalk, or other hard landscaping, the excavation should have a frost taper incorporated to prevent differential heaving around the building.

7.13 Underground Utilities

At the subject site, the burial depth of water-bearing utility lines is typically 2.4 m below the ground surface or as dictated by local applicable codes. If this depth is not achievable, equivalent thermal insulation should be provided. The contractor should retain a professional engineer to provide detailed drawings for excavation and temporary support of the excavation walls during construction.

The Occupational Health and Safety Act (OHSA) of Ontario indicated that side slopes in the sand above the water could be classified as Type 3 soil and sloped no steeper than 1H:1V or be shored. Below the groundwater level, the soils are considered to be Type 4 Soil and the excavation side slopes must be sloped from their bottom cut back at 3H:1V. Otherwise, lateral support for all excavations such as trench boxes should be used.

The engineer designing utilities shall ensure the proposed utility pipes can tolerate compaction loads.

The recommendations within this section are intended to be a supplement to, and not a replacement of the most recent local municipal requirements.

7.13.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials:

- Bedding for buried utilities should consist of an OPSS 1010 "Granular A" material and be placed in accordance with municipal requirements, assuming the subgrade soils are not allowed to become disturbed. All utility pipes and high amps electrical conduits shall receive a minimum of 150 mm bedding.
- The use of clear stone is not recommended for use as pipe bedding. The voids in the stone may result in a low gradient water flow and infiltration of fines from the surrounding soils and cover materials, causing settlement and loss of support to pipes and structures.
- The cover material should be a service sand material or an OPSS 1010 "Granular A". The dimensions should comply with the pertinent specification section.
- The bedding, spring line, and cover should be compacted to at least 98% of its SPMDD.
- All covers are to be compacted to 100% SPMDD if they are intersecting structural elements.
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.
- If the encountered subgrade below the utility line is clay or silt, it is recommended that the utility bedding be separated from the native soil by a non-woven geotextile.

7.13.2 Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches underlying pavement areas, the backfill should be placed and compacted in uniform lift thickness compatible with the selected compaction equipment and not thicker than 300 mm. Each lift should be compacted to a minimum of 98% of its SPMDD.
- The backfill placed in the upper 0.3 m below the pavement subgrade elevation should be compacted to a minimum of 100% of its SPMDD.
- Excavation backfill should attempt to match the texture of the existing adjacent soils. If imported materials are used, side slopes with frost tapers are recommended. Frost tapers should be a back-slope of 10H:1V through the frost zone, (i.e.,1.4 m from finished grade).
- During backfilling, care should be taken to ensure the backfill proceeds in equal stages simultaneously on both sides of the pipe; and
- No frozen material should be used as backfill; neither should the trench base be allowed to freeze.

The quality and workmanship in the construction are as important as the compaction standards themselves. It is imperative that the guidelines for the compaction be followed for the full depth of the trench to achieve satisfactory performance.

7.13.3 Clay Seals

Clay seals are recommended as a seepage barrier for all utility trenches. In the absence of clay seals, there is a potential for the trench to act as a drain into the proposed building. To avoid such an effect, clay seals are recommended at intervals along the utility line alignment at a frequency prescribed by the civil engineer, and at the property lines. The clay seal shall be constructed of low permeability material, such as silty clay, to a minimum thickness of 1.0 m, clay seal material shall be according to OPSS 1205 and OPSD 802.095. The clay seal (i.e., silty clay) material shall be compacted to a minimum of 95% SPMDD in loose lifts of no thicker than 300 mm. Acceptable imported clay material may be used for the construction of the clay seals.

8.0 CEMENT TYPE AND CORROSION POTENTIAL

Four (4) 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 and the laboratory results for the chemical analysis are shown in appendix D.

The American Water Works Association (AWWA) publication ‘Polyethylene Encasement for Ductile-Iron Pipe Systems’ ANSI/AWWA C105/A21.5-10 dated October 1, 2010 assigns points based on the results of the above tests. A soil that has a total point score of 10 or more is considered to be potentially corrosive to ductile iron pipe. Based on the results obtained for the sample submitted, the Site soils, are considered to be moderately corrosive to neutral to buried steel elements.

The analytical results of the soil samples were compared with applicable Canadian Standards Association (CSA) A23.1-04 and are given in Table 8-1 below.

Table 8-1: Additional Requirement for Concrete Subjected to Sulphate Attack

Class of Exposure	Degree of Exposure	Water Soluble Sulphate in Soil Sample (%)	Cementing Material to be Used
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.2 – 2.0	HS or HSb
S-3	Moderate	0.1 – 0.2	MS, MSb, LH, HS, or HSb

The chemical sulphate content analyses for selected soil samples tested indicate a sulphate concentration of maximum of a 0.0087 % in soil, as shown in Table 6-4, indicating a “moderate to low” risk for sulphate attack on concrete material.

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.

9.0 PAVEMENT STRUCTURE

No details are provided on the traffic loads. However, it is understood that the parking lot and surrounding paved area is to be used frequently by light to medium weight vehicles and occasional fire trucks. The pavement structure is most likely to be placed on Engineered Fill material overlaying the native soil. The subgrade should be proof rolled and all loose and soft spots should be subexcavate and replaced with OPSS 1010 Granular A or B Type II compacted to 98% SPMDD under the direction of a geotechnical engineer.

Grading fill below the pavement structure should Engineered Fill conforming to OPSS 1010 Granular A or B Type II materials compacted to 98% SPMDD. The pavement structure proposed in this design considers the relatively low traffic movement of lightweight passengers to heavy fire trucks.

The proposed pavement structure for light-weight vehicles parking area and access road is included in Table 9-1.

Table 9-1: "Medium Duty" Pavement Structure

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

The proposed pavement structures are designed for proof rolled subgrades. The base and subbase materials, i.e., Granular A for base and Granular B Type II for subbase, shall conform to OPSS 1010. Both base and sub-base should be compacted to 100% SPMDD. The recommended Superpave 12.5 and 19 can be replaced with HL-3 and HL-8, respectively if required.

The light-duty pavement is expected to render a lower quality performance and it is only recommended for areas accessible only to light-weight passenger vehicles. Access and fire routes shall receive the heavy-duty design.

10.0 CONSTRUCTION CONSIDERATIONS

McIntosh Perry requests to be retained once the plans and specifications are finalized to review the documents and ensure the recommendations in this report are adequately addressed.

The recommendations presented in this report are based on the assumption that an adequate level of construction monitoring by qualified geotechnical personnel during construction will be provided. The bedrock quality during construction should be confirmed by extending a 1.5 m probe holes into the bedrock within the footing footprints. These holes will need to be reviewed by the geotechnical engineer to confirm that no significant mud seams or voids exist. All bearing surfaces should be inspected and approved by experienced geotechnical personnel prior to placing the footings or lean mix concrete slabs.

In addition, an adequate level of construction monitoring should include laboratory and field test during construction. This includes Full time compaction testing of Engineered Fill and part time compaction testing of exterior foundation wall backfill with laboratory testing for the proposed fill soils for this Site. Also, periodic testing of concrete is required.

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 vibration should be kept at a minimal level to avoid soil disturbance and associated unexpected settlement to the nearby roadway, load bearing elements, and utilities. Also, the noise level should be kept at a tolerance level of noise per the City of Ottawa requirements. Vibration and deformation monitoring will be required throughout the construction.

A separate monitoring program should be developed by the shoring designer to monitor the inward movements of the excavation support system to ensure compliance with the design assumptions and avoidance of adverse impacts on nearby structures and buried services. Similar requirements apply for dewatering impacts.

As also noted earlier in this report, the existing fill cannot be used as Engineered Fill, bedding, cover, or any part of the pavement structure. It can only be used as general backfill below the pavement structure.

11.0 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.

LICENSED PROFESSIONAL ENGINEER
M. A. T. AL-KHAZAALI
100512672
June 6/2023
PROVINCE OF ONTARIO

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1083 - 1095 MERIVALE ROAD - MULTI-STOREY BUILDING
OTTAWA, ONTARIO

APPENDIX A
LIMITATIONS OF REPORT

McINTOSH PERRY

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.

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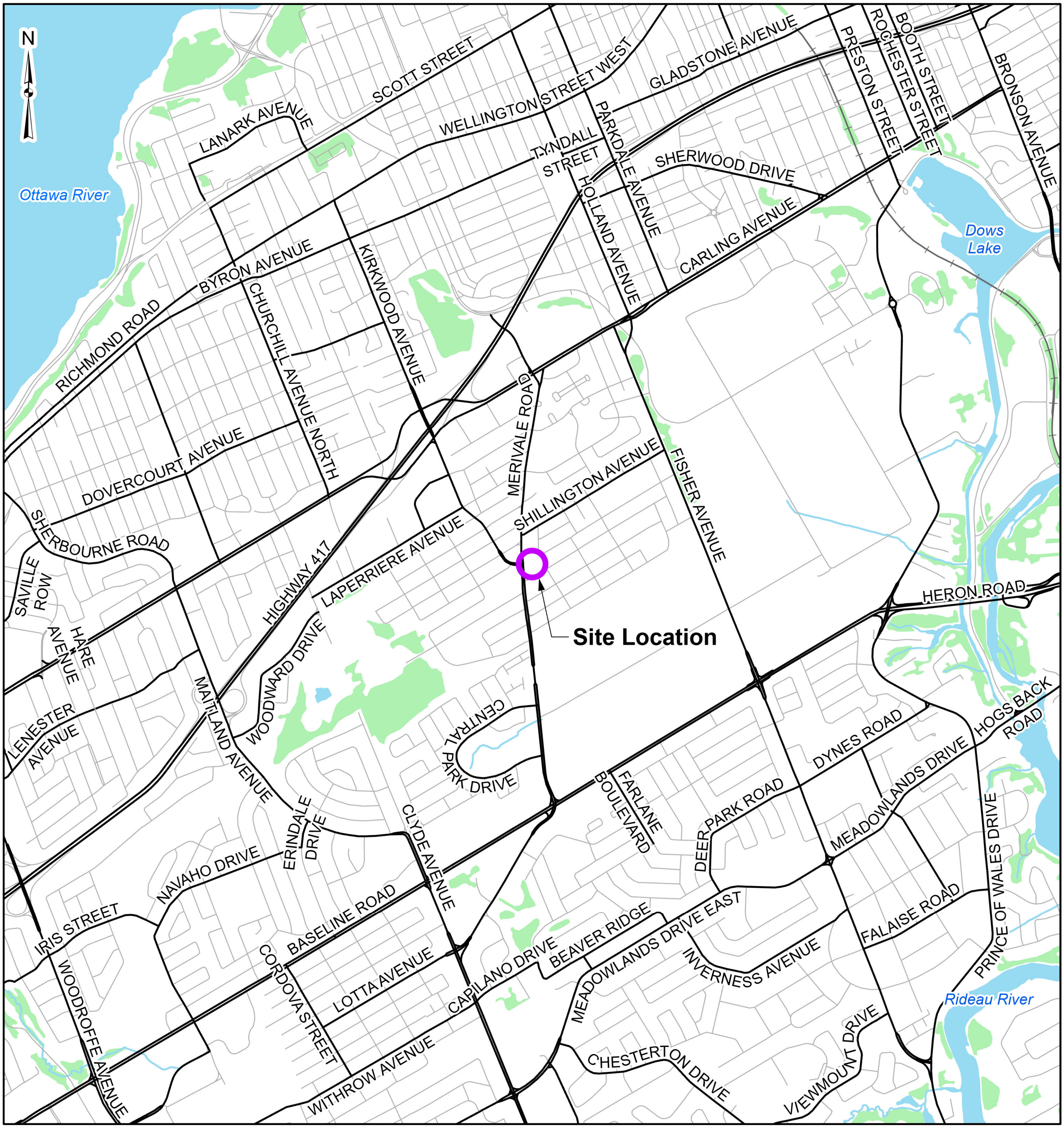
1083 - 1095 MERIVALE ROAD - MULTI-STOREY BUILDING
OTTAWA, ONTARIO

APPENDIX B
SITE AND BOREHOLE LOCATION PLANS

McINTOSH PERRY



LEGEND

- Site Location
- Local Road
- Major Road
- Railroad
- Watercourse
- Waterbody
- Wooded Area

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2023.

CLIENT:	CSV ARCHITECTS
PROJECT:	1083 - 1095 MERIVALE ROAD - MULTI-STOREY BUILDING
TITLE:	SITE LOCATION

MCINTOSH PERRY
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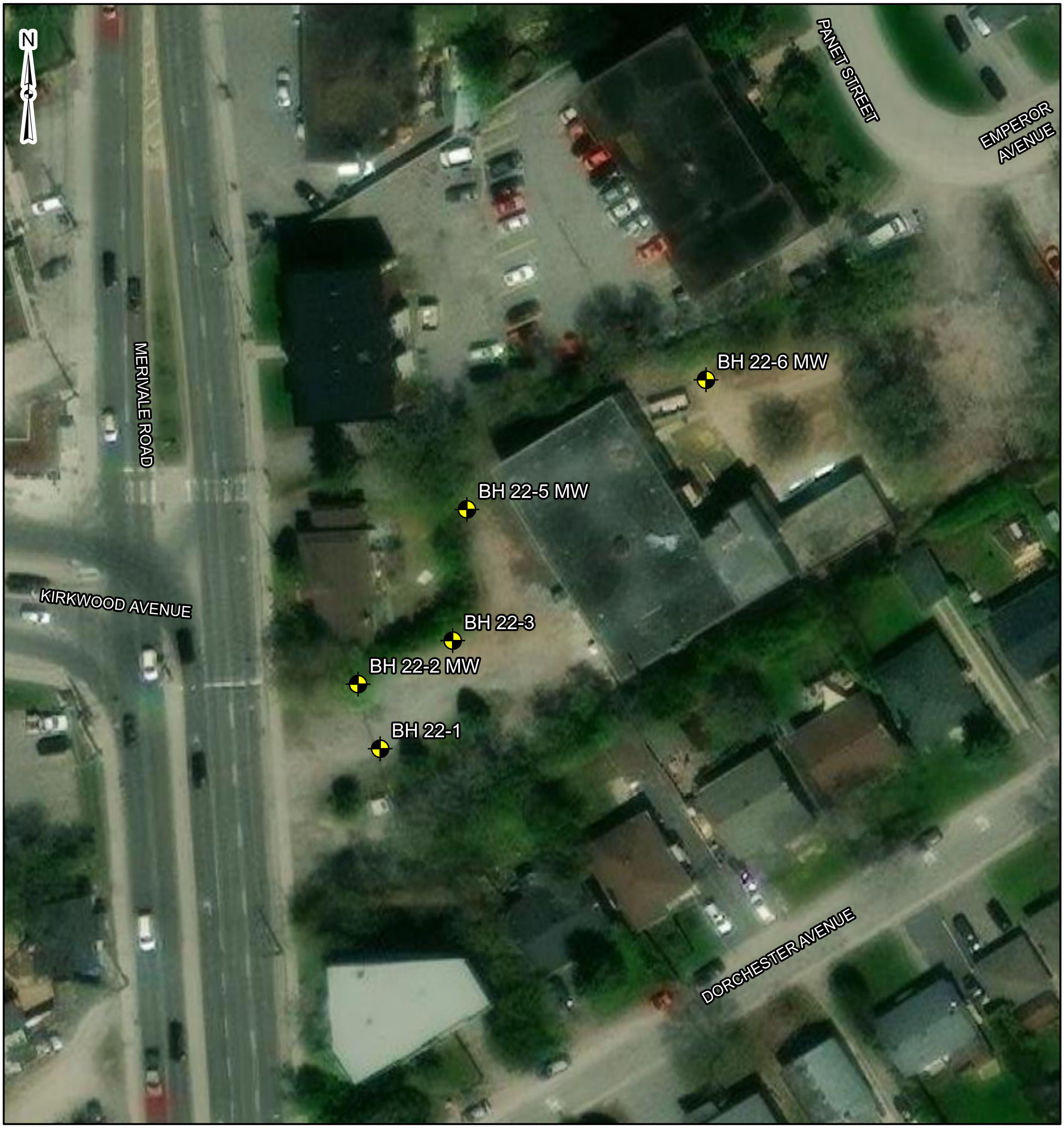
PROJECT NO:	CCO-22-3530	FIGURE:
Date	Feb., 07, 2023	1
GIS	AH	
Checked By	MA	

C:\Users\a.hamed\McIntosh Perry\GIS - Documents\Projects\2022\CCO\CCO-22-3530 CSV_SOGH Apartment_1083 Merivale Road\aprx\Geotech\CCO-23-3530_Geotech.aprx



20 10 0 20
Scale 1:750 Metres

LEGEND

Borehole Locations

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2023.

CLIENT:	CSV ARCHITECTS
PROJECT:	1083 - 1095 MERIVALE ROAD - MULTI-STOREY BUILDING
TITLE:	BOREHOLE LOCATIONS

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PROJECT NO:CCO-22-3530		FIGURE: 2
Date	Feb., 07, 2023	
GIS	AH	
Checked By	MA	

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OTTAWA, ONTARIO

APPENDIX C
BOREHOLE LOGS

MCINTOSH PERRY

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'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	-°	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

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

McINTOSH PERRY

PROJECT NO.: **CCO-22-3530**

PROJECT: Geotechnical Investigation - Proposed Building

CLIENT: CSV Architects

PROJECT LOCATION: 1083 and 1095 Merivale Rd, Ottawa, ON

Drilling Date: Jan-10-2023 - Jan-10-2023

BH Location: Lat: 45.376983; Long: -75.732891

Drilling Equipment: CME 750

Drilling Method: Hollow Stem Augers

Remarks:

BH No: 22-1

Datum: Geodetic

Elevation: 87.2 m asl

Compiled by: JF

Checked by: MAK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)	GROUNDWATER CONDITIONS	DEPTH	ELEVATION
87.2	Fill						0.0	
0.0	**Fill**, sandy gravel, trace clay, dark brown, compact, dry							87
86.8 / 0.4	**Fill**, sandy silt, with gravel and limestone rock fragments, brown, very dense, dry to moist	1	GRAB					
		2	SS	50/ 125mm	8%		1.0	86
		3	SS	50/ 0mm	17%			
85.3 / 1.9	**End of borehole** Auger refusal on inferred bedrock Borehole dry on completion							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH (kPa): Field. Shear Vane (x) & Sensitivity (s); Pocket Penetrometer ✖; ● Quick Triaxial; ○ Unconfined; 20 40 60 80

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

WATER CONTENT (%): 10 20 30 40 50 60 70 80 90

Remarks and Grain Size Distribution (%): Unit Weight (kN/m³), Pocket Penetro. (kPa); GR SA SI CL

GRAPH NOTES: 30/3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity; ○ 8=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCO-22-3530**

PROJECT: Geotechnical Investigation - Proposed Building

CLIENT: CSV Architects

PROJECT LOCATION: 1083 and 1095 Merivale Rd, Ottawa, ON

Drilling Date: Jan-10-2023 - Jan-10-2023

BH Location: Lat: 45.377063; Long: -75.732931

Drilling Equipment: CME 750

Drilling Method: Hollow Stem Augers

Remarks:

BH No: 22-2 MW

Datum: Geodetic

Elevation: 86.9 m asl

Compiled by: JF

Checked by: MAK

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)	Remarks
86.9	Fill					
0.0	**Fill**, sandy silt, with gravel, brown to dark grey, loose, dry	1	GRAB			
86.3 / 0.6	**Fill**, clayey silt, some limestone rock fragments, dark brown, dense, dry	2	SS	37	38%	
		3	SS	24	50%	
85.1 / 1.8	**Silty sand**, some limestone fragments, brown, compact to very dense, moist to wet					
84.6 / 2.3	**Limestone Bedrock**, shale parting, strong, grey, moderately weathered, closely spaced, thinly bedded, poor to fair quality based on RQD.	4	SS	50/ 75mm		
		5	RC	RQD = 44%		TCR=100%
						Compressive Strength = 141 MPa
		6	RC	RQD = 55%		TCR=100%
						Mud seam
						Mud seam
		7	RC	RQD = 66%		TCR=100%
81.3 / 5.6	**End of borehole** Monitoring well installed					

W. L. 81.9 Feb 13, 23

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH (kPa): Field. Shear Vane (x) & Sensitivity (s); Pocket Penetrometer ✖; ● Quick Triaxial; O Unconfined: 20 40 60 80

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

WATER CONTENT (%): 10 20 30 40 50 60 70 80 90

Remarks and Grain Size Distribution (%); Unit Weight (kN/m^3); Pocket Penetro. (kPa); GR SA SI CL

GRAPH NOTES: 30/3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity; O 8=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCO-22-3530**

PROJECT: Geotechnical Investigation - Proposed Building

CLIENT: CSV Architects

PROJECT LOCATION: 1083 and 1095 Merivale Rd, Ottawa, ON

Drilling Date: Jan-10-2023 - Jan-10-2023

BH Location: Lat: 45.377118; Long: -75.732765

Drilling Equipment: CME 750

Drilling Method: Hollow Stem Augers

Remarks:

BH No: 22-3

Datum: Geodetic

Elevation: 86.9 m asl

Compiled by: JF

Checked by: MAK

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)	Remarks and Grain Size Distribution (%) GR SA SI CL
86.9	Fill					
0.0	**Fill**, sandy silt with gravel, trace clay, with limestone rock fragments, brown to dark grey, very dense, dry to moist	1	GRAB			
		2	SS	52	54%	52 34 (14)
		3	SS	50/ 50mm	8%	
84.4 / 2.5	**Limestone Bedrock**, shale parting, strong, grey, weathered to moderately weathered, closely spaced, thinly bedded, poor to excelent quality based on RQD.	4	SS	50/ 25mm	4%	
		5	RC	RQD = 59%		TCR=97%
		6	RC	RQD = 96%		TCR=100% Mud seam
		7	RC	RQD = 67%		TCR=96%
81.3 / 5.6	**End of borehole**					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH (kPa): Field. Shear Vane (x) & Sensitivity (s); Pocket Penetrometer; Quick Triaxial; Unconfined: 20 40 60 80

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

WATER CONTENT (%): 10 20 30 40 50 60 70 80 90

Unit Weight (kN/m³) Pocket Penetro. (kPa)

Depth: 0.0, 1.0, 2.0, 3.0, 4.0, 5.0; Elevation: 86, 85, 84, 83, 82

GRAPH NOTES: 30/3 Upper value = Field Vane Shear Strength, Lower value = Vane Sensitivity; ○ 8=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCO-22-3530**

PROJECT: Geotechnical Investigation - Proposed Building

CLIENT: CSV Architects

PROJECT LOCATION: 1083 and 1095 Merivale Rd, Ottawa, ON

Drilling Date: Jan-10-2023 - Jan-10-2023

BH Location: Lat: 45.377281; Long: -75.732742

Drilling Equipment: CME 750

Drilling Method: Hollow Stem Augers

Remarks:

BH No: 22-5 MW

Datum: Geodetic

Elevation: 86.8 m asl

Compiled by: JF

Checked by: MAK

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)	Remarks and Grain Size Distribution (%) GR SA SI CL
86.8	Fill					
0.0	**Fill**, sandy silt, with gravel, brown, compact, dry	1	GRAB			
		2	SS	15	54%	21 50 (29)
85.3 / 1.5	**Sandy silt**, trace gravel (glacial till), with limestone rock fragments, brown, compact to dense, moist	3	SS	14	83%	
		4	SS	43	63%	
		5	SS	50/ 100mm	13%	
83.7 / 3.2	**Limestone bedrock**, shale parting, strong, grey, moderately weathered, closely spaced, thinly bedded, poor to fair quality based on RQD.	6	RC	RQD = 41%		Compresive Strength = 142 MPa; TCR=100%
		7	RC	RQD = 61%		TCR=100%
	Near vertical rock joint between El. 80.1 and 79.7 masl	8	RC	RQD = 63%		TCR=100%
79.7 / 7.1	**End of borehole** Monitoring well installed					

Groundwater: W. L. 81.0 Feb 13, 23

DEPTH: 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0

ELEVATION: 86, 85, 84, 83, 82, 80

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH (kPa): Field. Shear Vane (x) & Sensitivity (s); Pocket Penetrometer ✖; ● Quick Triaxial; O Unconfined; 20 40 60 80

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

WATER CONTENT (%) 10 20 30 40 50 60 70 80 90

Unit Weight (kN/m³) Pocket Penetro. (kPa)

GRAPH NOTES: 30/3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity; O 8=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCO-22-3530**

PROJECT: Geotechnical Investigation - Proposed Building

CLIENT: CSV Architects

PROJECT LOCATION: 1083 and 1095 Merivale Rd, Ottawa, ON

Drilling Date: Jan-11-2023 - Jan-11-2023

BH Location: Lat: 45.377445; Long: -75.732323

Drilling Equipment: CME 750

Drilling Method: Hollow Stem Augers

Remarks:

BH No: 22-6 MW

Datum: Geodetic

Elevation: 85.7 m asl

Compiled by: JF

Checked by: MAK

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)	Remarks and Grain Size Distribution (%) GR SA SI CL
85.7	Fill					
0.0	**Fill**, sandy silt with gravel, brown, compact, dry to moist	1	GRAB			
		2	SS	31	50%	17 47 (35)
83.7		3	SS	18	33%	
2.0	**Sandy silt**, with gravel (glacial till), with limestone rock fragments, brown, very dense, moist to wet	4	SS	50/ 75mm	13%	
83.0 / 2.7	**Limestone bedrock**, shale parting, moderately strong, light grey to grey, very weathered, very closely spaced, thinly beddedthinly bedded, very intensly fractured, very poor quality based on RQD.	5	RC	RQD = 16%		TCR=100%
80.0		6	RC	RQD = 6%		TCR=100%
5.7	**End of borehole** Monitoring well installed					

W. L. 83.4 Feb 13, 23

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH (kPa): Field. Shear Vane (x) & Sensitivity (s); Pocket Penetrometer; Quick Triaxial; Unconfined: 20 40 60 80

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

WATER CONTENT (%): 10 20 30 40 50 60 70 80 90

Unit Weight (kN/m^3) Pocket Penetro. (kPa)

GRAPH NOTES: 30/3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity; ○ 8=3% Strain at Failure

Retrieved Rock Cores

Borehole: BH22-2

RC-5: 84.6 to 84.4 masl (RQD = 44%)

RC-6: 84.4 to 82.8 masl (RQD = 55%)

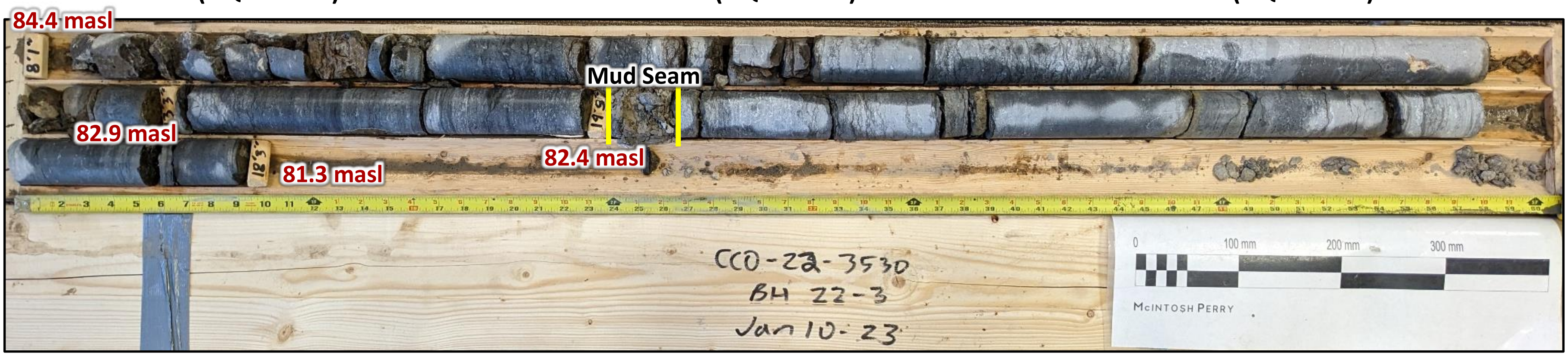
RC-7: 82.8 to 81.3 masl (RQD = 66%)

Borehole: BH22-3

RC-5: 84.4 to 82.9 masl (RQD = 59%)

RC-6: 82.9 to 82.4 masl (RQD = 96%)

RC-7: 82.4 to 81.3 masl (RQD = 67%)



McINTOSH PERRY

1083 – 1095 Merivale Road, Ottawa, Ontario

Project No: CCO-22-3530
Client: CSV Architects

Retrieved Rock Cores

Borehole: BH22-5

RC-6: 83.7 to 82.8 masl (RQD = 41%)	RC-7: 82.8 to 81.2 masl (RQD = 61%)	RC-8: 81.2 to 79.7 masl (RQD = 63%)

Borehole: BH22-6

RC-5: 83.0 to 81.6 m asl (RQD = 16%)	RC-6: 81.6 to 80.0 m asl (RQD = 6%)

McINTOSH PERRY	1083 – 1095 Merivale Road, Ottawa, Ontario	Project No: CCO-22-3530 Client: CSV Architects

McINTOSH PERRY

Unconfined Compressive Strength of Intact Rock Cores
ASTM D7012 Method C

Project No.:	CCO-22-3530	**Date Issued:**	February 7,2023
Lab No.:	OL-23001	**Report No.:**	1
Project Name:	1083 & 1085 Merival Road.		

Core No.:	1	**Moisture Condition:**	Dry as received		
Borehole Location:	BH22-2	**RC:**	6	**Depth (ft):**	9'1" - 9'5"
Date Sampled:	Jan 11,2023	**Received:**	Jan 11,2023	**Tested:**	Feb 7,2023
Core No.:	2	**Moisture Condition:**	Dry as received		
Borehole Location:	BH22-5	**RC:**	5	**Depth (ft):**	10'7" - 11'2"
Date Sampled:	Jan 11,2023	**Received:**	Jan 11,2023	**Tested:**	Feb 7,2023
Core No.:	3	**Moisture Condition:**	Dry as received		
Borehole Location:		**RC:**		**Depth (ft):**	
Date Sampled:		**Received:**		**Tested:**	

Core No. :	1	2	3
Diameter (mm)	47.3	47.2	
Thickness/Height (mm)	105.7	112.0	
Density (Kg/m^3)	2694	2692	
Compressive Strength (Mpa)	141.3	142.2	
Mass of Core (g)	500.6	526.4	
Description of Failure	Type 4	Type 2	

Remarks: Core #1: Diagonal fracture with no cracking through ends. Some columnar vertical cracking on Side.

Core #2: @ approximatly 104.8Mpa chips fell off the side of core. Well-formed cone on one end with vertical cracks running through caps.

Reviewed By: ____________ **Date:** Feb 7,2023

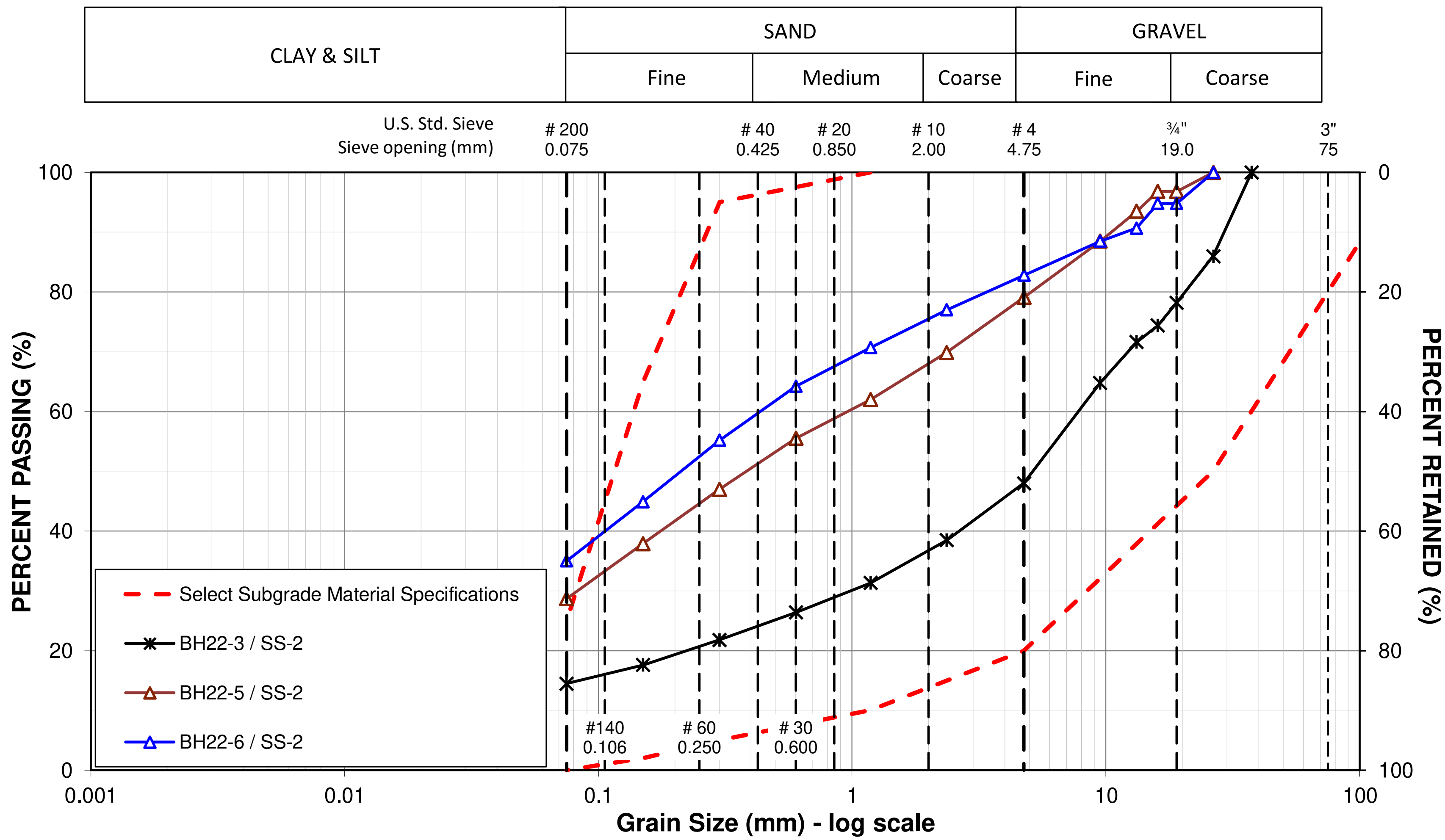
Jason Hopwood-Jones
Laboratory Manager

1083 - 1095 MERIVALE ROAD - MULTI-STOREY BUILDING
OTTAWA, ONTARIO

APPENDIX D
LAB RESULTS

McINTOSH PERRY

Unified Soil Classification System (USCS)



McINTOSH PERRY	Grain-Size Distribution Curve	Figure No. 3
	Fill	
	1083-1095 Merivale Road - Multi-storey Building, Ottawa, ON	CCO-22-3530

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Certificate of Analysis

McIntosh Perry Consulting Eng. (Nepean)

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

Client PO: CCO 22-3530
Project: 1083 Merivale Rd.
Custody: 137319

Report Date: 23-Jan-2023
Order Date: 12-Jan-2023

Order #: 2302517

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

Paracel ID	Client ID
2302517-01	Borehole 22-2 SS-2
2302517-02	Borehole 22-2 SS-3
2302517-03	Borehole 22-5 SS-3
2302517-04	Borehole 22-5 SS-4

Approved By:

Dale Robertson, BSc
Laboratory Director

Certificate of Analysis
Client: **McIntosh Perry Consulting Eng. (Nepean)**
Client PO: CCO 22-3530

Report Date: 23-Jan-2023
Order Date: 12-Jan-2023
Project Description: 1083 Merivale Rd.

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	17-Jan-23	17-Jan-23
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	16-Jan-23	16-Jan-23
Resistivity	EPA 120.1 - probe, water extraction	16-Jan-23	16-Jan-23
Solids, %	CWS Tier 1 - Gravimetric	16-Jan-23	16-Jan-23

Certificate of Analysis

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

Client PO: CCO 22-3530

Report Date: 23-Jan-2023

Order Date: 12-Jan-2023

Project Description: 1083 Merivale Rd.

	Client ID:	Borehole 22-2 SS-2	Borehole 22-2 SS-3	Borehole 22-5 SS-3	Borehole 22-5 SS-4
	Sample Date:	10-Jan-23 09:00	10-Jan-23 09:05	10-Jan-23 10:00	10-Jan-23 10:05
	Sample ID:	2302517-01	2302517-02	2302517-03	2302517-04
	MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	90.1	87.9	86.2	90.8
General Inorganics					
pH	0.05 pH Units	7.59	7.54	7.66	7.64
Resistivity	0.10 Ohm.m	30.3	38.0	22.3	46.9
Anions					
Chloride	10 ug/g dry	29	17	112	13
Sulphate	10 ug/g dry	85	87	41	22

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	10	ug/g						
Sulphate	ND	10	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	110	10	ug/g	108			1.0	35	
Sulphate	47.4	10	ug/g	45.6			3.8	35	
General Inorganics									
pH	7.07	0.05	pH Units	7.10			0.4	2.3	
Resistivity	66.9	0.10	Ohm.m	68.8			2.8	20	
Physical Characteristics									
% Solids	96.4	0.1	% by Wt.	96.1			0.3	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	206	10	ug/g	108	97.2	82-118			
Sulphate	146	10	ug/g	45.6	100	80-120			

Qualifier Notes:

Login Qualifiers :

Sample - One or more parameter received past hold time - Redox potential

Applies to samples: Borehole 22-2 SS-2, Borehole 22-2 SS-3, Borehole 22-5 SS-3, Borehole 22-5 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.

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Paracel ID: 2302517

Blvd. 4J8 s.com

Paracel Order Number (Lab Use Only)	Chain Of Custody (Lab Use Only)
2302517	№ 137319

Client Name: McIntosh Perry	Project Ref: 1083 Merivale RD.	Page 1 of 1
Contact Name: Jason Hepwood-Jones	Quote #:	Turnaround Time
Address: 104-215 Menton place, Nepean ON, K2H 9C1	PO #: CCO 22-3530	☐ 1 day ☐ 3 day
	E-mail: J.Hepwood-Jones@mcintoshperry.com	☐ 2 day ☑ Regular
Telephone: 613 453-0751		Date Required:

☐ REG 153/04 ☐ REG 406/19	Other Regulation	Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm/Sanitary Sewer) P (Paint) A (Air) O (Other)
☐ Table 1 ☐ Res/Park ☐ Med/Fine	☐ REG 558 ☐ PWQO	
☐ Table 2 ☐ Ind/Comm ☐ Coarse	☐ CCME ☐ MISA	
☐ Table 3 ☐ Agri/Other	☐ SU - Sani ☐ SU - Storm	
☐ Table ___	Mun:	
For RSC: ☐ Yes ☐ No	☐ Other:	

Required Analysis

#	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Date	Time	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	B (HWS)	Corrosivity Package	Sulphide	Redox Potential		
1	Borehole 22-2 SS-2	S			Jan 10, 23	9:00								✓	✓	✓		
2	Borehole 22-2 SS-3	S			Jan 10, 23	9:05								✓	✓	✓		
3																		
4	Borehole 22-5 SS-3	S			Jan 10, 23	10:00												
5	Borehole 22-5 SS-4	S			Jan 10, 23	10:05								✓	✓	✓		
6																		
7																		
8																		
9																		
10																		

Comments:

Method of Delivery: Walk-in

Relinquished By (Sign): [signature]	Received By Driver/Depot: [signature]	Received at Lab: Sreeparvathy Dhmai	Verified By: [signature]
Relinquished By (Print): Jason H-Jones	Date/Time: 01/12/23 2:36pm	Date/Time: JAN 12, 2023 04.24	Date/Time: Jan 13 2023
Date/Time: Jan 12, 2023 @ 1:00pm	Temperature: 22.1 °C	Temperature: 7.6 °C	pH Verified: ☐ By:

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Subcontracted Analysis

McIntosh Perry Consulting Eng. (Nepean)
215 Menten Place, Unit 104
Nepean, ON K2H 9C1
Attn: Jason Hopwood-Jones

Paracel Report No.	**2302517**	Order Date:	12-Jan-23
Client Project(s):	**1083 Merivale Rd.**	Report Date:	23-Jan-23
Client PO:	**CCO 22-3530**		
Reference:			
CoC Number:	**137319**		

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor's report is attached

Paracel ID	Client ID	Analysis
2302517-01	Borehole 22-2 SS-2	Redox potential, soil Sulphide, solid
2302517-02	Borehole 22-2 SS-3	Redox potential, soil Sulphide, solid
2302517-03	Borehole 22-5 SS-3	Redox potential, soil Sulphide, solid
2302517-04	Borehole 22-5 SS-4	Redox potential, soil Sulphide, solid

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Client:	Dale Robertson	Work Order Number:	488434
Company:	Paracel Laboratories Ltd. - Ottawa	PO #:	
Address:	300-2319 St. Laurent Blvd.	Regulation:	[No Reg - Always Include Reg Report]
	Ottawa, ON, K1G 4J8	Project #:	2302517
Phone/Fax:	(613) 731-9577 / (613) 731-9064	DWS #:	
Email:	drobertson@paracellabs.com	Sampled By:	
Date Order Received:	1/17/2023	Analysis Started:	1/23/2023
Arrival Temperature:	10.3 °C	Analysis Completed:	1/23/2023

WORK ORDER SUMMARY

ANALYSES WERE PERFORMED ON THE FOLLOWING SAMPLES. THE RESULTS RELATE ONLY TO THE ITEMS TESTED.

Sample Description	Lab ID	Matrix	Type	Comments	Date Collected	Time Collected
Borehole 22-2 SS- 2	1843100	Soil	None		1/10/2023	9:00 AM
Borehole 22-2 SS- 3	1843101	Soil	None		1/10/2023	9:05 AM
Borehole 22-5 SS- 3	1843102	Soil	None		1/10/2023	10:00 AM
Borehole 22-5 SS- 4	1843103	Soil	None		1/10/2023	10:05 AM

METHODS AND INSTRUMENTATION

THE FOLLOWING METHODS WERE USED FOR YOUR SAMPLE(S):

Method	Lab	Description	Reference
RedOx - Soil (T06)	Mississauga	Determination of RedOx Potential of Soil	Modified from APHA-2580B

REPORT COMMENTS

Samples received past hold time for Redox, proceed with analysis as per comments/client notes TJ 01/17/23

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 488434

This report has been approved by:

Marc Creighton
Laboratory Director

6820 Kitimat Road Unit 4, Mississauga, ON, L5N 5M3
Phone: (905) 821-1112   Fax: (905) 821-2095   Web: www.testmark.ca

TESTMARK Laboratories Ltd.
Committed to Quality and Service

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd. - Ottawa

Work Order Number: 488434

WORK ORDER RESULTS

Sample Description	Borehole 22 - 2 SS - 2		Borehole 22 - 2 SS - 3		Borehole 22 - 5 SS - 3		Borehole 22 - 5 SS - 4			
Sample Date	1/10/2023 9:00 AM		1/10/2023 9:05 AM		1/10/2023 10:00 AM		1/10/2023 10:05 AM			
Lab ID	1843100		1843101		1843102		1843103			
General Chemistry	Result	MDL	Result	MDL	Result	MDL	Result	MDL	Units	Criteria: [No Reg - Always Include Reg Report]
RedOx (vs. S.H.E.)	424	N/A	434	N/A	410	N/A	438	N/A	mV	~

LEGEND

Dates: Dates are formatted as mm/dd/year throughout this report.

MDL: Method detection limit or minimum reporting limit.

~: In a criteria column indicates the criteria is not applicable for the parameter row.

Quality Control: All associated Quality Control data is available on request.

Field Data: Reports containing Field Parameters represent data that has been collected and provided by the client. Testmark is not responsible for the validity of this data which may be used in subsequent calculations.

Sample Condition Deviations: A noted sample condition deviation may affect the validity of the result. Results apply to the sample(s) as received.

Reproduction of Report: Report shall not be reproduced, except in full, without the approval of Testmark Laboratories Ltd.

ICPMS Dustfall Insoluble: The ICPMS Dustfall Insoluble Portion method analyzes only the particulate matter from the Dustfall Sampler which is retained on the analysis filter during the Dustfall method.

Regulation Comparisons: Disclaimer: Please note that regulation criteria are provided for comparative purposes, however the onus on ensuring the validity of this comparison rests with the client.

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

09-February-2023

Paracel Laboratories
Attn : Dale Robertson

300-2319 St.Laurent Blvd.
Ottawa, ON
K1G 4K6, Canada

Phone: 613-731-9577
Fax:613-731-9064

Date Rec. : 17 January 2023
LR Report: **CA12496-JAN23**
Reference: Project#: 2302517

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide (Na2CO3) %
1: Analysis Start Date		09-Feb-23
2: Analysis Start Time		14:18
3: Analysis Completed Date		09-Feb-23
4: Analysis Completed Time		15:44
5: QC - Blank		< 0.04
6: QC - STD % Recovery		118%
7: QC - DUP % RPD		ND
8: RL		0.02
9: Borehole 22-2 SS-2	10-Jan-23 09:00	0.04
10: Borehole 22-2 SS-3	10-Jan-23 09:05	< 0.04
11: Borehole 22-5 SS-3	10-Jan-23 10:00	< 0.04
12: Borehole 22-5 SS-4	10-Jan-23 10:05	< 0.04

RL - SGS Reporting Limit
ND - Not Detected

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

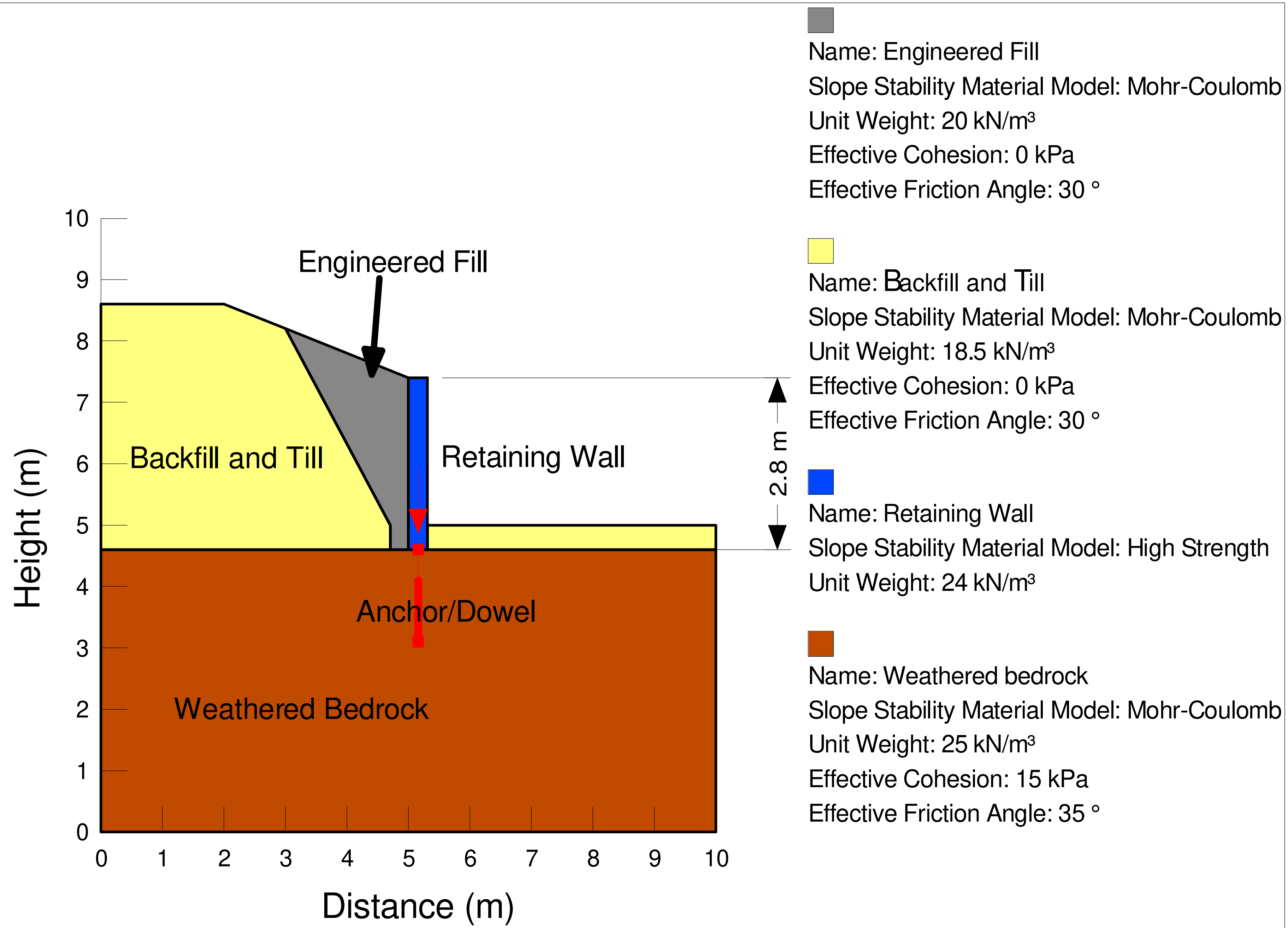
OnLine LIMS

0003225590

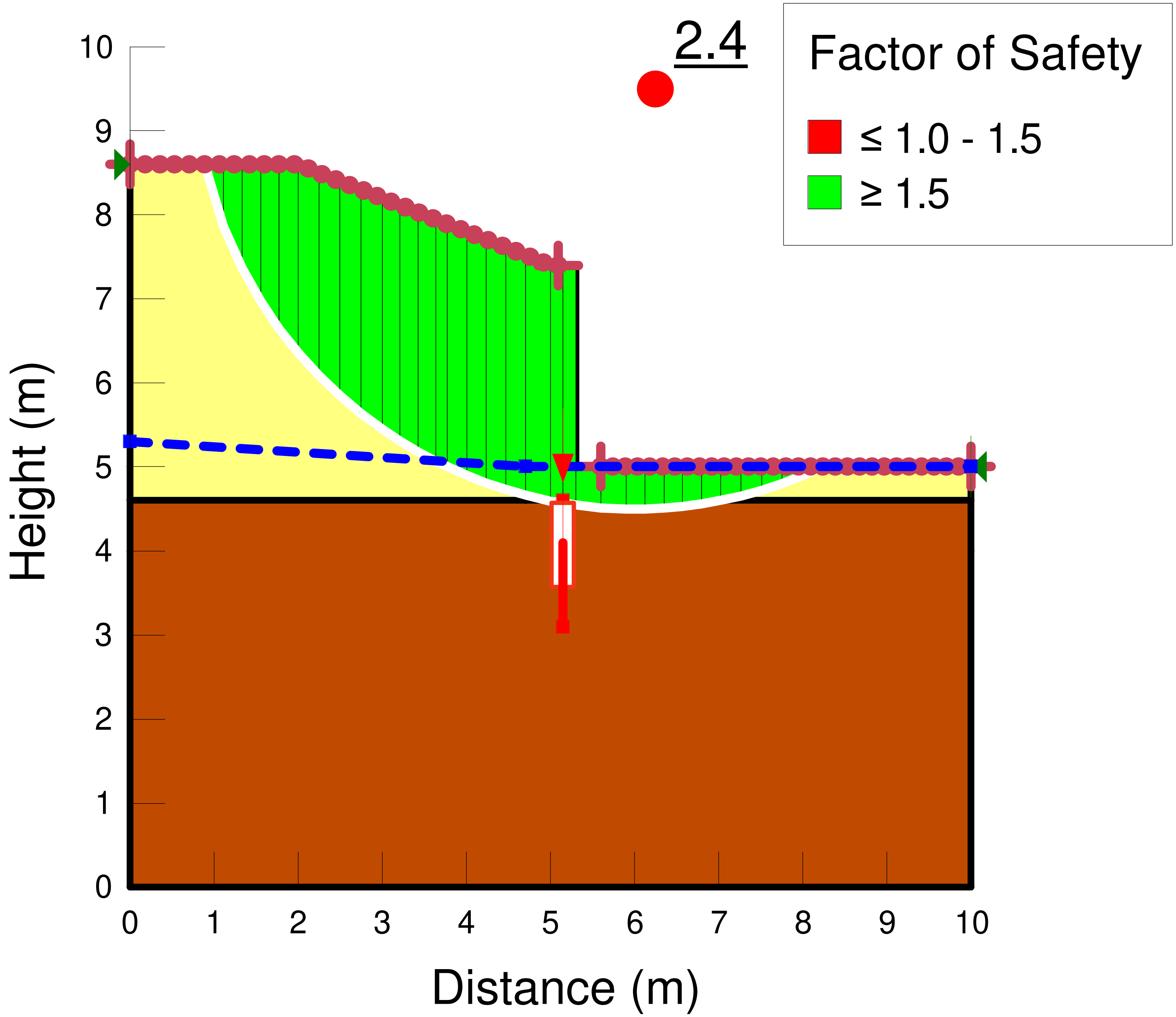
1083 - 1095 MERIVALE ROAD - MULTI-STOREY BUILDING
OTTAWA, ONTARIO

APPENDIX E
GLOBAL STABILITY ANALYSIS

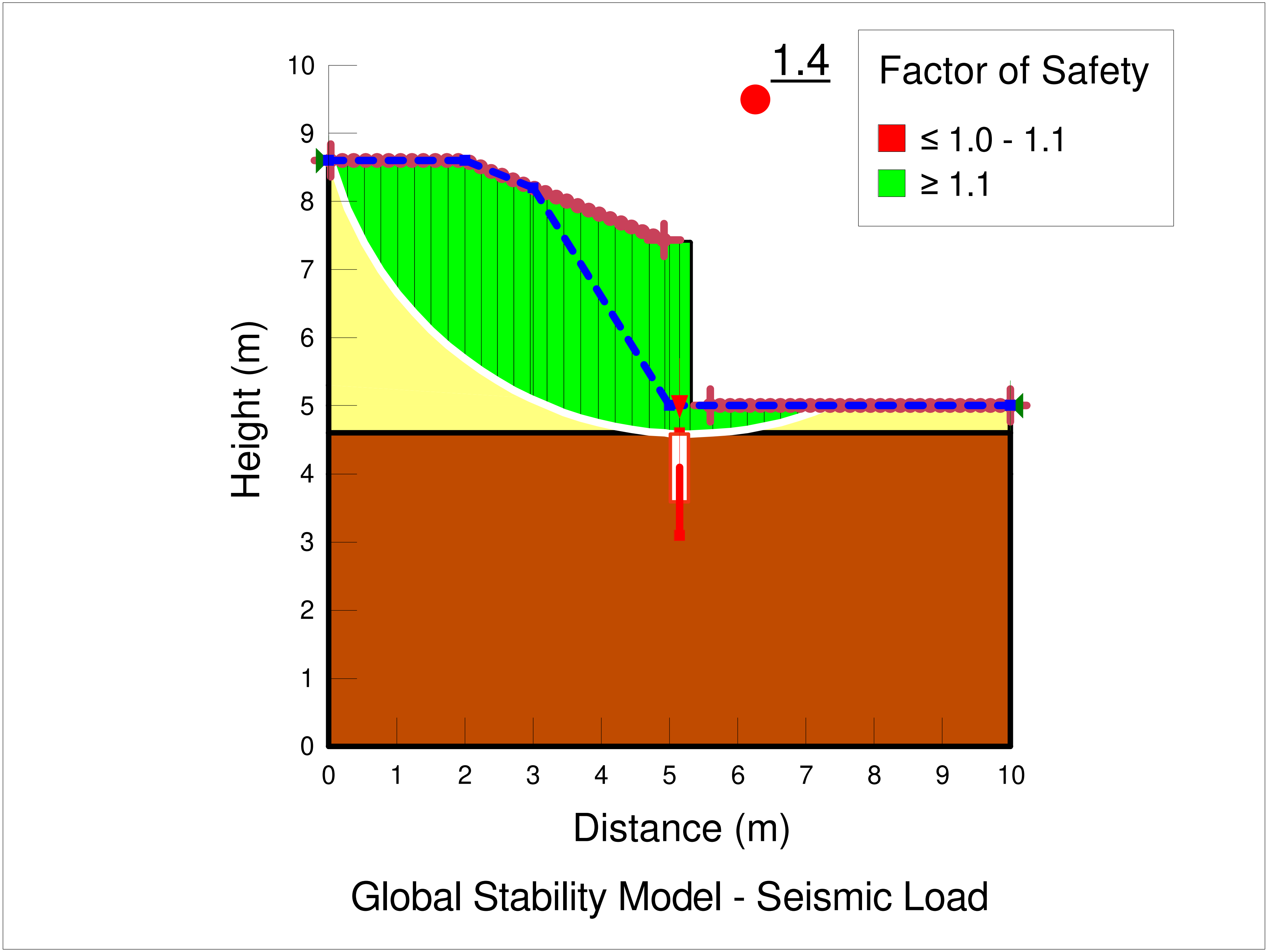
McINTOSH PERRY



Global Stability Model



Global Stability Model - Static Load



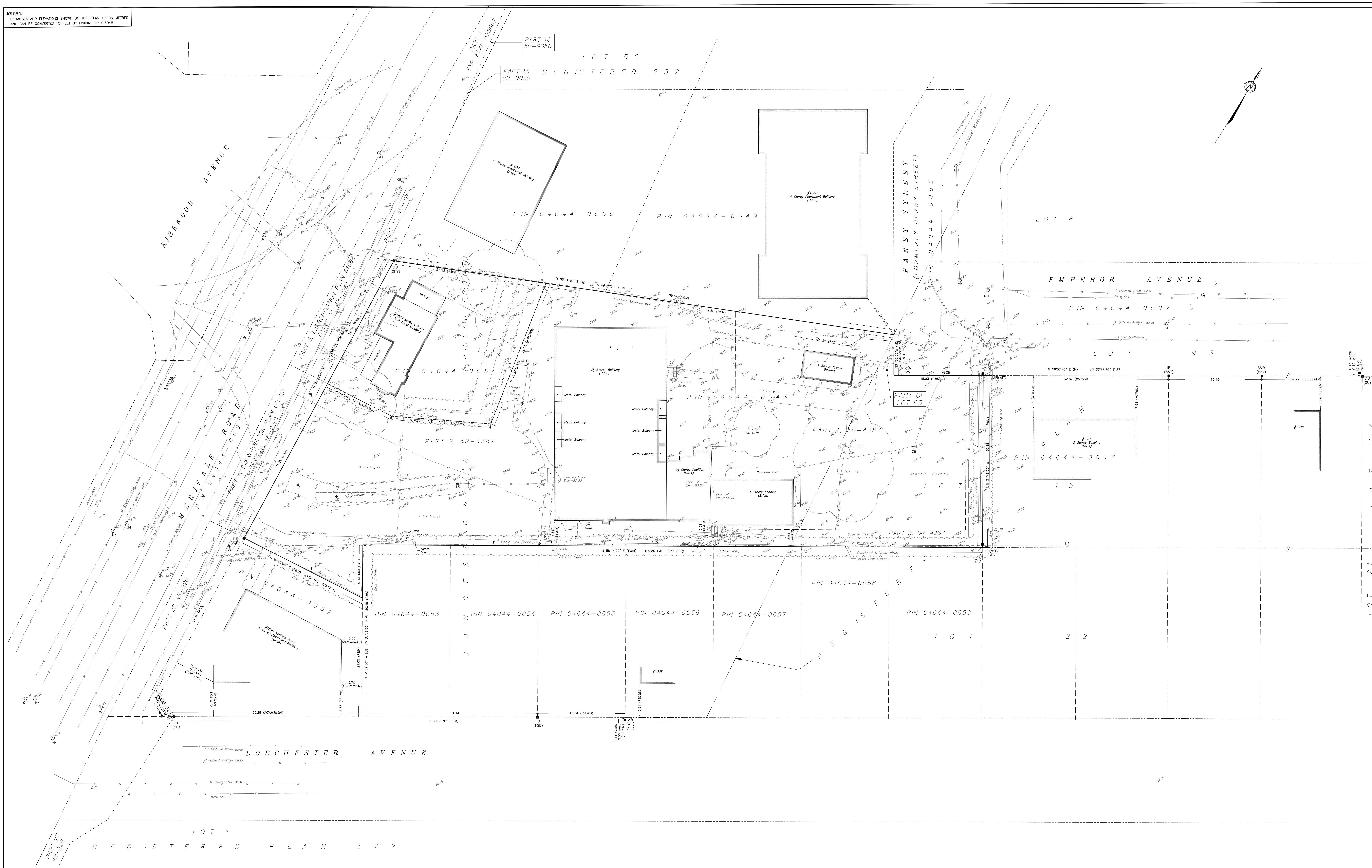
1.4
Factor of Safety
≤ 1.0 - 1.1
≥ 1.1
Height (m)
10
9
8
7
6
5
4
3
2
1
0
0
1
2
3
4
5
6
7
8
9
10
Distance (m)
Global Stability Model - Seismic Load

1083 - 1095 MERIVALE ROAD - MULTI-STOREY BUILDING
OTTAWA, ONTARIO

APPENDIX F
SURVEY AND DRAWINGS

McINTOSH PERRY

METRIC
DISTANCES AND ELEVATIONS SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048



TOPOGRAPHIC SURVEY OF
PART OF LOTS 15 AND 93 REGISTERED PLAN 294 AND PART OF LOT "L" CONCESSION 'A' (RIDEAU FRONT) CITY OF OTTAWA

SCALE 1 : 200

0 5 10 20 metres

FAIRHALL, MOFFATT & WOODLAND LIMITED
ONTARIO LAND SURVEYORS

ELEVATION NOTES

1. ELEVATIONS SHOWN HEREON ARE REFERRED TO GEODETIC DATUM (CVGD28).
2. ELEVATIONS FOR MANHOLE COVERS AND CATCH BASINS HAVE TO BE INDEPENDENTLY CONFIRMED BEFORE THEY CAN BE ACCEPTED FOR FINAL DESIGN OR CONSTRUCTION PURPOSES.
3. IT IS THE RESPONSIBILITY OF THE USER OF THIS INFORMATION TO VERIFY THAT THE JOB BENCHMARKS HAVE NOT BEEN ALTERED OR DISTURBED AND THAT THEIR RELATIVE ELEVATION AND DESCRIPTION AGREE WITH THE INFORMATION SHOWN ON THIS DRAWING.

UTILITY NOTES

1. THIS DRAWING CANNOT BE ACCEPTED AS ACKNOWLEDGING ALL UNDERGROUND UTILITIES AND IT WILL BE THE RESPONSIBILITY OF THE USER TO CONTACT THE RESPECTIVE UTILITY AUTHORITIES FOR CONFIRMATION OR LOCATION.
2. UNDERGROUND UTILITIES, AS REPORTED ON THIS DRAWING, ARE BASED ON AN ACTUAL 'FIELD LOCATE' BY THE RESPECTIVE UTILITY AGENCIES OR HAVE BEEN COMPILED FROM DATA OBTAINED FROM THE FOLLOWING SOURCES:
   a) CITY OF OTTAWA PUBLIC UTILITIES REGISTRY
   b) UNDERGROUND UTILITY LOCATORS USL-1
3. BEFORE ANY WORK INVOLVING PROBING, EXCAVATING, ETC., A FIELD LOCATION OF UNDERGROUND PLANT BY THE PERTINENT UTILITY AUTHORITY IS MANDATORY.

NOTES

BEARINGS ARE ASTRONOMIC AND ARE REFERRED TO THE EASTERLY LIMIT OF PARTS 28, 29 AND 30 AS SHOWN ON PLAN 4R-226 HAVING A BEARING OF N 03° 39' 00" W.

LEGEND

■ – SURVEY MONUMENT FOUND
IB – IRON BAR
SIB – STANDARD IRON BAR
SSIB – SHORT STANDARD IRON BAR
CC – CUT CROSS
Ø – ROUND
(P) – PLAN 5R-4387
(M) – MEASURED
(S) – SET
DIA. – DIAMETER
PIN – PROPERTY IDENTIFIER NUMBER
(SU) – SOURCE UNKNOWN
(W) – UTILITY WIRES ELEVATION
(T) – TOP OF RETAINING WALL
(WIT) – WITNESS
CLF – CHAIN LINK FENCE
(857) – FAIRHALL, MOFFATT & WOODLAND LIMITED, O.L.S., (REF 70-294 NP)
(FSD) – FARLEY, SMITH & DENIS SURVEYING LTD., O.L.S. (REF. 75-01, 127-11)
(WJW) – W.J. WEBSTER, O.L.S. (REF. 4-294, 9-294)
(647) – FARLEY & MARTIN LTD., O.L.S. (PLAN OF #1030 PANET STREET, DATED OCTOBER 05, 1973)
(AOV) – ANNIS, O'SULLIVAN, VOLLEBEKK LTD., O.L.S., (PLAN OF #1099 MERIVALE ROAD, DATED FEBRUARY 13, 1975) (PLAN OF #1083 MERIVALE ROAD, DATED JANUARY 17, 1984)
(CITY) – CITY OF OTTAWA
(JGP) – J.G.PAYETTE O.L.S., (PLAN OF #1095 MERIVALE ROAD, DATED SEPTEMBER 07, 1982)
CB – CATCHBASIN
WV – WATER VALVE
UP – UTILITY POLE
– GUY WIRE AND ANCHOR
FH – FIRE HYDRANT
MH – MANHOLE
– SIGN
– DECIDUOUS TREE
—S— – SANITARY SEWER
—ST— – STORM SEWER
—W— – WATERMAIN
—G— – GAS LINE
—T— – TRAFFIC
—B— – BELL
—UH— – HYDRO
– OVERHEAD UTILITY WIRES
—C— – ATRIA (FIBER OPTICS)
—SL— – STREET LIGHTING

SURVEYOR'S CERTIFICATE

I CERTIFY THAT:
1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT, THE LAND TITLES ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED ON AUGUST 09, 2021.

DATE

JOHN H. GUTRI
ONTARIO LAND SURVEYOR

ASSOCIATION OF ONTARIO LAND SURVEYORS
PLAN SUBMISSION FORM
2173721

THIS PLAN IS NOT VALID UNLESS IT IS AN EMBOSSED ORIGINAL COPY ISSUED BY THE SURVEYOR
In accordance with Regulation 1026. Section 29 (3).

Fairhall Moffatt & Woodland LIMITED
ONTARIO LAND SURVEYORS
OTTAWA
Surveying and Land Information Services
100-600 TERRY FOX DRIVE, KANATA, ONTARIO K2L 4B6
TEL: (613) 591-2580 FAX: (613) 591-1495
www.fmw.on.ca

JOB No. AB22300
E 364918, N 5026655
REFERENCE No. 92-A(RF)NP
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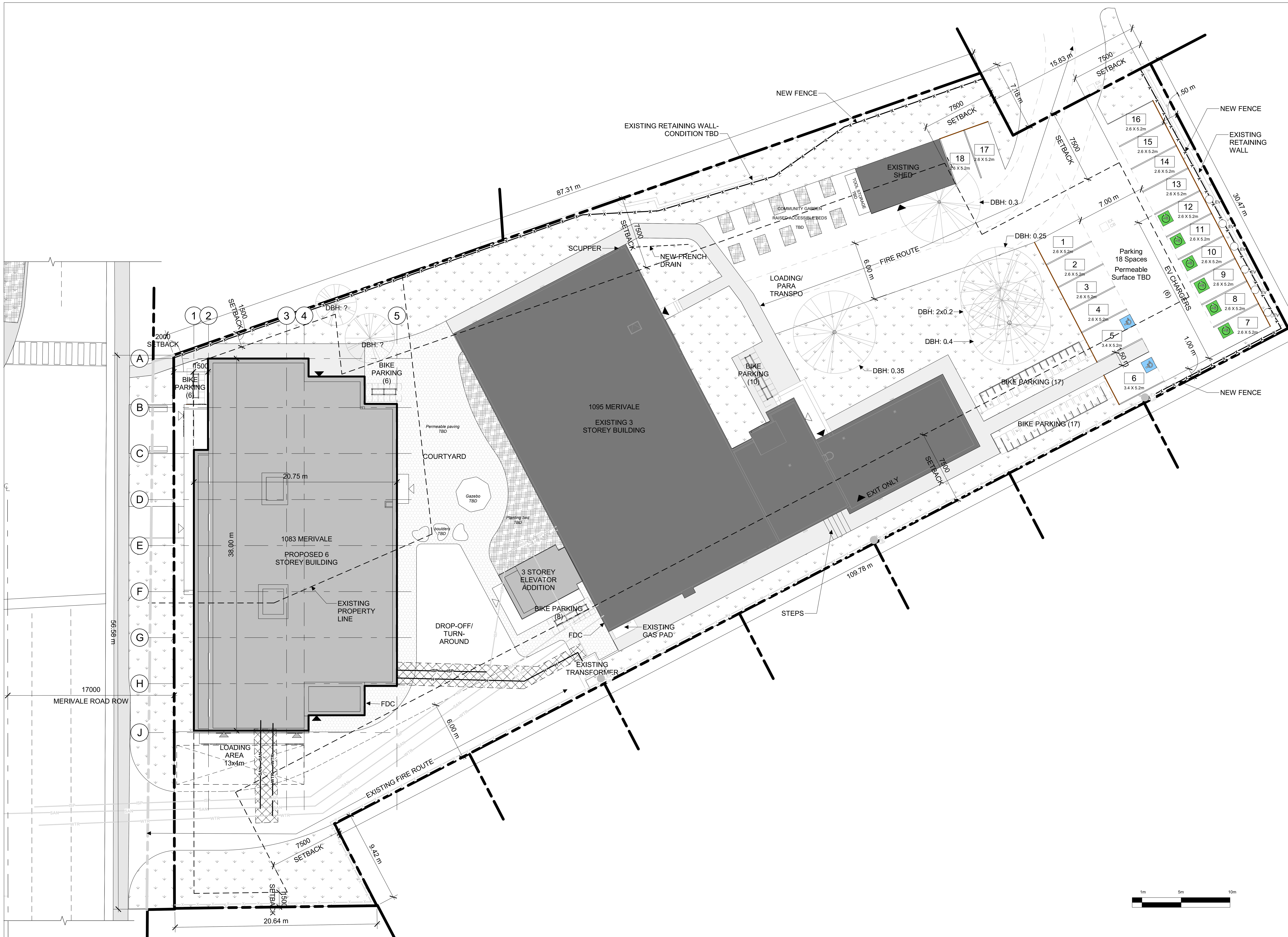

1 SPC_SITE
A.100 1 : 200

SITE PLAN GENERAL NOTES:

1. ALL GENERAL SITE INFORMATION AND CONDITIONS COMPILED FROM EXISTING PLANS AND SURVEYS
2. DO NOT SCALE THIS DRAWING
3. REPORT ANY DISCREPANCIES PRIOR TO COMMENCING WORK. NO RESPONSIBILITY IS BORN BY THE CONSULTANT FOR UNKNOWN SUBSURFACE CONDITIONS
4. CONTRACTOR TO CHECK AND VERIFY ALL DIMENSIONS ON SITE AND REPORT ANY ERRORS AND/OR OMISSIONS TO THE CONSULTANT
5. REINSTATE ALL AREAS AND ITEMS DAMAGED AS A RESULT OF CONSTRUCTION ACTIVITIES TO THE SATISFACTION OF THE CONSULTANT
6. CONTRACTOR TO LAYOUT PLANTING BEDS, PATHWAYS ETC. TO APPROVAL OF CONSULTANT PRIOR TO ANY JOB EXCAVATION
7. THE ACCURACY OF THE POSITION OF UTILITIES IS NOT GUARANTEED - CONTRACTOR TO VERIFY PRIOR TO EXCAVATION
8. INDIVIDUAL UTILITY COMPANY MUST BE CONTACTED FOR CONFIRMATION OF UTILITY EXISTENCE AND LOCATION PRIOR TO DIGGING
9. ALL DISTURBED AREAS TO BE RESTORED TO ORIGINAL CONDITION OR BETTER UNLESS OTHERWISE NOTED

SITE PLAN KEYNOTES:

- # SAMPLE TEXT
- # SAMPLE TEXT
- # SAMPLE TEXT MULTI LINE
- # SAMPLE TEXT

SITE PLAN LEGEND:

- EXISTING ELEMENT
- ASPHALT PAVING
- GRASS
- CONCRETE SIDEWALK
- CONCRETE PAD
- MULCH/PLANTING
- GRAVEL/RIVERSTONE/MAINTENANCE STRIP
- STONE DUST/SAND
- PAVER TYPE 1
- PAVER TYPE 2
- EXISTING MATERIAL 1
- EXISTING MATERIAL 2
- EMERGENCY EXIT
- SERVICE DOORS
- BUILDING MAIN ENTRANCE
- PROPERTY LINE
- FENCE PER LANDSCAPE
- DOMESTIC WATER
- SANITARY
- STORM
- ELECTRICAL SERVICE (BELOW GRADE)
- GAS
- INTERNET SERVICE PROVIDER
- CATCH BASIN
- LIGHT STANDARD
- FIRE HYDRANT
- MANHOLE
- UTILITY POLE
- EV CHARGING STATION
- SIAMESE CONNECTION
- FDC FIRE DEPARTMENT CONNECTION
- DROPPED CURB
- TREE
- SHRUB

LEGAL DESCRIPTION

REFERENCE SURVEY

MUNICIPAL ADDRESS 1083 MERIVALE ROAD, OTTAWA, ON K1Z 6A9

SITE AREA	______m²
BUILDING AREA	726 m²
GROSS FLOOR AREA	______m²
BUILDING HEIGHT	______m 6 STOREYS
ZONE	____________
SCHEDULE 1:	AREA ______m²
SCHEDULE 2:	AREA ______m²

ZONING PROVISION	REQUIRED	PROVIDED
MIN. LOT WIDTH		
MIN. LOT AREA		
MIN. FRONT YARD SETBACK		
MIN. CORNER YARD SETBACK		
MIN. REAR YARD SETBACK		
MIN. INTERIOR YARD SETBACK		
MAX. HEIGHT		
AMENITY AREA		
LANDSCAPED AREA		

PARKING QUEING + LOADING	REQUIRED	PROVIDED
RESIDENTIAL SPACES 1083	9	9
RESIDENTIAL SPACES 1095	7	7
OFFICE SPACES 1083	2	2
VISITOR SPACES	0	0
ACCESSIBLE PARKING	2	2
BICYCLE PARKING	62	64
REFUSE COLLECTION	16 cu yd	16 cu yd
GARBAGE COLLECTION	8cu yd	8 cu yd

CSV ARCHITECTS
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STRUCTURAL ENGINEER
name
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MECHANICAL ENGINEER
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ELECTRICAL ENGINEER
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phone
email

CIVIL ENGINEER
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address
address
phone
email

LANDSCAPE ARCHITECT
name
address
address
phone
email

STAMP

REV DATE ISSUE

NOTES

3. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL OTHER PROJECT DRAWINGS AND SPECIFICATIONS.
4. DO NOT SCALE DRAWINGS. CONTRACTOR SHALL BE RESPONSIBLE TO VERIFY DIMENSIONS ON SITE.
5. ALL WORK SHALL BE IN ACCORDANCE WITH THE ONTARIO BUILDING CODE AND ALL SUPPLEMENTS AND APPLICABLE MUNICIPAL REGULATIONS.

CLIENT

SHEPHERDS OF GOOD HOPE
OTTAWA
ONTARIO, CANADA

PROJECT

SGH 1083 MERIVALE

1083 Merivale Road
Ottawa, ON K1Z 6A9

TITLE

SITE PLAN

PROJECT NO: 2021-0111
DRAWN:
APPROVED:
SCALE: 1 : 200
DATE PRINTED: 2022-11-10 11:38:50 AM

REV DRAWING NO.

A.100