2830 CARLING AVE. AND 810 VICK AVE. GEOTECHNICAL REPORT

Project No.: CCO-21-1191

Prepared for:

1408505 Ontario Inc.
Attention: Mr. Robert A. Wilson
88 David Drive
Ottawa, Ontario, K2G 2N5

Holzman Consultants Inc.
311 Richmond Road | Suite 203
Ottawa, ON K1Z 6X3

Prepared by:

McIntosh Perry
104-215 Menten Place
Ottawa, ON K2H 9C1

November 2020

McINTOSH PERRY

TABLE OF CONTENTS

APPENDICES

GEOTECHNICAL INVESTIGATION and
FOUNDATION DESIGN and RECOMMENDATION REPORT
2830 Carling Ave. and 810 Vick Ave. Ottawa, Ontario

1.0 INTRODUCTION

This report presents the factual findings obtained from a geotechnical investigation performed at the above-mentioned site for the proposed three-storey apartment building with basement and underground parking levels. The fieldwork was carried out on September 14, 2020, to September 16, 2020, and comprised of three boreholes to a maximum depth of 15.9 m.

The purpose of the investigation was to explore the subsurface conditions at this site and to provide borehole location plans, a record of borehole logs, and laboratory test results. This report provides anticipated geotechnical conditions influencing the design and construction of the proposed three-storey apartment building above one level of basement above parking level. The report also includes recommendations for foundation design. Recommendations are offered based on the authors' interpretation of the subsurface investigation and test results. The readers are referred to Appendix A, Limitations of Report, which has an integral part of this document.

The investigation was performed at the request of the 1408505 Ontario Inc. and Holzman Consultants Inc.

2.0 SITE DESCRIPTION

The site is located in a residential street. Currently, the site is occupied by two existing bungalows, and both lots are heavily vegetated. The site access along Carling Avenue is bounded by trees, a wood picket fence, and a concrete curb. On Vick Ave., the site is accessible through the front yard and driveway. The front yard has three fully grown maple trees. On judge Street, the site is accessible through a narrow driveway as per the latest site plan provided, and as shown on Figure 2, in Appendix B.

Properties and borehole locations are shown in Figure 2, in Appendix B. To the north, the site is bounded by Carling Ave. and to the south side by townhouse buildings. To the east and west sides, the site is bounded by Vick Ave. and Judge Street. Multi-storey residential buildings are located across the site on both Vick Ave. and Judge Street.

3.0 PROJECT UNDERSTANDING

It is understood that the proposed building is a three-storey apartment building, with basement and a parking level. Due to natural slopes of the intersecting streets, the proposed basement level will be lower than the entrance level from Vick Avenue and higher than the Carling Avenue level. The basement and three residential floors are proposed wood frame structure. Below the basement there is one level of parking structure which is proposed as a concrete structure. The proposed building is to be constructed on a relatively flat landscape. Apartment units will be walk-up, and there is no need for an elevator pit in the foundation system. The underground parking entrance and exit access will be on Judge street.

4.0 FIELD PROCEDURES

The staff of McIntosh Perry Consulting Engineers (McIntosh Perry) visited the site before the drilling investigation to mark out the proposed borehole locations to obtain utility clearance to identify the location of underground infrastructures. Utility clearance was carried out by Underground Service Locators (USL-1) on behalf of McIntosh Perry. Public and private utility authorities were informed, and all utility clearance documents were obtained before the commencement of drilling work.

The equipment used for drilling was owned and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario. Boreholes were advanced using hollow stem augers aided by remote-controlled multi-power drill rig in BH20-1 and 20-2 and hollow stem auger and core washing aided by track-mounted CME 750 drill rig in BH20-3. Boreholes were advanced to a maximum depth of 15.9 m (El. 65.3 m) below the ground level. Soil samples were obtained at 0.75 m intervals in boreholes up to 12.8 m (EL. 68.7 m) and below this level and due to the uniformity of the soil profile, only one sample collected at 15.3 m (El. 65.9 m) in BH20-3. The samples were collected using a 51 mm outside diameter split spoon sampler following the Standard Penetration Test (SPT) procedure. Boreholes were backfilled with auger cuttings and bentonite holeplug and restored to the original surface. Borehole locations are shown in Figure 2, included in Appendix B.

5.0 IDENTIFICATION AND TEST PROCEDURES

All samples were logged as retrieved, and visual description and soil type identification were added to the logs. Subsequently, soil descriptions were confirmed by additional tactile examination of the soils in the laboratory. Laboratory grain-size distribution analysis on representative SPT samples was performed at McIntosh Perry geotechnical lab in accordance with the American Society for Testing Materials (ASTM) test procedures.

Paracel Laboratories Ltd., in Ottawa, carried out chemical tests on three representative soil samples to determine the soil corrosivity characteristics.

Test procedures are listed below;

ASTM C136 – Sieve Analysis of Fine and Coarse Aggregates (LS-602)
LS-702 – Determination of Particle Size Analysis of Soils
ASTM D1586 – Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

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

6.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

6.1 Site Geology

Based on published physiography maps of the area (Ontario Geological Survey), the site is located within the Ottawa Valley Clay Plains. Surficial geology maps of southern Ontario indicate the site is underlain by alluvial deposits. The alluvial deposits in this region are predominantly clay, silt, sand, gravel, may contain organic remains.

6.2 Subsurface Conditions

In general, the site stratigraphy consists of three layers of shallow topsoil/fill, followed by a thick deposit of sand with different portions of silt and gravel. A till layer of different portions of sand and gravel with some silt is interbedded the sand layer. The sand layer extends to the maximum depth of investigation in BH 20-3. Based on published well records data, the bedrock is expected to be at a deeper depth at approximately 22 m below the existing ground surface. For classification purposes, the soils encountered at this site can be divided into three major zones.

a) Topsoil/Fill
b) Sand
c) Till

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

6.2.1 Topsoil/Fill

A thin layer of topsoil was encountered in BH 20-1 only since both BH 20-2 and 20-3 were drilled on the paved driveways. Between borehole 20-1, 20-2, and 20-3, the topsoil layer was observed to be black to brown to silty sand to sand, with traces of clay and a presence of organic matter. The topsoil is followed by a fill layer in all boreholes. The thickness of the fill layer varies between boreholes and ranges between 0.6 m to 2.1 m. The fill is composed of various portions of sand and gravel with a trace of silt and was observed to be brown, dry to damp, and loose to compact. The SPT 'N' value ranges from 5 to 8 blows/300mm.

6.2.2 Sand

Underlying the fill, was a thick layer of sand with different portions of silt and traces of gravel, observed to be light brown, dry to damp, and compact to very dense. The SPT 'N' value ranges from 10 to 77 blows/300mm. Drilling advanced in all boreholes using hollow stem augers. Due to the nature and compactness of the sand, the drilling resumed with core washing in BH20-1 and 20-3. The maximum depth that was reached with core washing in BH20-1 was 11.4m and in BH20-3 was 15.9m, after which it was very difficult to advance and drilling was terminated.

Five samples underwent grain size analysis testing, and the layer was observed to contain, on average, 6.0% gravel, 87% sand, 12% silt and clay. The sand layer extends to the maximum depth of investigation in BH20-3. A till layer is interbedded the sand between El. 73 and El. 69 m. A summary of the grain size distribution for this layer is shown in Table 6-1. Test results are shown in Figures 4 and 5, included in Appendix B.

Table 6-1: Grain Size Distribution of the Sand Layer

Grain Size	Range (%)
Gravel	0.7 – 12
Sand	80 – 94
Fines	5.8 – 18.3

6.2.3 Till: Sand and Gravel with some Silt

A till layer composes of sand and gravel, some silt with presence of cobbles and boulders was encountered in BH20-3 between El. 73 and El. 69 m. The till was observed to light brown, damp to moist, and very dense, with SPT 'N' values ranging from 63 to 107 blows/300mm. One representative sample underwent grain size analysis testing, and the layer was observed to contain 39% gravel, 47% sand, 14% silt and clay. A summary of the grain size distribution for this layer in BH20-1 is shown in Table 6-2.

Table 6-2: Grain Size Distribution of the Silty Sand Layer in BH20-1

Grain Size	(%)
Gravel	39
Sand	47
Fines	14

6.3 Groundwater

No groundwater was observed in open boreholes and all boreholes were dry. Two monitoring wells were installed in borehole BH20-3 at 9.0m depth (El. ~ 72.0m) and 4.6m depth (El. ~ 76.5m), and their assembly is shown on the borehole log. The groundwater table was monitored in both wells on October 30, 2020. Both boreholes were dry. However, the groundwater level may be expected to fluctuate due to seasonal changes.

Table 6-3: Monitoring Well Information and Groundwater Level Readings

Borehole	Monitoring well depth/Elevation (m)	Monitoring Date	Surface El. (m)	Groundwater Depth (m)	Water Table El. (m)
BH20-3	4.6m/ El. ~ 76.5m	2020-10-30	81.1	--	--
BH20-3	9.0m/ El. ~ 72.0m	2020-10-30	81.1	--	--

It is noteworthy that the ground surface elevation of the site is approximately 20 m higher than the water level of the Ottawa River which is approximately 700 m away. Given the coarse material encountered on site and the anticipated high hydraulic conductivity of the sand, the groundwater elevation at this site is expected to be near the river level.

6.4 Chemical Analysis

The chemical test results conducted by Paracel Laboratories in Ottawa, Ontario, to determine the resistivity, pH, sulphate and chloride content of three representative soil samples are shown in Table 6-4 below. Chemical test results are included in Appendix D and summarized in below table.

Table 6-4: Soil Chemical Analysis Results

Borehole	Sample	Depth / El. (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (Ohm-m)
BH20-2	SS-05	3.0 ~ 3.7	7.96	0.0034	0.0066	49.7
BH20-3	SS-13	9.1 ~ 9.7	8.19	0.0026	0.0028	71.9
BH20-3	SS-17B	12.6 ~ 12.8	8.28	0.0019	0.0053	63.5

7.0 DISCUSSIONS AND RECOMMENDATIONS

7.1 General

This section of the report provides engineering recommendations on the geotechnical design aspect of the project based on the project requirements and our interpretation of the subsurface soil information. The recommendations presented herein are subject to the limitations noted in Appendix A "Limitations of Report" which forms an integral part of this document.

The foundation engineering recommendations presented in this section have been developed following Part 4 of the 2015 National Building Code of Canada (NBCC) and 2012 Ontario Building Code (OBC) extending the Limit State Design approach.

7.2 Overview

It is understood that the proposed apartment building is a three (3) storey timber structure with a basement and a concrete structure underground parking level. It is also understood that the finished floor elevation for the underground parking level of the proposed development will be approximately at ~ 78.5 m.

For the current project, the following list summarizes some key geotechnical facts that were considered in the suggested geotechnical recommendations:

- The expected foundation loads for the four (4) levels of wood frame and underground concrete garage can be supported on the underlying sand layer by shallow foundation system.
- The proposed structure can be designed using a seismic Site Class D provided that the boundary zones of the shear walls and all column loads are extended to and supported on the dense to very dense sand layer by spread footings.
- Excavation for foundations, and underground parking and basement levels will be advanced below the existing ground level through the sand deposit. In addition, the footprint of the proposed development is adjacent to occupied residential buildings on the south and roadway at other three sides. Shoring system, such as soldier piles, or sheet piles with tieback anchors will be required during excavation and construction stages.
- The surface and groundwater inflow to the excavation can be handled by pumping from well-filtered sumps established on the floor of the excavation. The actual inflow into the excavation will depend on many factors including, but not limited to, the contractor's schedule, the rate of excavation, the size of the excavation, and the time of the year at which the excavation is to occur. Based on the encountered stratigraphy and the amount of groundwater intake, application for PTTW is not required. If more

precise information on potential groundwater seepage is needed, a separate permeability test can be carried in the existing monitoring well as part of a separate scope of work.

7.3 Site Preparation

The sand deposit can exhibit collapsing behavior upon excavation. To protect the adjacent buildings and roads, a shoring system is required during both excavation and construction stages to protect the adjacent properties and utilities.

Shoring system installation requires to keep minimal vibration level to avoid soil disturbance and associated unexpected settlement to the nearby buildings, streets, and utilities. Also, the noise level should be kept at a tolerance level of noise as per the City of Ottawa requirements. Vibration and deformation monitoring will be required throughout shoring installation, excavation, construction, and backfilling.

Soldier piles and wood lagging is the preferred option. Sheet pile installation is also feasible. Once excavation is completed the walers can be removed as the wall is backfilled. The upper waler at the south, bordering the adjacent residential building, shall not be removed and soldier piles and wood lagging shall remain in place at the south side.

The design of the tieback anchor system is the responsibility of the contractor. The contractor shall hire a professional engineer to provide a detailed design for the anchors. The expected ultimate bond stress for gravity-grouted soil anchor is 80 kPa and pressure-grouted soil anchor is 100 kPa.

The following recommendation can be used to determine the anchor load. The minimum diameter of the drilled hole shall be 100mm. Casing shall be driven to increase the normal stress and friction. The drill hole should be cleaned and the grout should be placed as quickly as possible after the hole has been drilled.

7.4 Foundation Excavation

The expected underground parking level will be at about an elevation of 78.5 m. Excavation for the construction of the basement, underground parking, and foundation will proceed through the native sand deposit. Excavating of overburden soil shall be performed using conventional hydraulic excavating equipment. The Occupational Health and Safety Act (OHSA) of Ontario indicated for soils that when is dry, it may run easily into well-defined conical pile could be classified as Type 3 soil and sloped no steeper than 1V:3H. The south side of excavation cannot be sloped and has to be retained.

At the time of the investigation, the groundwater level in the proximity of the area of the proposed development was measured in two monitoring wells installed in BH20-3. The readings were taken 45 days after installation to allow the groundwater table to come to equilibrium and stabilize in the well. No groundwater was observed in both monitoring wells.

7.5 Foundations

In general, the subsurface conditions in the area of the proposed low-rise building consists of a thick layer of sand that is interrupted by a gravelly silty sand layer (till). The expected depth of the bedrock based on the well records in the area is ~ 21.3m (70') (El. ~ 59.8m) from the existing ground surface.

It is understood that the finished underground parking floor elevation for the new building is proposed to be at 78.5, and the underside of the foundations will likely be at an elevation of 78 to 77.5 m. Based on these elevations, it appears that spread footings will be bearing on the compact to dense sand layer. However, due to natural slopes of the intersecting streets, a maximum of 1.0 m grade raise with engineered fill may be required to accommodate for any level difference. Granular A conforming to OPSS 1010 compacted to 100% Standard Proctor Maximum Dry Density (SPMDD) shall be used for grade raise below the foundation level.

The SPT field test results, 'N' values within the expected depth and influence zone (twice of the footing width) of a spread footing range between 10 to 69 blows/300mm. The sand layer can be classified according to the Canadian Foundation Engineering Manual (CFEM) (2006) as compacted to very dense sand. The estimated average angle of internal friction (ϕ') within the stress influence zone below the footing is approximately 32°. The sand layer is a competent layer and can provide suitable support to the expected loads from the structure.

7.5.1 Shallow Foundations

For shallow spread footings, the overburden soil below the columns and foundation walls can be excavated to the level of founding. The subgrade shall be proof rolled before constructing the spread footings.

7.5.1.1 Bearing Resistance

Due to the presence of a competent sand layer, shallow footings with a minimum of 1.5 m in a shorter dimension bearing on the sand may be considered to support the structural loads of the proposed development if recommended bearing capacities are adequate.

The mechanical properties of the sand layer were derived from SPT field test. The average value of SPT 'N' blows for 2B distance below the foundation level was used to estimate the effective soil friction angle, ϕ'. The ϕ'-value and the horizontal soil-footing interface friction angle, δ' are given in Table 8-2. Load and Resistance Factor Design (LRFD) approach following the National Building Code of Canada (NBCC) (2015) recommendations were used to determine the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) geotechnical resistances. For ULS conditions, the unfactored ULS bearing capacity of the spread footing was determined using the general bearing capacity formula as per the CFEM (2006) using the effective soil friction angle, ϕ' value in Table 7-2. A geotechnical resistance factor of 0.5 as per the NBCC recommendations can be used to obtain the factored ULS bearing resistance. Furthermore, For SLS bearing capacity, allowable bearing capacity based on SPT test results and 25 mm settlement was determined.

The bearing capacity of footings is also a function of the soil surcharge above the footing. Footings shall not be designed for any elevation above those noted in the bearing capacity table.

Geotechnical surface resistance values at the founding level (bearing capacities) are provided for Ultimate Limit State (ULS) and Serviceability Limit State (SLS). Bearing capacities are listed in the below table;

Table 7-1: Factored ULS and SLS Bearing Resistance

Footing Type	Max. El. (m)	Min. Soil Cover (m)	Min dim. (m)	ULS (kPa)	SLS (kPa)
square footing	78.0	0.75	1.5	350	200
Strip footing	78.0	0.75	1.5	300	200

Table 7-2: Unfactored Shearing Parameters for the Sand and Till based on SPT 'N' values

Soil Layer	ϕ' [§]		δ' [*]
	Hatanaka and Uchida (1996)	Schmertmann (1975)	
Sand	33°	32°	24°
Till	35°	35°	26°

[§] ϕ': Effective Soil Friction Angle

[*] δ': Horizontal Soil-Footing Interface Friction Angle ($\delta' = 0.75\phi'$)

7.5.1.2 Frost Protection

Based on the freezing index for the Southern Ontario Region provided for this site, the frost penetration depth is expected at 1.8 m below the ground surface. Frost penetration depth is estimated based on the OPSD 3090.101, Foundation Frost Penetration Depths for Southern Ontario.

The encountered native sand is classified as low frost susceptibility material based on provincial guidelines.

All perimeter and exterior foundation elements or interior foundation elements in unheated areas should be provided with a minimum of 1.8 meters of earth cover for frost protection purposes. Frost protection depth can be reduced to 1.5 m for those buildings constantly heated during the cold season.

7.6 Seismic Site Classification

Seismic site classification is completed based on NBCC (2015) and OBC (2012) Section 4.1.8.4 and Table 4.1.8.4.A. This classification system is based on the average soil properties in the upper 30 m and accounts for site-specific shear wave velocity, standard penetration resistance, and plasticity parameters of cohesive soils.

Selected spectral responses in the general vicinity of the site for 2% chance of exceedance in 50 years (2500 years return period) are as indicated in Table 7-3, shown below and in Appendix E;

Table 7-3: Selected Seismic Spectral Responses (2% in 50 Yrs) – NRCan 2010

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.630	0.305	0.136	0.046	0.322

Based on the subsurface condition and field and SPT values, the site can be classified as Seismic Site Class (D).

7.6.1 Liquefaction Potential

Soil stratigraphy for the site consists of a thick sand deposit that extends to the maximum depth of investigation which is approximately 16.0 m below the existing ground level in BH20-3. The native sand layer is interbedded by a relatively thick dense till layer that is approximately 4.25m thick.

Herein liquefaction susceptibility of the native sand was evaluated. The native sand and till were found non-susceptible to liquefaction. The results of the analysis are presented in Appendix E.

7.7 Engineered Fill

Footings shall be installed on native soil. Any over excavation shall be leveled by engineered fill. Granular A conforming to OPSS 1010 compacted to 100% Standard Proctor Maximum Dry Density (SPMDD) shall be used to level any over excavation below the foundation level.

If grade raise with engineered fill is required to accommodate for any level difference, Granular A conforming to OPSS 1010 compacted to 100% Standard Proctor Maximum Dry Density (SPMDD) shall be used for grade raise below the foundation level. The proposed engineered fill, beyond footings influence zone, can be any material conforming to granular criteria as outlined in OPSS 1010. Material conforming to 'Granular' criteria are considered free draining and compactable and can be utilized as the engineered fill. This can apply to the backfill beyond foundation walls and engineered fill in between the footings. The engineered fill shall be compacted to a minimum of 98% SPMDD.

All fill should be placed in horizontal lifts of uniform thickness of no more than 300 mm before compaction at appropriate moisture content determined by the Proctor test. The requirement for fill material and compaction may be addressed with a note on the structural drawing for foundation or grading drawing, and with a Non-Standard Special Provision (NSSP). Any topsoil, organics, or loose sand should be removed before placing engineered fill material.

7.8 Slabs-on-Grade

Slab-on-grades are considered free-floating (not attached to the foundation walls) and should be supported on a minimum of 200 mm of Granular A bedding compacted to 100% SPMDD. The requirements of the fill underneath slab-on-grade is noted in section 7.7 Engineered Fill.

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

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

For the design of the slab-on-grade, the modulus of subgrade reaction (k) is required. Modulus of subgrade reaction is a multi-function complex correlation that varies with the subgrade material, grade-raise fill material, and the flexural stiffness of the structural slab. However, simplified assumptions were made to estimate the spring modulus for slab-on-grade on compacted Granular A. To estimate the modulus of subgrade reaction, it was assumed that a 2 m square section of the concrete slab-on-grade under the applied loads. Since the modulus of subgrade reaction is needed for the ultimate failure design of the slab, it is assumed the failure can occur at a 25 mm deformation. Considering these assumptions, a subgrade reaction modulus of 20,000 $kN/m^2/m$ can be used for the design of the interior slab-on-grade. This k-value is only valid for the construction of slab-on-grade on compacted Granular A bedding. This value shall not be used for the native subgrade.

It is understood the parking slab and the entry and exit ramps will be all concrete and there are no asphalt paved areas as part of this project.

7.9 Lateral Earth Pressure

Free draining material should be used as backfill material for foundation walls. If proper drainage is provided, "at rest" condition may be assumed for calculation of earth pressure on foundation walls. The following parameters are recommended for the granular backfill.

Table 7-4: Lateral Pressure parameters for Granular A and B and Horizontal Backfill

Pressure Parameter		Expected Value			
		Granular A	Granular B	Other OPSS1010 'Granular'	Native Sand
Unit Weight (γ) kN/m^3	Above groundwater	22.5	21.7	21.7	18.0
	Below groundwater	12.7	11.9	11.9	8.19
Angle of Internal Friction (ϕ)		35°	32°	31°	32°
Coefficient of Active Earth Pressure (k_a)		0.27	0.31	0.32	0.31
Coefficient of Passive Earth Pressure (k_p)		3.69	3.23	3.12	3.23
Coefficient of Earth Pressure at Rest (k_o)		0.43	0.47	0.48	0.47

7.10 Sidewalks and Hard Surfacing

The width and extent of the sidewalks will be defined as per the architectural drawings. The designer shall provision adequate slope, based on applicable codes, to provide appropriate runoff discharge. Expansion, construction, and dummy joints shall be spaced as required by the applicable standards. Sidewalks can be categorized under residential/commercial use, and therefore, the concrete sidewalks should have a thickness of 150 to 200 mm. Requirements of OPSD 310.010 'Concrete Sidewalk', OPSD 310.020 'Concrete Sidewalks Adjacent to Curb and Gutter' and OPSD 310.030 'Concrete Sidewalk Ramps at intersection' are recommended for the construction of the concrete sidewalk. A minimum of 150 mm bedding of OPSS Granular A compacted to 100% SPMDD is required for the concrete sidewalk panels.

All proposed new curbs shall be constructed as per applicable standards. It is recommended to follow City of Ottawa detail provided in SC3, Concrete Curb, and Sidewalk as a minimum requirement. All curbs shall receive a minimum of 150 mm Granular A bedding on approved subgrade free from soft, loose, and organic material.

7.11 Cement Type and Corrosion Potential

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

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

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

8.0 CONSTRUCTION CONSIDERATIONS

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

As noted in the previous sections, all grade adjustments due to over-excavation, within the shallow footings influence zone, shall be done using OPSS Granular A.

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

Foundation walls should be backfilled with free-draining material with granular material conforming to OPSS 1010 Granular criteria. However, the native soil can provide drainage if it is proposed to be used for any portion of the design with no compaction requirement.

A geotechnical engineer or technician should attend the site to confirm the native subgrade, type of fill material, and level of compaction. All bearing surfaces should be inspected by experienced geotechnical personnel prior to placing the footings to ensure the excavated subgrade it as the reported and recommended condition.

Vibration monitoring should be carried out during excavation and construction phases to ensure that the vibration levels at the existing surrounding structures and utilities are maintained below tolerable levels.

9.0 GROUNDWATER SEEPAGE

The groundwater was not observed in both monitoring wells in BH20-03 and is expected to be at greater depth. However, depending on the construction season, surface runoff can seep into the excavation due to high hydraulic permeability of the native sand and groundwater may present above the depth of excavation. Hydraulic conductivity value of the native sand is expected approximately 1x10E-3. This 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 value can be used for the selection of the pump capacity for dewatering. The excavated subgrade must be kept dry at all times to minimize the disturbance of the subgrade. If excavation proceeds below the groundwater level, the water level shall be lowered to a minimum of 1 m below the proposed bottom of excavation before excavation and compaction. Groundwater elevation is expected to fluctuate seasonally. Any surface water infiltrating into the open excavation can be removed through conventional sump and pump methods. The subgrade shall be kept dry at all times, especially before compaction and proof rolling.

Under the new regulations (O.Reg 63/16 and O.Reg 387/04), a Permit to Take Water (PTTW) is required from the Ministry of the Environment, Conservation and Parks (MOECP) if a volume of water greater than 400,000 liters per day is pumped from the excavation under normal operation, but more than 50,000 liters per day, the water taking will not require a PTTW, but will need to be registered in the EASR as a prescribed activity. Since the excavations will likely be above the groundwater level, it is considered unlikely that a PTTW would be required. The site designer shall decide on the permit application based on the expected excavation volume.

The design of the dewatering system should be the responsibility of the contractor. An outlet(s) should be identified, which the contractor can use to dispose of the pumped groundwater and incident precipitation. In order for pumped groundwater to be discharged to a City sewer, the groundwater quality needs to meet the City of Ottawa Sewer Use By-law limits, and a separate sewer discharge permit or City approval is required.

10.0 SITE SERVICES

At the subject site, the burial depth of water-bearing utility lines is typically 2.4 m below the ground surface. 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.

Excavation will proceed through the topsoil, and native sand. Excavating of overburden soil shall be performed using conventional hydraulic excavating equipment. Cobbles or boulders larger than 300 mm in diameter, if encountered, should be removed from the side slopes for worker safety.

The Occupational Health and Safety Act (OHSA) of Ontario indicated that side slopes in the sand above the water table could be classified as Type 3 soil and below the water table as Type 4 soil and sloped no steeper than 3H:1V or be shored. If space restrictions exist, the excavations can be carried out within trench boxes, which is fully braced to resist lateral earth pressure.

Due to the potential for long term settlement of topsoil and organic materials and the effects of this settlement on service lines sensitive to level change, the existing topsoil, and organic materials are not considered suitable for the support of site services. Utilities should be supported on a minimum of 150 mm bedding of Granular A compacted to a minimum of 98% of SPMDD. Utility cover can be Granular A or Granular B type II compacted to 96% SPMDD. All covers are to be compacted to 100% SPMDD if they are intersecting structural elements. The engineer designing utilities shall ensure the proposed utility pipes can tolerate compaction loads.

To extend the life of buried utilities, it is recommended utility bedding and backfill to be separated from the native soil by filter geotextile.

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
Nov. 16/21
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
N. AZIZI TAVAKKOLI
100150911
Nov 16/21
PROVINCE OF ONTARIO

Mohammed Al-Khazaali, Ph.D., P.Eng.
Geotechnical Engineer

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

REFERENCES

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

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

3) Google Earth, Google, 2015.

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

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

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

7) MTO – Pavement Design and Rehabilitation Manual

8) Natural Resources Canada – Seismic Hazard Calculator

2830 CARLING AVE. AND 810 VICK AVE.

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.

2830 CARLING AVE. AND 810 VICK AVE.

APPENDIX B
FIGURES

McINTOSH PERRY

800 400 0 800

Scale 1:25,000 Metres

LEGEND

Site Location
Watercourse
Major Road
Waterbody
Local Road
Wooded Area

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2020.

CLIENT: HOLZMAN CONSULTANTS INC.

PROJECT: GEOTECHNICAL INVESTIGATION
2830 CARLING AVENUE AND 810 VICK AVENUE

TITLE: SITE LOCATION

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

PROJECT NO:CCO-21-1191		FIGURE:
Date	Oct., 02, 2020	1
GIS	EU	
Checked By	MA	

C:\Users\e.ungun\McIntosh Perry\GIS - Documents\Projects\2021\CCO\CCO-21-1191 First Bay Properties Inc 2380 Carling\GIS\aprx\Geotech\CCO-21-1191-Geotech.aprx

LEGEND

Borehole Location

REFERENCE

GIS data provided by the Ontario Ministry of Natural Resources and Forestry, 2020.

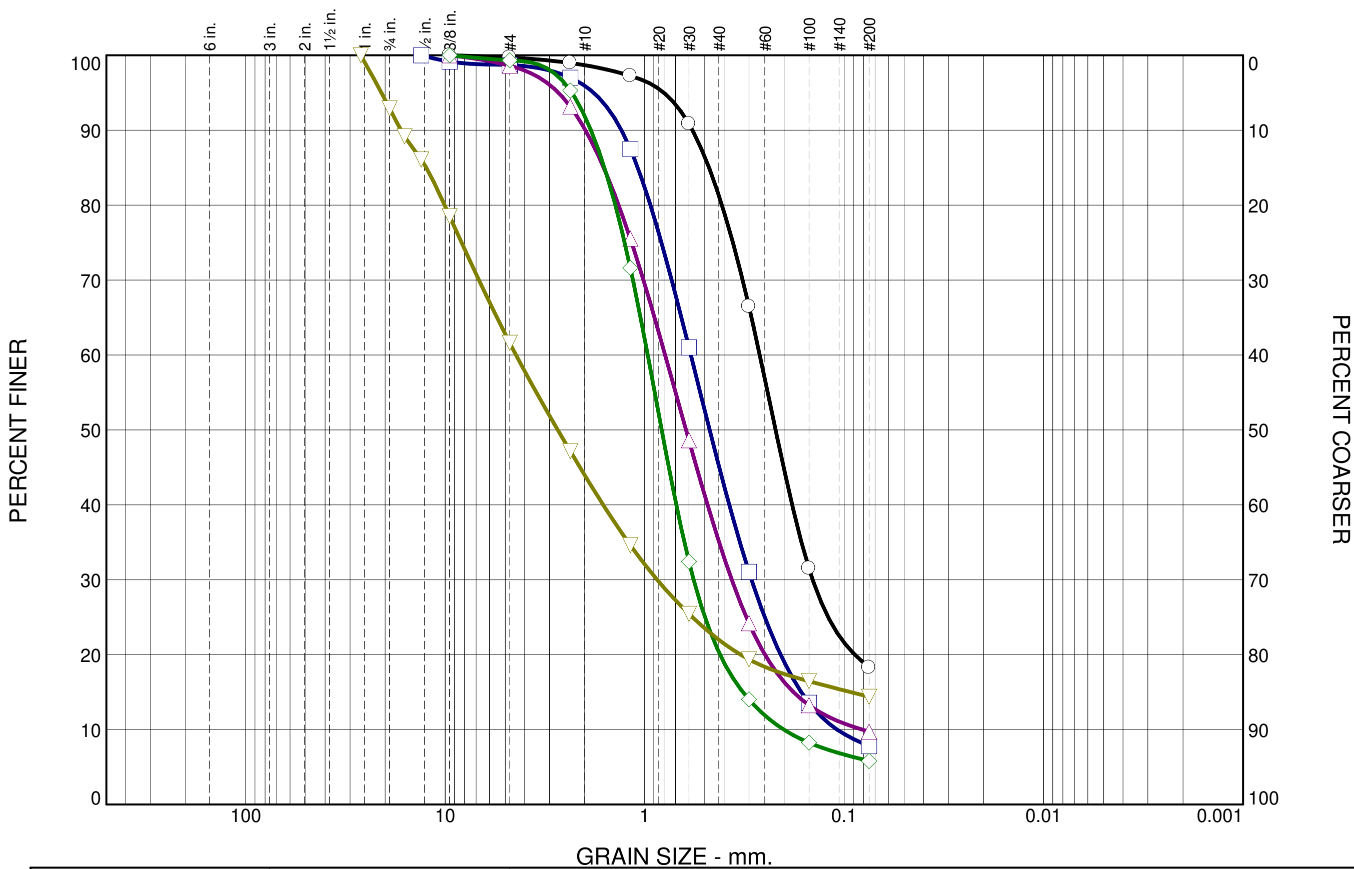
CLIENT:	HOLZMAN CONSULTANTS INC.
PROJECT:	GEOTECHNICAL INVESTIGATION 2830 CARLING AVENUE AND 810 VICK AVENUE
TITLE:	BOREHOLE LOCATIONS

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

PROJECT NO:CCO-21-1191		FIGURE:
Date	Nov., 04, 2020	2
GIS	EU	
Checked By	MA	

C:\Users\e.ungun\McIntosh Perry\GIS - Documents\Projects\2021\CCO\CCO-21-1191 First Bay Properties Inc 2380 Carling\GIS\aprx\Geotech\CCO-21-1191-Geotech.aprx

Particle Size Distribution Report



	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.3	1.0	17.4	63.0	18.3	
□	0.0	0.0	1.3	2.7	50.7	37.5	7.8	
△	0.0	0.0	1.4	8.4	55.0	25.5	9.7	
◇	0.0	0.0	0.7	7.4	71.5	14.6	5.8	
▽	0.0	7.0	31.4	17.6	22.0	7.6	14.4	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	Carling&Vick	BH2001SS06	12.5-14.5'	Sand some Silt	
□	Carling&Vick	BH2001SS10	22.5-24.5'	Sand trace Silt trace Gravel	
△	Carling&Vick	BH2002SS12	25.0-27.0'	Sand trace Silt trace Gravel	
◇	Carling&Vick	BH2003SS09	20.0-22.0'	Sand trace Silt	
▽	Carling&Vick	BH2003SS16	37.5-39.5'	Sand and Gravel some Silt	

McINTOSH PERRY

Client: First Bay Properties Inc.

Project: Geotech Investigation - Proposed Low Rise Building, 2830 Carling Ave. & 810 Vick Ave.

Project No.: CCO-211191

Figure 3

Checked By: H.Smith

Particle Size Distribution Report

	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	7.5	4.5	1.8	28.2	49.5	8.5	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	Carling&Vick	BH2003SS18	50.0-52.0'	Sand some Gravel trace Silt	

McINTOSH PERRY

Client: First Bay Properties Inc.
Project: Geotech Investigation - Proposed Low Rise Building, 2830 Carling Ave. & 810 Vick Ave.
Project No.: CCO-211191 **Figure 4**

These results are for the exclusive use of the client for whom they were obtained.

Checked By: H.Smith

2830 CARLING AVE. AND 810 VICK AVE.

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

BOREHOLE No BH20-1

DATE:	14/09/2020 - 15/09/2020	**LOCATION:**	810 Vick Ave. and 2830 Carling Ave.	**ORIGINATED BY:**	M.A.
PROJECT:	CCO-21-1191	**COORDINATES:**	Lat: 45.358033986 , Lon: -75.79720097	**COMPILED BY:**	M.A.
CLIENT:	Holzman Consultant Inc.	**DATUM:**	Geodetic	**CHECKED BY:**	N.T.
ELEVATION:	82.19 m	**REMARK:**		**REPORT DATE:**	08/10/2020

Column headings: DEPTH - feet; DEPTH - meters; SOIL PROFILE (ELEVATION - m / DEPTH - m; DESCRIPTION); SYMBOL; SAMPLES (TYPE AND NUMBER; STATE; RECOVERY; "N" or RQD); GROUNDWATER CONDITIONS; DYNAMIC CONE PEN. RESISTANCE PLOT (20 40 60 80); SHEAR STRENGTH (kPa) (Vane test: ◇ Intact, ◆ Remolded; Lab vane: □ Intact, ■ Remolded; 20 40 60 80 100); WATER CONTENT and LIMITS (%) (W_P W W_L; 25 50 75); REMARKS & GRAIN SIZE DISTRIBUTION (%) (G S M C)

Elevation (m) / Depth (m)	Description	Sample	Recovery	"N" or RQD	Remarks & Grain Size (G S M C)
82.2	Natural ground surface				
82.2 / 0.0	Topsoil: Silty sand, traces of clay, black to brown, dry to damp. Presence of organic matter.	SS-01A	100	6	
82.0 / 0.2	Fill : Silty sand, some gravel, brown, damp, loose to compact.	SS-01B	55	6	Auger Grinding
81.4 / 0.8	Fill : Silty sand, brown, damp, loose to compact.	SS-02	71	7	Auger Grinding
80.7 / 1.5	Sand, traces of silt, light brown, damp, compact to dense.	SS-03	58	18	
		SS-04	67	15	Uniform, fine to medium grain-size Sand
		SS-05	67	11	
78.5 / 3.7	Sand, light brown, damp, dense to very dense.	SS-06	79	30	1 81 18
		SS-07	83	53	Uniform, fine to medium grain-size Sand to the end of borehole
		SS-08	83	69	
76.1 / 6.1	Sand, traces of silt and gravel, light brown, damp, very dense.	SS-09	75	58	
		SS-10	79	54	2 91 7
		SS-11	75	46	
		SS-12	79	46	
		SS-13	83	46	

Depth scale: feet 5, 10, 15, 20, 25, 30; meters 1–9.

\\LICENSES7\Sobek\Geotec80\Style\Log_Borehole_v5.sty

McINTOSH PERRY

BOREHOLE No BH20-1

DATE:	14/09/2020 - 15/09/2020	**LOCATION:**	810 Vick Ave. and 2830 Carling Ave.	**ORIGINATED BY:**	M.A.
PROJECT:	CCO-21-1191	**COORDINATES:**	Lat: 45.358033986 , Lon: -75.79720097	**COMPILED BY:**	M.A.
CLIENT:	Holzman Consultant Inc.	**DATUM:**	Geodetic	**CHECKED BY:**	N.T.
ELEVATION:	82.19 m	**REMARK:**		**REPORT DATE:**	08/10/2020

DEPTH - feet	DEPTH - meters	ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	STATE	RECOVERY	"N" or RQD	GROUNDWATER CONDITIONS	DYNAMIC CONE PEN. RESISTANCE PLOT (20 40 60 80) / SHEAR STRENGTH (kPa) Vane test: ◇ Intact, ◆ Remolded; Lab vane: □ Intact, ■ Remolded (20 40 60 80 100)	WATER CONTENT and LIMITS (%) W_P W W_L (25 50 75)	REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C
		72.4 / 9.8	Sand, some gravel, light brown, damp, very dense.		SS-13		83	46				Drilling was terminated at 10.67m. Core washing with NQ casing was started after pulling out the augeres. The core washing advnced to 11.43 m and was terminated due to the rig capacity and significant resistance from the sand.
	10				SS-14		83	50				
35	11				SS-15		79	48				
		70.8 / 11.4	**End of Borehole** **No water was observed in open borehole.**									
40	12											
	13											
45	14											
	15											
50	16											
55	17											
	18											
60												

\\LICENSES7\Sobek\Geotec80\Style\Log_Borehole_v5.sty

McINTOSH PERRY

BOREHOLE No BH20-2

DATE: 15/09/2020 - 15/09/2020
PROJECT: CCO-21-1191
CLIENT: Holzman Consultant Inc.
ELEVATION: 82.17 m

LOCATION: 810 Vick Ave. and 2830 Carling Ave.
COORDINATES: Lat: 45.358130872 , Lon: -75.797292716
DATUM: Geodetic
REMARK:

ORIGINATED BY: M.A.
COMPILED BY: M.A.
CHECKED BY: N.T.
REPORT DATE: 08/10/2020

ELEVATION - m / DEPTH - m	DESCRIPTION	TYPE AND NUMBER	RECOVERY	"N" or RQD	REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C
82.2	Natural ground surface				
0.0	Asphalt.				
82.1 / 0.1	Fill : Silty sand and gravel, brown, dry to damp.	GS-01			
81.4 / 0.8	Fill : Sand, some silt, traces of gravel, light brown, damp, loose.	SS-02	63	5	
80.8 / 1.4	Fill : Silty sand, traces of gravel, brown, damp, loose. Presence of organic matter.	SS-03	67	6	
80.0 / 2.1	Sand, traces of silt and gravel, brown, loose. Presence of organic matter.	SS-04	79	5	
79.3 / 2.9	Sand, light brown, damp to moist, compact.	SS-05	79	18	Uniform fine to medium grain size Sand
		SS-06	75	13	
		SS-07	79	10	
77.0 / 5.2	Sand, damp to moist, dense to very dense.	SS-08	79	29	Uniform fine to medium grain size Sand
		SS-09	100	77	
		SS-10	92	69	
		SS-11	87	64	
73.9 / 8.2	Sand, traces of gravel, light brown, damp to moist, dense to very dense.	SS-12	87	42	2 89 9 Uniform coarse grain size Sand to the end of borehole
73.0 / 9.1	Sand, some gravel, light brown, damp to moist, very dense.	SS-13	83	43	

Column headings also printed: DEPTH - feet (5, 10, 15, 20, 25, 30); DEPTH - meters (1–9); SOIL PROFILE; SYMBOL; SAMPLES; STATE; GROUNDWATER CONDITIONS; DYNAMIC CONE PEN. RESISTANCE PLOT 20 40 60 80; SHEAR STRENGTH (kPa) Vane test ◇ Intact ◆ Remolded, Lab vane □ Intact ■ Remolded, 20 40 60 80 100; WATER CONTENT and LIMITS (%) W_P W W_L 25 50 75.

\\LICENSES7\Sobek\Geotec80\Style\Log_Borehole_v5.sty

McINTOSH PERRY

BOREHOLE No BH20-2

DATE:	15/09/2020 - 15/09/2020	**LOCATION:**	810 Vick Ave. and 2830 Carling Ave.	**ORIGINATED BY:**	M.A.
PROJECT:	CCO-21-1191	**COORDINATES:**	Lat: 45.358130872 , Lon: -75.797292716	**COMPILED BY:**	M.A.
CLIENT:	Holzman Consultant Inc.	**DATUM:**	Geodetic	**CHECKED BY:**	N.T.
ELEVATION:	82.17 m	**REMARK:**		**REPORT DATE:**	08/10/2020

DEPTH - feet	DEPTH - meters	SOIL PROFILE: ELEVATION - m / DEPTH - m	SOIL PROFILE: DESCRIPTION	SYMBOL	SAMPLES: TYPE AND NUMBER	SAMPLES: STATE	SAMPLES: RECOVERY	SAMPLES: "N" or RQD	GROUNDWATER CONDITIONS	DYNAMIC CONE PEN. RESISTANCE PLOT (20 40 60 80) / SHEAR STRENGTH (kPa) Vane test: ◇ Intact ◆ Remolded; Lab vane: □ Intact ■ Remolded (20 40 60 80 100)	WATER CONTENT and LIMITS (%) W_P W W_L (25 50 75)	REMARKS & GRAIN SIZE DISTRIBUTION (%) G S M C
		72.4 9.8	**End of Borehole** **No water was observed in open borehole.**		SS-13		83	43				
	10											
35	11											
40	12											
	13											
45	14											
	15											
50	16											
55	17											
	18											
60												

\\LICENSES7\Sobek\Geotec80\Style\Log_Borehole_v5.sty

McINTOSH PERRY

BOREHOLE No BH20-3

DATE:	16/09/2020 - 16/09/2020	**LOCATION:**	810 Vick Ave. and 2830 Carling Ave.	**ORIGINATED BY:**	M.A.
PROJECT:	CCO-21-1191	**COORDINATES:**	Lat: 45.357803679 , Lon: -75.797771179	**COMPILED BY:**	M.A.
CLIENT:	Holzman Consultant Inc.	**DATUM:**	Geodetic	**CHECKED BY:**	N.T.
ELEVATION:	81.10 m	**REMARK:**		**REPORT DATE:**	08/10/2020

Column headings: DEPTH - feet; DEPTH - meters; SOIL PROFILE (ELEVATION - m / DEPTH - m; DESCRIPTION; SYMBOL); SAMPLES (TYPE AND NUMBER; STATE; RECOVERY; "N" or RQD); GROUNDWATER CONDITIONS; DYNAMIC CONE PEN. RESISTANCE PLOT (20 40 60 80); SHEAR STRENGTH (kPa) (Vane test: ◇ Intact, ◆ Remolded; Lab vane: □ Intact, ■ Remolded; 20 40 60 80 100); WATER CONTENT and LIMITS (%) (W_P W W_L; 25 50 75); REMARKS & GRAIN SIZE DISTRIBUTION (%) (G S M C)

Elevation (m) / Depth (m)	Description	Sample	Recovery	"N" or RQD	Remarks & Grain size (G S M C)
81.1	Natural ground surface				
0.0	Asphalt.				
81.0 / 0.1	Fill : Sand and gravel, traces of silt, brown, damp.	GS-01			
80.9 / 0.2	Fill : Sand, brown, damp.				
80.3 / 0.8	Sand, traces of silt, light brown, damp, compact.	SS-02	58	12	
79.7 / 1.4	Sand, traces of silt and gravel, light brown, damp, compact to dense.	SS-03	67	19	
		SS-04	83	29	
78.1 / 3.0	Sand, traces of silt and gravel, light brown, damp to moist, dense to very dense.	SS-05	83	35	
		SS-06	87	36	
		SS-07	87	39	Uniform coarse grain size Sand
		SS-08	83	40	A piece of limestone (20 mm dia.) in the sample.
75.0 / 6.1	Sand, traces of silt and gravel, light brown, damp, dense to very dense.	SS-09	75	32	1 94 5
		SS-10	92	44	Uniform very coarse grain size Sand
		SS-11	92	52	
72.9 / 8.2	Till : Sand and gravel, light brown, dry to damp, very dense. Presence of cobbles and boulders.	SS-12	83	63	A piece of limestone in the sampler tip.
		SS-13	79	68	Auger grinding

Depth scale (feet): 5, 10, 15, 20, 25, 30. Depth scale (meters): 1, 2, 3, 4, 5, 6, 7, 8, 9.

\\LICENSES7\Sobek\Geotec80\Style\Log_Borehole_v5.sty

McINTOSH PERRY

BOREHOLE No BH20-3

DATE:	16/09/2020 - 16/09/2020	**LOCATION:**	810 Vick Ave. and 2830 Carling Ave.	**ORIGINATED BY:**	M.A.
PROJECT:	CCO-21-1191	**COORDINATES:**	Lat: 45.357803679 , Lon: -75.797771179	**COMPILED BY:**	M.A.
CLIENT:	Holzman Consultant Inc.	**DATUM:**	Geodetic	**CHECKED BY:**	N.T.
ELEVATION:	81.10 m	**REMARK:**		**REPORT DATE:**	08/10/2020

SOIL PROFILE | **SAMPLES** | **GROUNDWATER CONDITIONS** | **DYNAMIC CONE PEN. RESISTANCE PLOT** (20, 40, 60, 80) | **SHEAR STRENGTH (kPa)** Vane test: ◇ Intact, ◆ Remolded; Lab vane: □ Intact, ■ Remolded (20, 40, 60, 80, 100) | **WATER CONTENT and LIMITS (%)** W_P W W_L (25, 50, 75) | **REMARKS & GRAIN SIZE DISTRIBUTION (%)** G S M C

ELEVATION - m / DEPTH - m	DESCRIPTION	TYPE AND NUMBER	RECOVERY	"N" or RQD	REMARKS & GRAIN SIZE DISTRIBUTION (%)
		SS-13	79	68	
					Auger grinding
		SS-14	79	74	
					A piece of limestone in the sampler tip
		SS-15	58	88	
					Auger grinding
		SS-16	67	107	G 38, S 48, M 14
					Shatered pieces of limestone (boulder) in the spoon
					Auger grinding
		SS-17A	67	65	
68.5 / 12.6	Sand, some gravel, light brown, dry to damp, very dense.	SS-17B	100		
					Drilling was terminated at 12.2 m.
					Core washing with NQ casing was started at 12.2 m. The core washing advnced difficulty to 15.24 m and was terminated due to significant resistance from the soil
65.9 / 15.2	Sand, some gravel, traces of silt, light brown, very dense.	SS-18	71	47	G 10, S 81, M 9
65.1 / 16.0	**END OF BOREHOLE**				

DEPTH - feet: 35, 40, 45, 50, 55, 60
DEPTH - meters: 10, 11, 12, 13, 14, 15, 16, 17, 18

\\LICENSES7\Sobek\Geotec80\Style\Log_Borehole_v5.sty

2830 CARLING AVE. AND 810 VICK AVE.

APPENDIX D
LAB RESULTS

Only selected pages from the third-party lab are included in this appendix

MCINTOSH PERRY

PARACEL LABORATORIES LTD. | TRUSTED. RESPONSIVE. RELIABLE.

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

Certificate of Analysis

McIntosh Perry Consulting Eng. (Nepean)

215 Menten Place, Unit 104
Nepean, ON K2H 9C1
Attn: Harrison Smith

Client PO:
Project: CO-21-1191
Custody: 128749

Report Date: 7-Oct-2020
Order Date: 30-Sep-2020

Order #: 2040490

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

Paracel ID	Client ID
2040490-01	BH20-02 SS5
2040490-02	BH20-03 SS13
2040490-03	BH20-03 SS17 B

Approved By:

Dale Robertson, BSc
Laboratory Director

Certificate of Analysis

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

Client PO:

Report Date: 07-Oct-2020

Order Date: 30-Sep-2020

Project Description: CO-21-1191

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	6-Oct-20	6-Oct-20
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	2-Oct-20	3-Oct-20
Resistivity	EPA 120.1 - probe, water extraction	6-Oct-20	7-Oct-20
Solids, %	Gravimetric, calculation	2-Oct-20	5-Oct-20

Certificate of Analysis

Client: McIntosh Perry Consulting Eng. (Nepean)

Client PO:

Report Date: 07-Oct-2020

Order Date: 30-Sep-2020

Project Description: CO-21-1191

	Client ID:	BH20-02 SS5	BH20-03 SS13	BH20-03 SS17 B	-
	Sample Date:	11-Sep-20 09:00	11-Sep-20 09:00	11-Sep-20 09:00	-
	Sample ID:	2040490-01	2040490-02	2040490-03	-
	MDL/Units	Soil	Soil	Soil	-
Physical Characteristics					
% Solids	0.1 % by Wt.	96.6	97.1	96.9	-
General Inorganics					
pH	0.05 pH Units	7.96	8.19	8.28	-
Resistivity	0.10 Ohm.m	49.7	71.9	63.5	-
Anions					
Chloride	5 ug/g dry	66	28	53	-
Sulphate	5 ug/g dry	34	26	19	-

Qualifier Notes:

None

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.

2830 CARLING AVE. AND 810 VICK AVE.

APPENDIX E
SEISMIC HAZARD CALCULATION

McINTOSH PERRY

2010 National Building Code Seismic Hazard Calculation

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

Site: 45.358N 75.798W **User File Reference:** 2830 Carling Ave. and 810 Vick Ave. 2020-10-07 21:12 UT

Requested by: McIntosh Perry

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.2)	0.630	0.382	0.245	0.087
Sa (0.5)	0.305	0.184	0.120	0.042
Sa (1.0)	0.136	0.086	0.055	0.017
Sa (2.0)	0.046	0.028	0.018	0.006
PGA (g)	0.322	0.198	0.121	0.037

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

References

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

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

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

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

Liquefaction Evaluation for the Proposed Development on

2830 Carling Ave. and 810 Vick Ave.

Project #: CCO-21-1191

Soil stratigraphy for the site of the proposed development consists of a thick sand deposit that extends to the maximum depth of investigation which is approximately 16.0 m below the existing ground level in BH20-3. The native sand layer is interbedded by a relatively thick dense till layer that is approximately 4.25m thick. Herein liquefaction susceptibility of the native sand layer and the till layer is evaluated.

For coarse-grained soils with fines content up to 35%, the corrected SPT resistance can be used to determine the susceptibility of the coarse-grained soil to liquefaction according to Canadian Foundation Engineering Manual CFEM (2006). Five representative samples from the native sand layer and one representative sample from the till layer underwent grain size analysis. The percentage of gravel, sand, silt and clay are presented in Table 1.

Table 1: Grain Size Distribution of native Sand/Silty Sand

Borehole No.	Sample No.	$(N_1)_{60}$	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	r_d	*CSR*
BH20-01	○ SS-06	35	3.8 – 4.4	0.3	81.4	18.3		0.97	0.021
BH20-01	△ SS-10	48	6.9 – 7.4	1.3	90.9	7.8		0.95	0.020
BH20-02	◇ SS-12	36	7.6 – 8.2	1.4	88.9	9.7		0.94	0.020
BH20-03	□ SS-09	30	6.0 – 6.7	0.7	93.5	5.8		0.95	0.020
BH20-03	▽ SS-16	44	11.4 – 12.0	38.4	47.2	14.4		0.86	0.018
BH20-03	⬢ SS-18	29	15.2 – 15.8	12.0	79.5	8.5		0.76	0.016

To evaluate the liquefaction susceptibility of the native sand and till layers using SPT test results, Cyclic Stress Ratio (CSR) has to be estimated based on site seismicity characteristics that were obtained from seismic calculator available on Natural Resources Canada website. CSR can be calculated using the following formula:

$$CSR = 0.65 \times \frac{a_{max} . \sigma_v}{g . \sigma'_{v0}} \times r_d$$

where a_{max} is the peak ground surface acceleration for the designed earthquake, g is gravity acceleration (9.81 m/s^2), σ_v is total vertical overburden pressure, σ'_{v0} is the initial effective overburden pressure and r_d is stress reduction factor at the depth of interest. r_d and *CSR* values are presented in Table 1.

Based on the calculated CSR and corrected SPT values, Figure 1 from CFEM can be used to evaluate the native sand and till layers susceptibility to liquefaction. The CSR results and the corrected SPT 'N' values were plotted on the figure and the native sand and till layers were found to be non-susceptible to liquefaction.

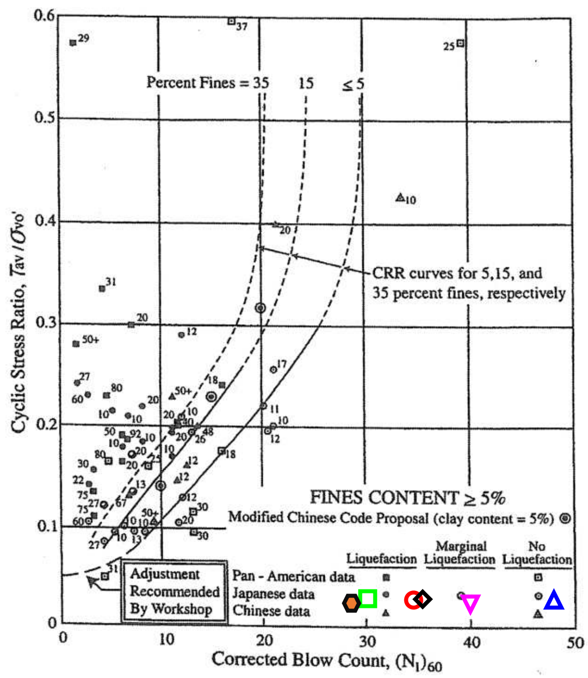
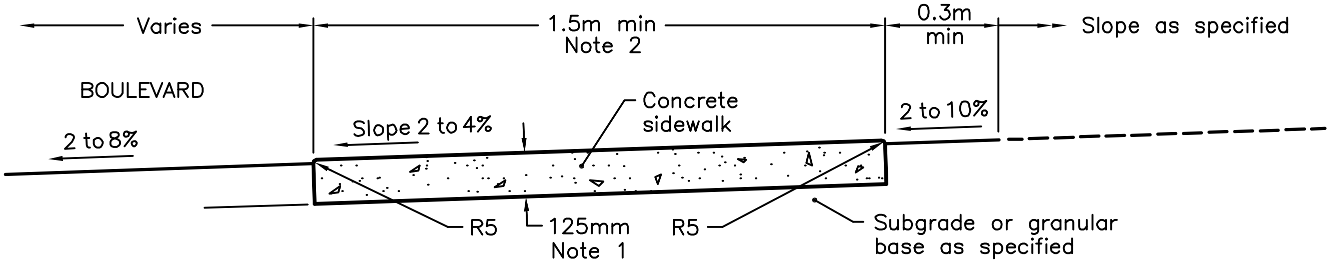


Figure 1: CRS vs Corrected SPT N value, $(N_1)_{60}$ (modified from CFEM 2006)

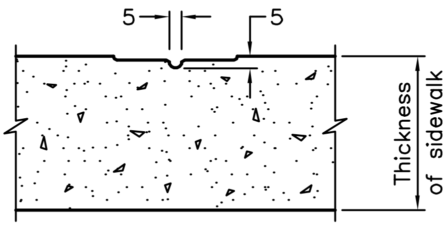
2830 CARLING AVE. AND 810 VICK AVE.

APPENDIX F
RELEVANT STANDARDS

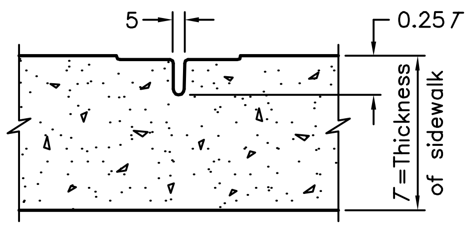
McINTOSH PERRY



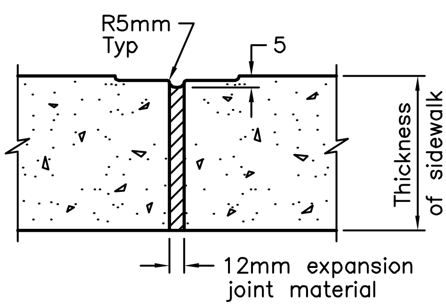
TYPICAL SECTION



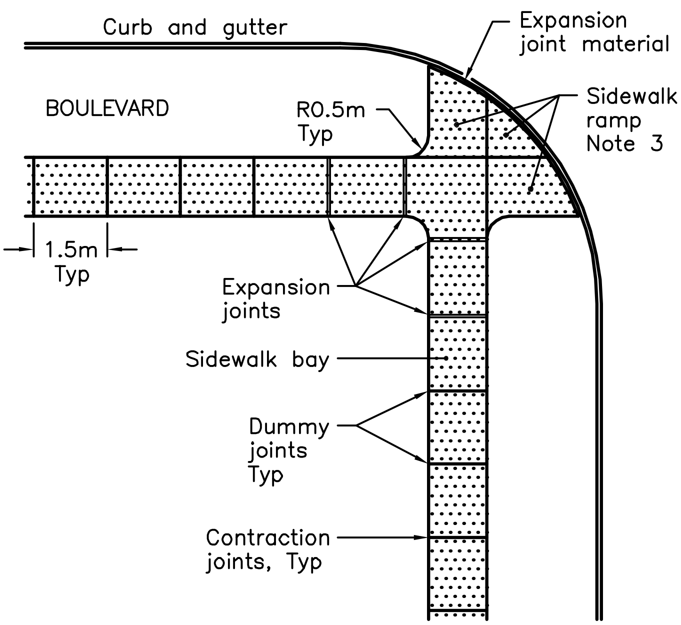
DUMMY JOINT (OPTIONAL)



CONTRACTION JOINT



EXPANSION JOINT



JOINT LAYOUT

NOTES:

1 Sidewalk thickness at residential driveways and adjacent to curb shall be 150mm. At commercial and industrial driveways, the thickness shall be 200mm.

2 Sidewalk width shall be wider when specified.

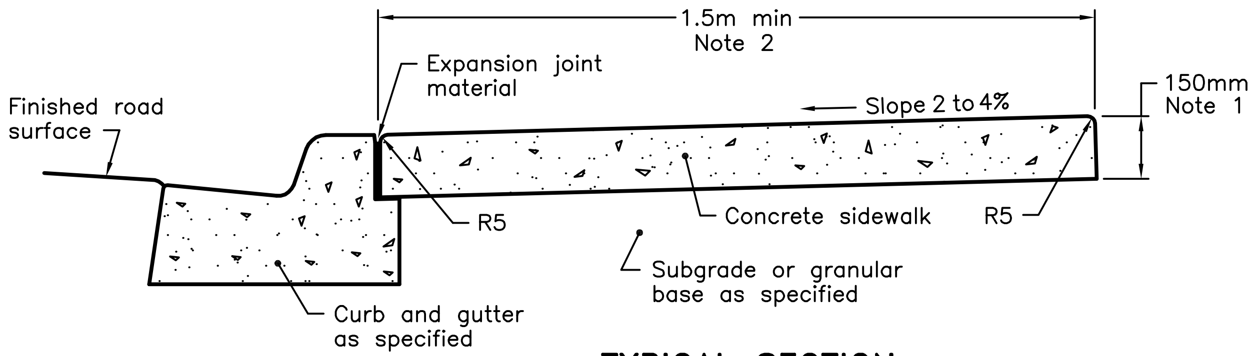
3 This OPSD shall be read in conjunction with OPSD 310.030, 310.031, 310.032, 310.033 and 310.039.

A All dimensions are in millimetres unless otherwise shown.

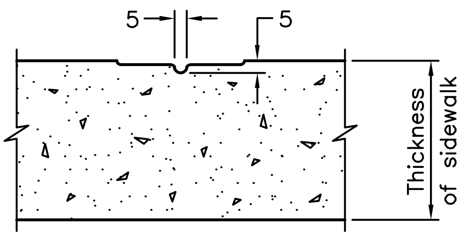
ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2015 | Rev 2

CONCRETE SIDEWALK

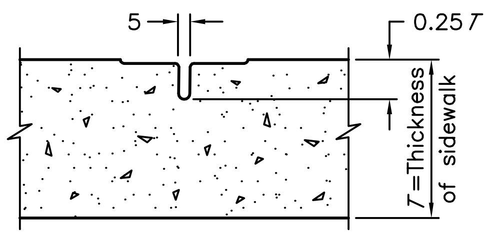
OPSD 310.010



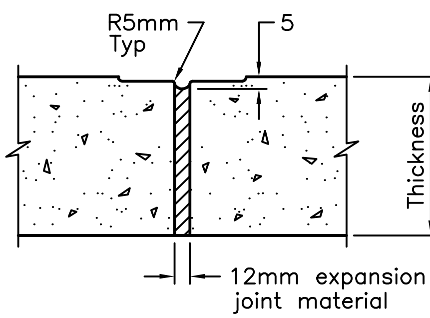
TYPICAL SECTION



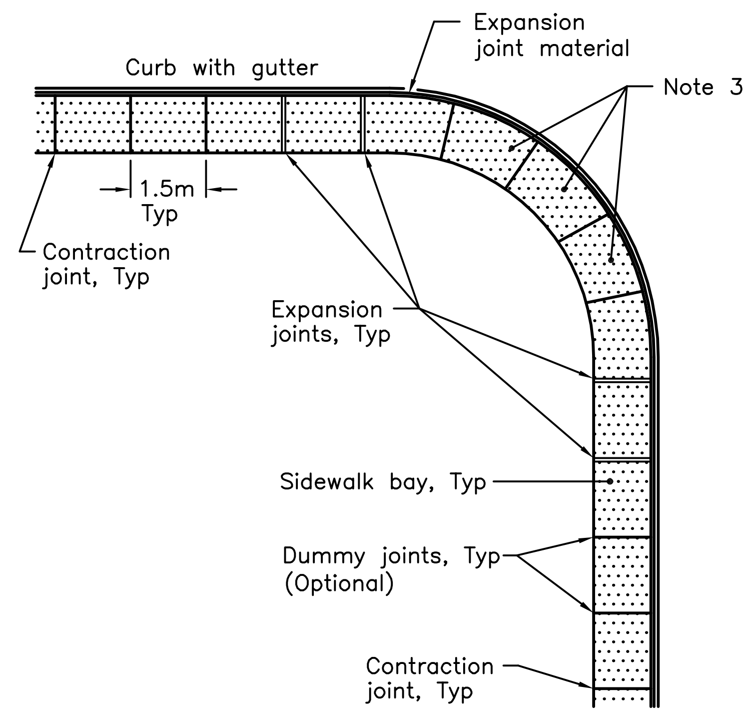
DUMMY JOINT



CONTRACTION JOINT



EXPANSION JOINT



JOINT LAYOUT

NOTES:

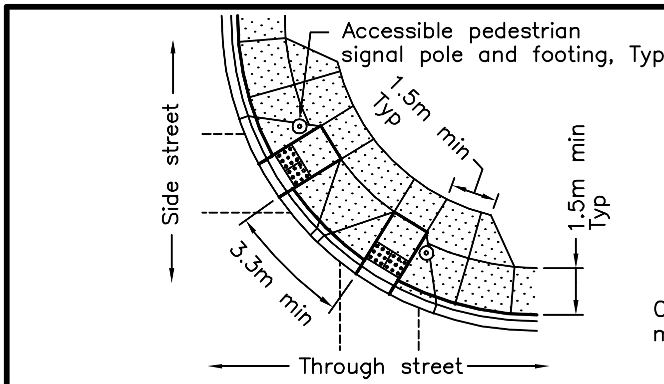
1 At commercial and industrial driveways, the thickness shall be 200mm.

2 Sidewalk width shall be wider when specified.

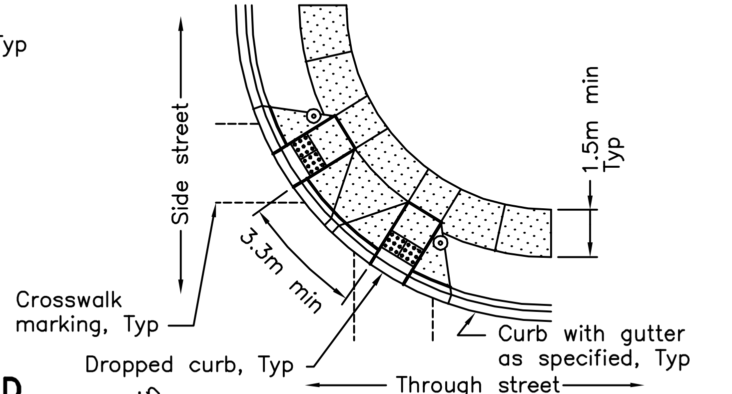
3 This OPSD shall be read in conjunction with OPSD 310.030, 310.031, 310.033, and 310.039.

A All dimensions are in millimetres unless otherwise shown.

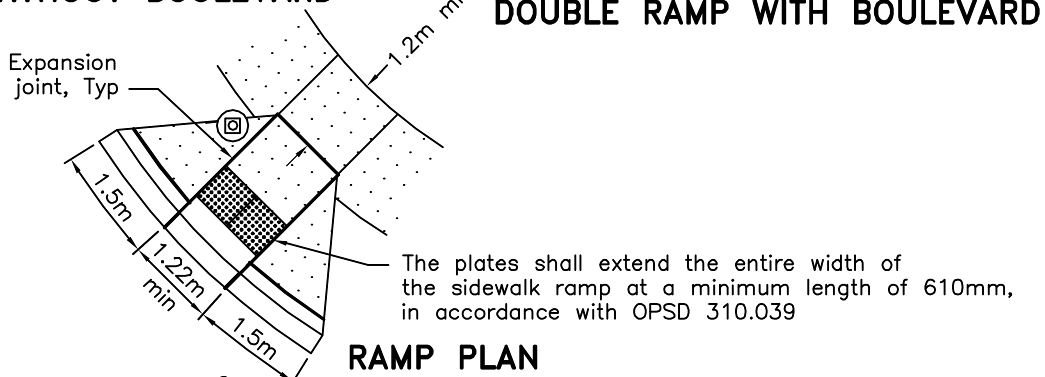
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2019	Rev 3
CONCRETE SIDEWALK ADJACENT TO CURB WITH GUTTER		
	OPSD 310.020	



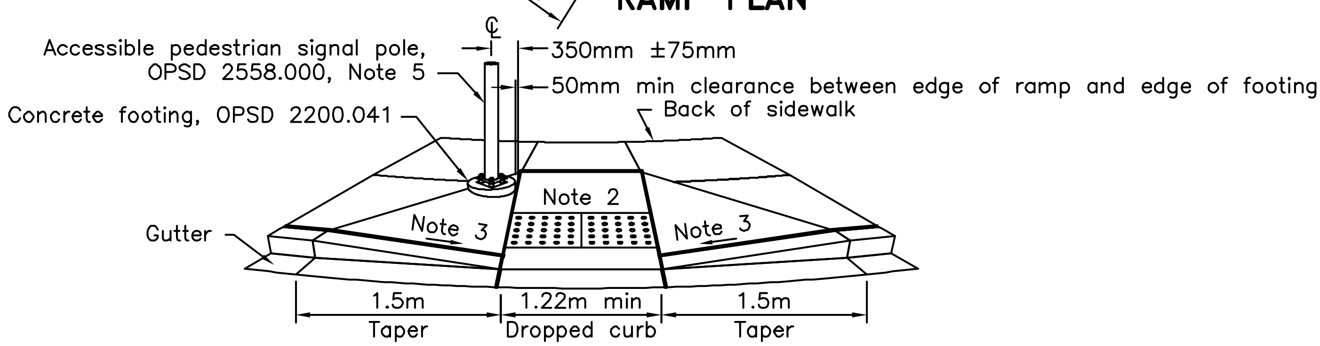
DOUBLE RAMP WITHOUT BOULEVARD



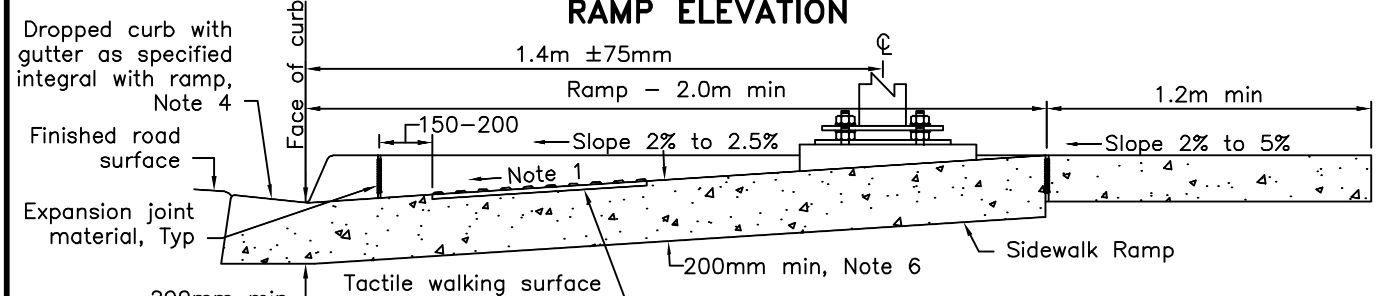
DOUBLE RAMP WITH BOULEVARD



RAMP PLAN



RAMP ELEVATION



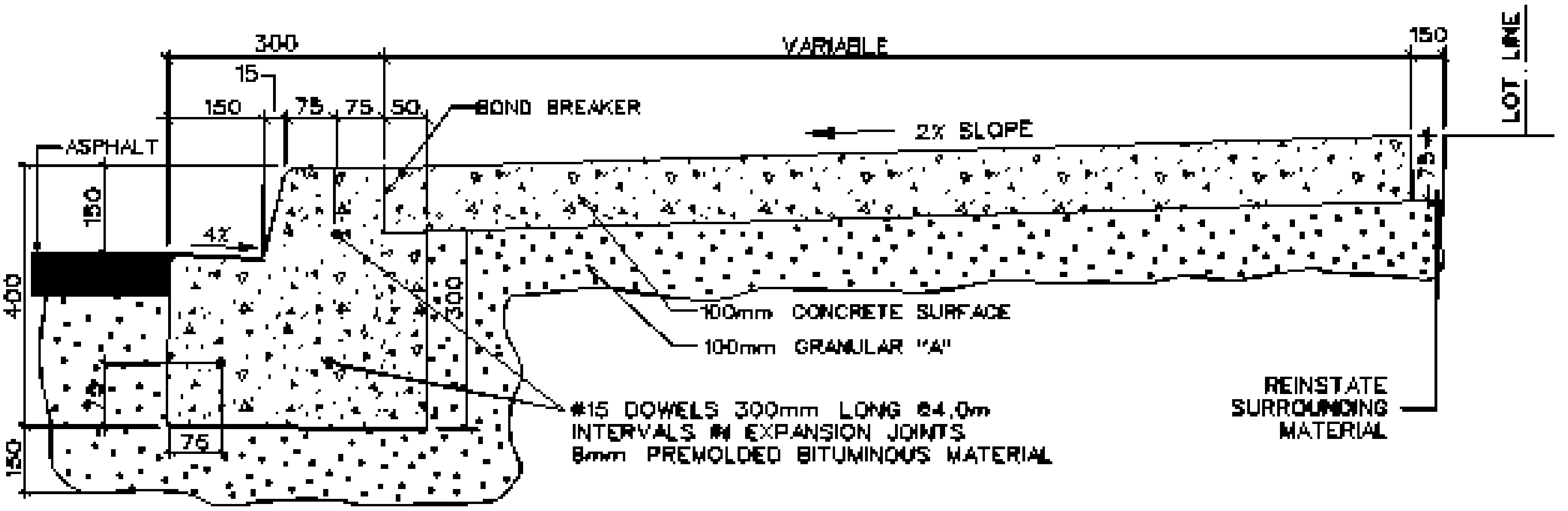
RAMP SECTION

NOTES:

1 Slope of ramp shall not exceed 8%.
2 Cross slope of ramp shall not exceed 2% in either direction.
3 Cross slope of flared side of ramp shall not exceed 8%.
4 Dropped curb at ramp shall be modified to eliminate 30 mm step at gutter.
5 Accessible pedestrian signal to have tactile arrow that aligns with direction of crosswalk.
6 Minimum thickness of ramp is 200mm. Minimum thickness of sidewalk and flared sides adjacent to ramp is 150mm.

A Where only one crosswalk is present at an intersection, only one curb ramp is required.
B All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2015	Rev 1
CONCRETE SIDEWALK RAMPS AT SIGNALIZED INTERSECTIONS		
	OPSD 310.030	



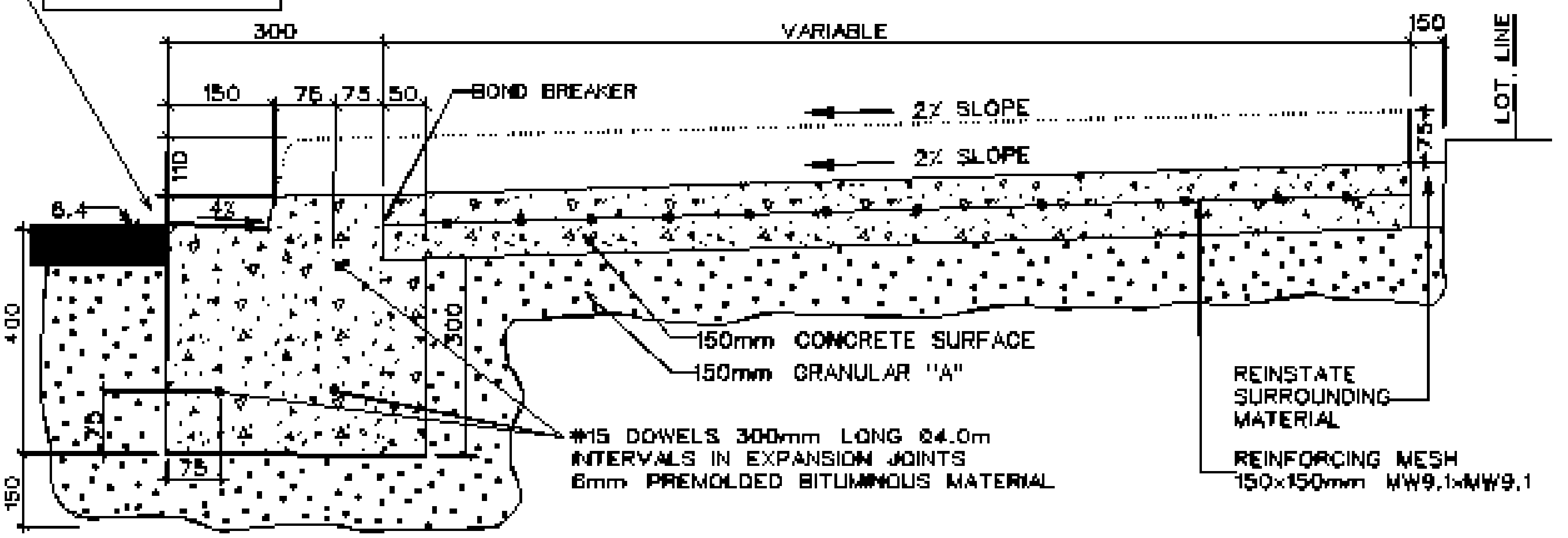
TYPICAL SIDEWALK SECTION

PEDESTRIAN

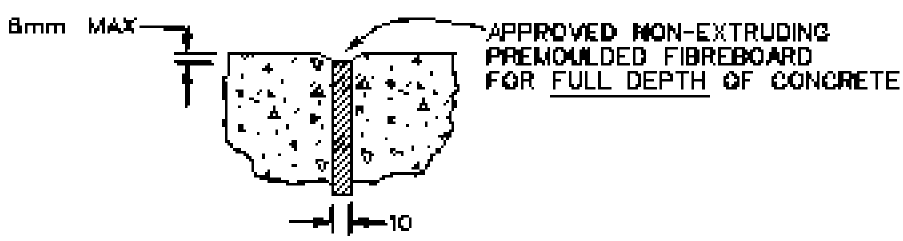
MINIMUM	0mm
DESIRED	15mm
MAXIMUM	25mm

VEHICULAR

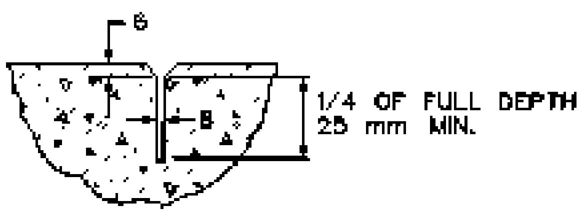
MINIMUM	15mm
DESIRED	25mm
MAXIMUM	40mm



SECTION AT DRIVEWAY AND PEDESTRIAN RAMPS



EXPANSION JOINT PROFILE



DUMMY JOINT PROFILE

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS SHOWN OTHERWISE.

Ottawa

CONCRETE CURB AND GUTTER WITH SIDEWALK

DATE: MAY 2001

REV. DATE: MARCH 2002

DWG. No.: SC3