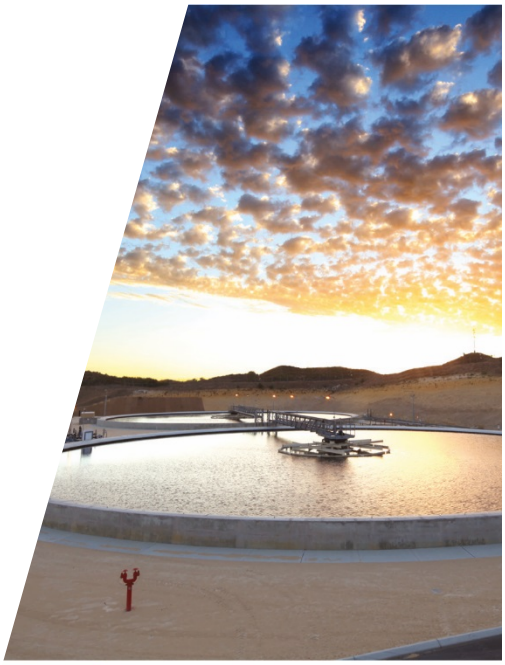


Geotechnical Investigation

Port Dalhousie Legion Redevelopment
57 Lakeport Road
St. Catharines, Ontario

Rankin Construction Inc.



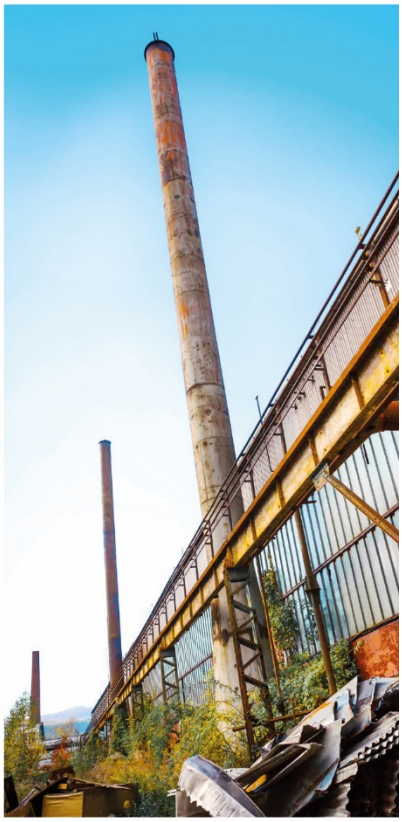






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Appendix B Rock Core Photo Log
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1. Introduction

GHD Limited (GHD) has been retained by Rankin Construction Inc. (Rankin) to conduct geotechnical and preliminary hydrogeological investigations for an eight storey 1,736 m^2 building structure with grade level and terrace parking, new driveways and associated utility infrastructure proposed to be constructed at a 2,200 m^2 property located at 57 Lakeport Road, St. Catharines, Ontario (Site or Property). The Site Location Map is shown as Figure 1.

In response to request from Mr. Jim Frank of Rankin, GHD submitted proposal for geotechnical and preliminary hydrogeological investigations at the Site referenced 11131634-98 and dated September 6, 2018. Authorization to proceed with the geotechnical investigation scope of works (SOW) was received from Mr. Frank via email dated September 18, 2018. GHD has carried out the geotechnical investigation to determine to subsurface soil and groundwater conditions for the proposed infrastructure. This report includes the findings of the geotechnical investigation and provides the geotechnical design and construction recommendations based thereon.

A preliminary hydrogeological investigation has also been conducted at the Site, and the findings are being issued under separate cover.

2. Previous Investigations

The following reports were reviewed by GHD concurrent to preparation of this geotechnical investigation report:

- Peto MacCallum Ltd. (August 1987): Geotechnical Investigation, Addition and Alterations, Royal Canadian Legion, Branch No. 350, Port Dalhousie, Ontario
- GHD Limited (September 2, 2016): Phase Two Environmental Site Assessment, 57 Lakeport Road, St. Catharines, Ontario
- GHD Limited (June 29, 2017): Phase Two Environmental Site Assessment, 57 Lakeport Road, St. Catharines, Ontario
- GHD Limited (July 11, 2017): Geotechnical Desktop Study, Proposed Seven Storey Condominium Building, 57 Lakeport Road, St. Catharines, Ontario

The relevant information from the above reports has been incorporated into this report where required. The reader is referred to the above reports for the details.

3. Site Description

The Property is located in the north end of St. Catharines, Ontario, in the Port Dalhousie Heritage District. The existing building located on the eastern portion of the Site was originally constructed in 1890 as a repair facility to support the Old Welland Canal lock formerly operated to the south of the Site until the Royal Canadian Legion acquired and occupied the Site in 1951. The exterior portion of the Site is complete with commercial parking on an asphalt paved parking lot occupying the western portion of the Property. The existing and proposed developments are shown on Figure 2.

4. Field and Laboratory Work Programs

The proposed geotechnical investigation SOW comprised of advancing a total of five (5) boreholes within the proposed building footprint to auger refusal on bedrock and core drilling to a depth of 5 m into bedrock at two (2) of the borehole locations.

Prior to commencement of the field activities GHD prepared a Site-specific Health and Safety Plan (HASP) to be implemented during the field investigation program. The HASP presented the visually observed Site conditions and identified potential physical hazards to field personnel. Required personal protective equipment were also listed in the HASP. It was mandatory for all GHD personnel involved in the field program to read, understand and abide by the HASP. Health and Safety requirements in the HASP were implemented throughout the field investigation program.

Details of the field and laboratory work programs are provided in the subsections below.

4.1 Fieldwork Program

The SOW comprised of advancing five boreholes, referenced BH1-18 to BH5-18 at the Site. The approximate as-drilled borehole locations are shown on Figure 2. All five boreholes were advanced to split spoon sampler or auger refusal on probable bedrock at depths ranging from 9.4 (BH1-18) to 18.3 (BH3-18) m below ground surface (BGS). The presence of bedrock was confirmed at the locations of BH1-18 and BH5-18 through diamond coring techniques. The bedrock was cored to a depth of 5.8 m and 4.1 m below the bedrock surface at the locations of boreholes BH1-18 and BH5-18, respectively.

The completed depth and ground surface elevation of each borehole are provided in the table below.

Table 4.1 Borehole Ground Surface Elevations and Depths

Borehole No.	Ground Surface Elevation (m amsl)	Bedrock Depth (m BGS)	Borehole Depth (m BGS)
BH1-18	78.82	9.43	16.54
BH2-18	78.97	10.67	12.35
BH3-18	77.67	18.29	18.42
BH4-18	76.10	17.53	17.60
BH5-18	76.82	17.38	22.82

GHD obtained underground utility clearances within proximity of the borehole locations at the Site prior to commencement of the field drilling program through Ontario1Call and other public utility owners. Additionally, a specialist utility locating subcontractor was retained by GHD to identify buried plant owned utilities within proximity of the borehole locations, prior to mobilizing for the field-drilling program.

The drilling program for the geotechnical investigation was conducted in September 28 to October 3, 2018 using a truck mounted Dietrich D50 drill rig equipped with solid and hollow stem augers operated by Elite Drilling Services. Drilling was completed under the full time supervision of GHD's geotechnical staff who logged the stratigraphy of overburden and bedrock in each borehole. The borehole logs are presented in Appendix A.

Representative samples of the strata were obtained utilizing a 50 millimeter (mm) outer diameter split-barrel sampler, advanced by dropping a 63.5 kilogram (kg) hammer 760 mm, in accordance with the Standard Penetration Test (SPT) method (ASTM D1586). The SPT results are reported as 'N' values on the borehole logs at the corresponding depths.

Bedrock was inferred to be present when relatively little advancement of the boreholes was observed. This was considered to be refusal to augering or SPT sampling. Rock coring was carried out below the inferred bedrock depths at both BH1-18 and BH5-18 locations, in general accordance with ASTM D2113[1], to confirm the presence of bedrock and to investigate the bedrock lithology. Wire line techniques using NQ size cores (75.7 mm outside diameter and 50 mm inside diameter) were used to advance the boreholes into the bedrock. GHD documented the rock type, percentage recovery, rock quality designation (RQD), the amount of water loss/return, and presence of voids or cavities in the bedrock. The rock cores were placed in partitioned wooden core boxes to keep each core run separate with depths of recovery clearly marked. The percentage core recovery and RQD values are provided on the borehole logs included in Appendix A. Pictures of recovered core samples have been provided in Appendix B. Boreholes were backfilled with bentonite chips and soil cuttings in accordance with O. Reg. 903. Excess soil cuttings were spread around the borehole locations.

Soil samples obtained from the boreholes were observed in the field for type, texture, color and moisture, and were sealed in clean plastic containers. All samples were transferred to GHD's St. Catharines laboratory where they were re-examined by a senior geotechnical engineer to verify the accuracy of the initial soil description/identification and selection for laboratory testing.

4.2 Laboratory Work Program

Laboratory tests were performed on select, representative samples to evaluate physical properties of the bedrock that may affect the geotechnical aspects of project design and construction. Laboratory tests, consisting of the following, were performed in general accordance with applicable standards:

- Moisture content determination on all SPT soil samples collected.
- Unconfined compressive strength determination of 3 rock samples in accordance with ASTM D7012 - Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures.
- Elastic Moduli determination of 3 rock samples in accordance with ASTM D7012 - Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures.

The geotechnical laboratory test results are discussed in Section 4, and the laboratory test reports are provided in Appendix C.

[1] Standard Practice for Rock Core Drilling and Sampling of Rock for Site Exploration

5. Regional Geology

The Site is located in the physiographic region of Lake Iroquois Plain[2], which covers the lowland below the Niagara Escarpment up to the Lake Ontario shoreline.

A review of the Quaternary geology map 2496[3] shows that the Site is located on a peninsula shaped reclaimed piece of land within the Martindale Pond. The Quaternary deposits in the general Site area are comprised of glaciolacustrine clayey deposits. These deposits are underlain by clayey Halton Till that covers the Queenston Shale Bedrock.

A review of the bedrock topography map of the area[4] shows that the bedrock surface elevation within the study area is at approximately 64 m Above Mean Sea Level (AMSL), approximately 16 m or more below the existing grades.

6. Subsurface Conditions

The subsurface conditions are detailed on borehole logs included in Appendix A, and are generally consistent with the regional geology. Subsurface conditions were determined at the borehole locations only, and may vary at other locations. The boundaries between various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between various strata, rather than a precise plane of geological change.

A brief description of the soil stratigraphy encountered at each borehole location is summarized in order of depth in the following sections.

6.1 Pavement Structure

All five boreholes were advanced through the existing paved parking lot areas and encountered 60 to 125 mm thick asphaltic concrete or bituminous material at the ground surface.

Below the asphaltic concrete/bituminous material sand and gravel material, presumably used as a pavement base/subbase structure, was encountered. The thickness of sand and gravel material was estimated to range from approximately 0.7 m to 2 m. Inclusions of wood and clay were encountered within the deeper/thicker sand and gravel materials indicating that underlying fill deposits and sand and gravel materials have comingled at some locations.

6.2 Fill/Possible Fill

Fill deposits, mainly comprised of sandy silt to clayey silt, were encountered below the pavement structure at all five borehole locations. It is noted that similar fill deposits were encountered at the 2017 GHD Phase Two ESA monitoring well locations. The 1987 Peto MacCallum borehole logs also show the presence of similar fill deposits.

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[2] Chapman, L. J., and Putnam, D. F., (1984): The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, 270p. Accompanied by Map 2715 (coloured), Scale 1:600,000.

[3] Feenstra, B. H., (1984): Quaternary Geology of the Niagara-Welland Area; Ontario Geological Survey, Map 2496, Quaternary Geology Series, Scale 1:50,000 Geology 1969-1972

[4] Feenstra, B. H. (1981): Bedrock Topography of the Niagara and Niagara-on-the-lake Area, Southern Ontario; Ontario Geological Survey Preliminary Map P.2400, Bedrock Topography Series, Scale 1:50,000

The fill materials are generally comprised of clayey to sandy silty materials with inclusions of wood and brick fragments indicating that construction debris was likely used to reclaim the land from the Martindale Pond. The samples from the deeper fill deposits were observed to have significant peat/organic content, indicating that the fill deposits were dumped directly on the pond bed on the then existing peat deposits, which since then have completely comingled. The depth of fill deposits is summarized in the following table

Table 6.1 Fill Layer Depth

Borehole No.	Depth (m BGS)	Soils
2018 GHD Geotechnical Investigation		
BH1-18	9.2	Silt/Silty clay to 6.1, Peat to 9.2 m
BH2-18	7.8	Clayey silt to 6.1, silty sand to 7.8 m
BH3-18	12.2	Silty clay to 10.7, Peat to 12.2 m
BH4-18	9.2	Silty clay to 9.2 m
BH5-18	17.4	Sandy silt/silty sand to 17.4 m

The SPT 'N' values in the fill deposits general ranged from 2 to 7 with isolated values as high as 55. The general range of the SPT 'N' values is indicative of the very loose to loose or very soft to firm conditions, whereas the isolated high values are likely due to the presence of obstructions (brick and wood fragments etc.) in the fill deposits.

The moisture content of the fill deposit samples ranged from 13 to 43 percent indicating generally wet conditions although moisture contents in excess of 30 percent are likely due to the presence of organic content.

6.3 Native Soil

At the location of BH1-18 a veneer of approximately 300 mm thick silt was encountered below the fill deposits overlying the Shale bedrock. At the remaining borehole locations, except BH5-18, approximately 3 to 8.5 m thick native generally silty clayey deposits were encountered overlying the Shale bedrock. At the location of BH5-18 fill deposits were directly overlying the Shale bedrock.

Due to the limited SPT sampling, only one SPT "N' value of 2 was recorded in the clay deposits. A review of the Pet MacCallum borehole logs shows the presence of sand and silt alluvium instead of clay with the SPT 'N' values ranging from 0 to 4 increasing to 10 closer to the bedrock indicating generally very loose to loose state of relative density. The moisture content profile shown on the Peto MacCallum logs indicates a moisture content range of 20 to 35 percent with isolated values higher than 40 percent, which are likely due to the organic content.

6.4 Bedrock

6.4.1 Stratigraphy and Rock Quality

The depths of bedrock at boreholes BH1-18 to BH5-18 and historical boreholes are summarized in the following table.

Table 6.2 Bedrock depth and Bedrock Surface Elevations

Borehole No.	Ground Surface Elevation (m amsl)	Bedrock Depth (m BGS)	Bedrock Elevation (m AMSL)
2018 GHD Geotechnical Investigation			
BH1-18	78.82	9.43	69.39
BH2-18	78.97	10.67	68.30
BH3-18	77.67	18.29	59.38
BH4-18	76.10	17.53	58.57
BH5-18	76.82	17.38	59.44
1987 Peto MacCallum Investigation			
BH-1	78.27	16.46	61.81
BH-2	77.72	17.52	60.20
BH-3	77.11	18.47	58.64
BH-4	77.26	18.13	59.13
BH-5	78.06	18.99	59.07

A review of Table 6.2 shows that the relief in rock surface elevation across the Site is in the order of 9.5 m, however, a review of the Figure 2 shows that the relief is not gradual but appears to be relatively sharp close to the north-northwest boundary of the Property between the GHD boreholes BH1-18 and BH2-18, and Peto MacCallum boreholes BH1 and BH2. The relief between the Peto MacCallum boreholes and remaining GHD boreholes BH3-18 to BH5-18 is relatively small, in the order of 3 m between boreholes BH4-18 and BH-1.

A review of the borehole logs included in Appendix A and photo log of the rock cores included in Appendix B shows that the dominant rock type encountered at the Site is the uppermost Ordovician brick red Shale of the Queenston Formation, with some green shale partings and scattered limestone interbeds.

Logs of boreholes BH1-18 and BH5-18 show that the core recovery ranged from 93 to 100 percent, except for the first core run RC1 in BH5-18 where a recovery of 7 percent was recorded indicating a highly fractured/weathered bedrock zone. The RQD values in the Shale bedrock ranged from 43 to 72. According to ASTM D6032 'Standard Test Method for Determining Rock Quality Designation of Rock Core' the RQD value range of 50 - 75 is indicative of 'Fair Quality' rock.

6.4.2 Unconfined Compressive Strength

Three samples were tested for unconfined compressive strength in accordance with ASTM D7012. The compressive strength for the samples were found to be 54.8, 32.4 and 49.9 MPa. The modulus of elasticity 'E' of the three rock core samples was determined and was found to be 9,900, 5,600 and 7,600 MPa, respectively. The Poisson's ratio of the three samples was measured to be 0.27, 0.3 and 0.31. The compressive strength test results are included in Appendix C and includes the pre and post-test photos of the samples.

Based on the criterion of unconfined compressive strength and the results of the unconfined strength tests, the rock cores can be characterized as moderately hard to hard rock.

6.5 Groundwater

Groundwater observations were made in the boreholes during drilling and upon completion of drilling, except in BH1-18 and BH5-18, and are presented on the borehole logs. Observations could not be made in boreholes BH1-18 and BH5-18 due to the use of drilling fluid required for rock coring. Groundwater observations were also made in the monitoring wells installed by GHD during the 2017 Phase Two ESA investigation in March 2017 and October 2018. The groundwater observations are summarized in the following table.

Table 6.3 Groundwater Observations on Drilling Completion

BH/MW No.	Ground Surface Elevation (m amsl)	Groundwater Elevation (m AMSL)		
		Mar. 24, 2017	Sept 28 to Oct 3, 2018	Oct.12, 2018
2018 GHD Geotechnical Investigation				
BH1-18	78.82	-	-	-
BH2-18	78.97	-	73.03	-
BH3-18	77.67	-	75.23	-
BH4-18	76.10	-	71.53	-
BH5-18	76.82	-	-	-
2017 GHD Phase Two ESA Investigation				
MW1-17	79.26	77.00	-	76.81
MW2-17	78.99	-	-	75.47
MW3-17	78.54	75.56	-	76.11
MW4-17	78.01	76.68	-	76.12
MW5-17	77.59	Not Accessible		
MW6-17	76.60	75.21	-	75.31

The water level in the Old Welland Canal was recorded at 74.47 m AMSL on October 12, 2018.

Due to the close proximity of the Martindale Pond, and Lake Ontario, and variations in the seasonal weather conditions, groundwater level should be expected to fluctuate in response to rising/ebbing Pond/Lake water levels, wet weather events, etc.

7. Discussion and Recommendations

7.1 General

It is understood that the development activities at the Site will consist of constructing an eight storey structure with a below grade parking level. The Finished Floor Elevation (FFE) of the ground level will be set at elevation 80.00 m AMSL. The FFE of the below the below grade parking level will be set at elevation 76.75 m AMSL, with top of the elevator shaft set at 75.12 m AMSL.

The Site ground surface has a mild slope towards the Old Welland Canal. It is assumed that the existing grades will generally be maintained and therefore no major cut and fill operations are anticipated. The Site is underlain by thick fill deposits and lake bed sediments containing significant organic content extending generally to the bedrock at depths of 16.5 to 19 m BGS though in the western portion of the proposed building footprint, the bedrock could be as shallow as 9.5 m BGS.

The groundwater table is 1.2 to 3 m BGS with a general groundwater flow direction from the Lake towards the Old Welland Canal.

Based on the above discussed understanding of the project development and the subsurface soil and groundwater conditions encountered at the borehole locations the following comments and recommendations are provided.

7.2 Building Foundations

The option of supporting the proposed eight storey structure on shallow foundation can be eliminated due to the presence of unsuitable soils extending to the bedrock.

The Site soil stratigraphy is comprised of low shear strength and compressible fill deposits. The Site soil conditions are therefore not favourable for shallow foundation support. The structural slab of the below grade parking lot can be supported on driven steel H-piles, when designed as a pile cap. This section provides recommendations for selected steel H-piles; other driven pile types and sections are available and can be considered; additional recommendations may be required.

7.2.1 Axial Pile Capacity

For the proposed structure, the driven steel HP or closed-end pipe piles bearing into bedrock are suitable foundation options. The steel piles should be fitted with rock shoes/points, and driven to refusal into the intact bedrock, expected to be in the order of approximately up to 19 m BGS. The competent fresh bedrock may be further 1 to 2 m deeper below the surface of the highly weathered Shale bedrock.

The rock shoe used must be capable of penetrating through the hard residual soils/highly weathered bedrock and bearing/cutting into the sound bedrock surface for full bearing.

The end bearing capacity of a pile bearing into bedrock is mainly a function of the allowable working stresses that the pile material can tolerate, and the allowable settlements. It is our understanding from email dated November 19, 2018 from Rankin, that pipe pile sections (HSS 406x11) were previously acquired for this Project. Based on our experience, for preliminary design purposes, the geotechnical load carrying capacity of typical steel H-piles and the above-referenced pipe pile provided in the following table can be used for (Φ = 0.4) Ultimate Limit State (ULS) design.

Table 7.1 Axial Pile Capacity

Pile Type	Area (m^2)	Avg. Unconfined Rock Compressive strength (kPa)	Single Pile ULS Capacity (ϕ =0.4)	
			Bedrock Bearing Resistance (kPa)	Single Pile Capacity (kN)
HP360x132	0.131	45,700	18,120	2,372
HP310x110	0.095	45,700	19,017	1,816
HP310x79	0.091	45,700	19,086	1,746
HP250x85	0.066	45,700	20,038	1,323
HSS 406x11 (Pipe)	0.129	45,700	17,830	2,300

The Serviceability Limit State (SLS) end bearing resistance will be a function of the allowable settlement of the proposed structure. As the load required to cause the typical allowable settlement

of 25 mm would exceed the ULS capacity for the pile bearing into the bedrock, the factored ULS axial capacities provided above can be used for the SLS design.

All pile installation activities must be carried out under the full time supervision of a qualified geotechnical inspector, to ensure that the required penetration depth into the bedrock with acceptable penetration resistance of the driven piles is achieved.

7.2.2 General Recommendations for Pile Foundations

The following general recommendations should be followed for design and installations of pile foundation system:

- Driving difficulties due to the possible presence of cobbles and boulders in the till deposits covering the bedrock should be expected.
- The pipe piles should be filled with concrete after installation for increased stiffness that will increase the lateral load carrying capacity of the pipe piles.
- The piling hammer selected must not be oversized. An oversized hammer may create excessive vibrations that may result in significant ground subsidence.
- The piles must be installed under geotechnical supervision to ensure that the piles are constructed in accordance with the above recommendations.
- A specialist piling-contractor should review the borehole logs and verify piling suitability and capacity based on piling experience under similar conditions.
- The actual design capacity must be confirmed by dynamic pile analysis and testing carried out using Pile Driving Analyzer (PDA) in accordance with ASTM D4945 on at least two driven test piles. It is also recommended that at least 10 percent of the service piles be tested for axial load capacity in compression for structural integrity using PDA.

8. Construction and Design Considerations

8.1 Excavation and Dewatering

The excavation for the elevator shaft and underground utilities is expected to be 2 to 3 m deep below the existing grades. The OHSA regulations require that if workmen must enter an excavation deeper than 1.2 m, the excavation must be suitably sloped and/or braced in accordance with the OHSA requirements. OHSA specifies maximum slope of the excavations for four broad soil types as summarized in the following table:

Table 8.1 OHSA Maximum Slope Inclinations

Soil Type	Base of Slope	Maximum Slope Inclination
1	Within 1.2 m of bottom	1 horizontal to 1 vertical
2	Within 1.2 m of bottom of trench	1 horizontal to 1 vertical
3	From bottom of excavation	1 horizontal to 1 vertical
4	From bottom of excavation	3 horizontal to 1 vertical

OHSA Section 226 defines the four soil types as follows:

Type 1 soil:

1. Is hard, very dense and only able to be penetrated with difficulty by a small sharp object
2. Has a low natural moisture content and a high degree of internal strength
3. Has no signs of water seepage
4. Can be excavated only by mechanical equipment

Type 2 soil:

1. Is very stiff, dense and can be penetrated with moderate difficulty by a small sharp object
2. Has a low to medium natural moisture content and a medium degree of internal strength
3. Has a damp appearance after it is excavated

Type 3 soil:

4. Is stiff to firm and compact to loose in consistency or is previously excavated soil
5. Exhibits signs of surface cracking
6. Exhibits signs of water seepage
7. If it is dry, may run easily into a well-defined conical pile
8. Has a low degree of internal strength

Type 4 soil:

9. Is soft to very soft and very loose in consistency, very sensitive and upon disturbance is significantly reduced in natural strength
10. Runs easily or flows, unless it is completely supported before excavating procedures
11. Has almost no internal strength
12. Is wet or muddy
13. Exerts substantial fluid pressure on its supporting system. Ontario regulation 213/91, s. 226 (5)

The groundwater is expected to be encountered at or around the approximate FFE of the below grade parking level. For the localized excavations the fill deposits above the groundwater can be classified as Type 3 soils, and Type 4 soils if affected by groundwater seepage.

Excavated soil stockpiles must be kept at least 1 m away from the top of an excavation. Storm water should be directed away from an open excavation.

Suitable dewatering methodology will be a function of the excavation method. For a sealed shoring system such as sheet piles or secant piles sealed into the bedrock, groundwater can be controlled by filtered sump pumping techniques. For permeable shoring systems such as soldier pile-wood lagging systems or open cut excavations, deep wells or vacuum well points or similar methods may be required.

A preliminary hydrogeological investigation was conducted at the Site in order to determine the volume of groundwater to be controlled during construction activities. The preliminary hydrogeological investigation letter report is being issued under separate cover.

8.2 Lateral Earth Pressures

Structures subject to unbalanced earth pressures such as foundation walls, shoring systems, and other similar structures must be designed to resist lateral earth and hydraulic pressures. Based on the subsurface conditions encountered at the Site, the following soil parameters may be used for the design of the earth retaining structures:

Table 8.2 Lateral Earth Pressure Soil Parameters

Soil	Bulk Density (kN/m^3)	Angle of internal friction ϕ' (degrees)	Coefficients of Lateral Earth Pressures		
			Ka	Ko	Kp
Compact granular fill (existing or imported)	21	32	0.31	0.47	3.25
Existing sandy/clayey Fill	18	26	0.39	0.56	2.56
Peat/Soils with Organic Content	14	24	0.42	0.59	2.37
Native Soils	19	28	0.36	0.53	2.77
Weathered Shale	22	32	0.31	0.47	3.25
Intact Shale	24	Sound Rock			

If ground movements are to be limited around these structures then we recommend the use of the K_0 earth pressure coefficients instead of the Ka values.

8.3 Seismic Site Class

The 2012 OBC requires the assignment of a Seismic Site Class for calculations of earthquake design forces and the structural design based on a two percent probability of exceedance in 50 years. According to the 2012 OBC, the Seismic Site Class is a function of soil profile, and is based on the average properties of the subsurface soil strata to a depth of 30 m below the ground surface. The 2012 OBC provides the following three methods to obtain the average properties for the top 30 m of the subsoil strata:

- Average shear wave velocity
- Average Standard Penetration Test (SPT) values (uncorrected for overburden)
- Average undrained shear strength

The present geotechnical investigation comprised of boreholes with SPT 'N' value measurements that extended to a maximum depth of 22.8 m BGS.

Based on the results of the present geotechnical investigation, and the criteria listed in Table 4.1.8.4.A. of the 2012 OBC, the Seismic Site Class E can be used for preliminary design purposes.

It is recommended that the Seismic Site Class should be determined through actual shear wave velocity measurements. Multi-Channel Analyses of Surface Waves (MASW) is a relatively inexpensive and reliable method for measuring shear wave velocities. GHD can provide the recommended MASW services.

9. Site Servicing

Fill deposits were encountered in all the boreholes, which extends generally to depths greater than the expected utility installation depths. As such, the subgrade soils beneath the utility pipe are expected to consist of fills that are in a very loose to loose, or very soft to firm state. These subgrade comprised of fill deposits at the design pipe invert are not considered suitable bearing stratum for utility pipe support due to the unknown history, and variable and uncertain degree of compactness. These fill deposits extend generally to depths ranging from 7.8 to 17.4 m BGS. As such removal of these deposits may not be an option especially due to the high groundwater table. Due to the very loose to loose and/or very soft to firm state of these fill deposits/native soils, there is a potential of excessive differential settlement of the utility infrastructure. Consideration may therefore be given to using pipe materials such as High Density Polyethylene (HDPE) pipes constructed with heat-fused joints that will have more tolerance to differential settlements than the conventional concrete or plastic pipes with rubber-gasketed joints.

The bedding for trenched services should consist of unshrinkable concrete fill meeting OPSS.MUNI 1359 requirements. The pipe bedding should have a minimum thickness of 600 mm below the pipe and 300 mm above and adjacent to the pipe and should comply with the OPS Standards. Two layers of galvanized steel wire mesh should be placed in the 600 mm thick bedding, one near the bottom and one near the top in order to reduce the magnitudes of differential settlements. The thickness of the bedding may be increased, if required to cover irregularities in the bedding subgrade to assure uniform support to the pipe.

HDPE pipes are effective for drainage of hostile effluents, such as acid rain, acidic mine wastes, aggressive landfill leachates and effluents with high concentrations of road salts, fuels and motor oils. Laboratory studies indicate that only a negligible increase in abrasive wear of HDPE pipes may be expected when the pH drops from neutral (pH = 7) to medium-low acidic conditions (pH = 4).

9.1 Excavation Backfill

A relatively high-density backfill or a backfill that requires compaction effort including vibratory component may result in settlement of the utility pipes. The following options can be considered:

- Use of Cellular or Foam Concrete. Its density varies from 400 to 1,600 kg/m^3, corresponding to unit weight of 4 to 15.7 kN/m^3. With an approximately 20 percent foam content, it has good thermal insulation quality.
- The trench can be backfilled using self-compacting 19 mm clear stone or pea gravel which must be completely wrapped in geotextile filter fabric up to the groundwater level. Above the groundwater level, Geofoam can be used to backfill the trench up to the pavement subgrade levels. The typical density of Geofoam material ranges from 11 to 48 kg/m^3. Geofoam have high compressive strength and are known to have been used as pavement subgrade.

Some settlement should still be expected in areas of trench backfill. Therefore, it is recommended that as long a period of time as possible be left between backfilling the trenches and completing the surface structures such as pavements, walkways, etc.

10. Pavement Structure

10.1 Preparation

The geotechnical investigation has shown that the existing pavement structure has an average of approximately 100 mm of asphaltic material underlain by 0.8 to 1.5 m thick granular base/subbase material. Consideration therefore can be given to rehabilitating the pavement structure by replacing the existing asphaltic concrete.

After removal of the asphaltic concrete, the exposed base course should be compacted using a heavy duty smooth drum roller to minimum dry density of 100 percent standard Proctor maximum dry density (SPMDD) in the presence of the geotechnical engineer. During the compaction operations, loose and soft areas that cannot be compacted to 100 percent SPMDD should be sub-excavated to a competent stratum as directed by the geotechnical engineer, and replaced with OPSS Granular 'A' material compacted to a minimum 100 percent SPMDD in lifts not exceeding 300 mm. In-situ density testing to monitor the effectiveness of the compaction equipment in achieving the required densities is also recommended.

10.2 Pavement Structure Rehabilitation

The pavement structures can be rehabilitated using the following overlays for light duty (car parking) and heavy duty (driveways and heavy duty vehicle parking) areas:

Table 10.1 Component Thickness Pavement Rehabilitation

Pavement Layer	Compaction Requirements	Light Duty Pavement Design	Heavy Duty Pavement Design
Asphaltic Concrete HL3	92% Maximum Relative Density	40 mm	40 mm
Asphaltic Concrete HL8	92% Maximum Relative Density	60 mm	80 mm
Base/Subbase Course	Existing		

10.3 New Pavement Structure

For the new pavement structure built on the existing landscaped areas, the existing topsoil including its root zone should be stripped. The thickness of topsoil in the area is unknown as borehole investigation in the landscaped areas was not part of the scope of work. After stripping of the topsoil, the exposed subgrade should be compacted using a smooth drum roller to 98 percent SPMDD. The prepared subgrade should then be proofrolled using heavy duty earth moving equipment such as a grader or fully loaded triple–axle dump truck. Any soft or otherwise un-compactable areas identified should be sub-excavated to a competent stratum as identified by a geotechnical engineer and replaced with OPSS Granular ‘B’ material compacted to 98 percent SPMDD.

If raising of the grades is required, imported select subgrade material can be used, placed in thin lifts up to 300 mm in loose thickness, and compacted to 98 percent SPMDD. The pavement subgrade should be free of depressions and sloped at a gradient of 2 percent towards the edges. The following pavement structure is recommended for new light and heavy duty areas:

Table 10.2 Component Thickness Pavement Structure

Pavement Layer	Compaction Requirements	Light Duty Pavement Design	Heavy Duty Pavement Design
Asphaltic Concrete HL3	92% Maximum Relative Density	40 mm	40 mm
Asphaltic Concrete HL8	92% Maximum Relative Density	60 mm	80 mm
OPSS Granular 'A'/19 mm Crusher Run Limestone Base Course	100% SPMDD	150	150
OPSS Granular 'B'/ 50 mm Crusher Run Limestone Subbase	100% SPMDD	150	300

10.4 Pavement Grade Changes

Where the proposed asphalt surface elevations are lower than the existing surface, the existing pavement structure should be completely removed to accommodate the design pavement structure as proposed above.

Where the new asphalt surface will be higher than the existing asphalt surface, the existing asphaltic concrete should be removed, the grades can then be raised by adding OPSS Granular 'A' thickness on as required basis, and compacting to 100 percent SPMDD. Prior to raising grades, the surface of the existing granular should be proof-rolled using a loaded tandem truck, and any areas of significant deflection should be sub-excavated and replaced with OPSS Granular 'A' compacted to 100 percent SPMDD. Also, that entire surface should be compacted to a minimum of 100 percent SPMDD. The finished grades can be paved with the above recommended Hot-Laid HL8 and HL3 binder and wearing asphaltic concrete thicknesses, respectively.

10.5 Re-Use of Existing Granular Material

The split spoon or auger cutting samples collected, due to the limitations imposed by the sampler size and sampling methods are not representative of the grain size distribution of granular materials. Bulk samples obtained from test pits are better representatives of the grain-size distribution. It is therefore recommended that a test pit investigation should be carried out across the parking lot to determine grain size distribution of the existing parking lot granular materials and their compliance with OPSS Granular 'A' and Granular 'B' gradation specifications.

10.6 Drainage

The efficient drainage of water infiltrating through the pavement layers is essential for a durable pavement structure. Grading adjacent to pavement areas should be designed so that water is not allowed to pond adjacent to the outside edges of the pavement. The pavement subgrade should be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward the edge of pavement edge ditches or catch basins. For new pavement structure, depending on the area, storm water collection system comprised of catch basins should be

provided. Four stub drains, draining into the catch basin on each of its four sides should be provided at the subgrade level. The stub drains comprised of 100 mm diameter perforated PVC pipes should be at least 3 m long. Each pipe must be wrapped in a filter cloth with upstream end of the pipe capped. The pipe should be surrounded with 150 mm thick 19 mm clear stone also wrapped completely in filter cloth.

The finished pavement surface should be crowned to promote efficient cross-drainage toward the catch basins. Water should not be allowed to pond on the pavement surface or along its edges.

11. Construction Monitoring

The foundation installations must be monitored and evaluated by qualified personnel to ensure that the founding achieved is consistent with the design bearing intended by the geotechnical engineer. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the 2012 Ontario Building Code.

12. Limitations of the Report

This report is intended solely for Rankin Construction Inc. (Client) and other parties explicitly identified in the report and are prohibited for use by others without GHD's prior written consent. This report is considered GHD's professional work product and shall remain the sole property of GHD. Any unauthorized reuse, redistribution of or reliance on the report shall be at the Client and recipient's sole risk, without liability to GHD. Client shall defend, indemnify and hold GHD harmless from any liability arising from or related to Client's unauthorized distribution of the report. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The recommendations made in this report are in accordance with our present understanding of the project, the current site use, ground surface elevations and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of geotechnical engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our subsurface investigation and resulting understanding of the project, as defined at the time of the study. We should be retained to review our recommendations when the drawings and specifications are complete. Without this review, GHD will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

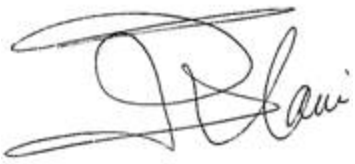
By issuing this report, GHD is the geotechnical engineer of record. It is recommended that GHD be retained during all earthwork operations to confirm the conditions of the subsoil are actually similar

to those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of our study is correctly carried forward to the construction phases.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments included in this report are based on the results obtained at the test locations only (five boreholes BH1-18 to BH5-18). The subsurface conditions confirmed at the five test locations may vary at other locations. The subsurface conditions can also be significantly modified by the construction activities on site (excavation, dewatering and drainage, blasting, pile driving, etc.). These conditions can also be modified by exposure of soils or bedrock to humidity, dry periods or frost. Soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction which could not be detected or anticipated at the time of our investigation. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by GHD is completed.

All of Which is Respectfully Submitted,

GHD Limited

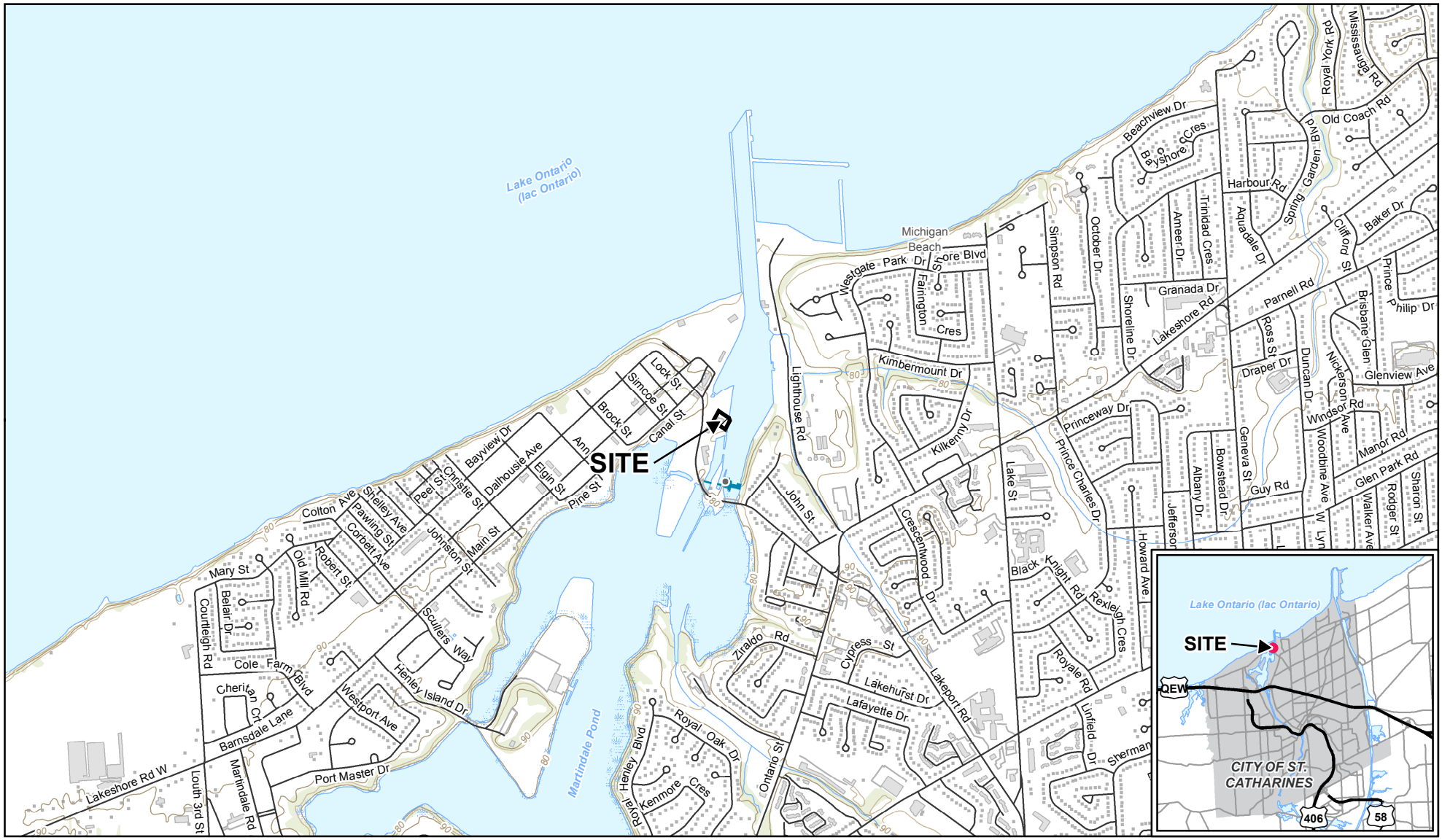


Hassan Gilani, M.Sc., P. Eng.



Wayne Russell, P. Eng.

Figures



Source: MNRF NRVIS, 2018. Produced by GHD under licence from Ontario Ministry of Natural Resources and Forestry, © Queen's Printer 2018

0 200 400 600
Meters

Coordinate System:
NAD 1983 UTM Zone 17N

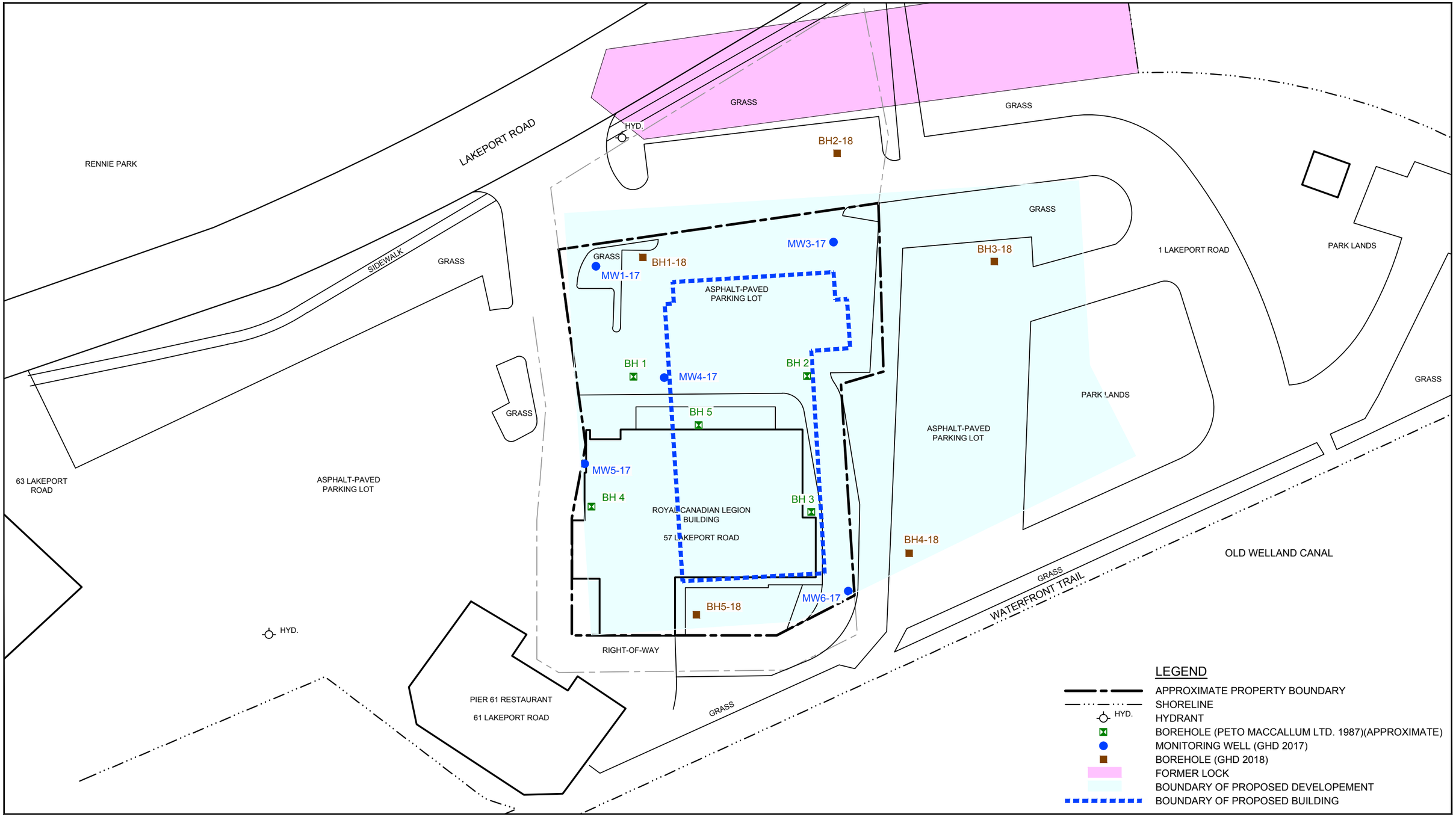
RANKIN CONSTRUCTION INC.
57 LAKEPORT ROAD, ST. CATHARINES, ONTARIO
GEOTECHNICAL INVESTIGATION

11131634-40
Nov 9, 2018

SITE LOCATION MAP

FIGURE 1

GIS File: Q:\GIS\PROJECTS\11131000s\11131634\Layouts\005\11131634-40(005)GIS-OT001.mxd



Source: Peto MacCallum Ltd. Borehole location Plan, 87 F 351, Aug. 1987;Raimondo + Associates Overall Site Plan, A1-100 8/10/2018

0 5 10 15m

RANKIN CONSTRUCTION INC.
57 LAKEPORT ROAD, ST. CATHARINES, ONTARIO
GEOTECHNICAL INVESTIGATION

BOREHOLE LOCATION PLAN

11131634-40

Nov 16, 2018

FIGURE 2

CAD File: C:\Users\skeall\Desktop\11131634-40(005)GN-SC001-edit.dwg

Appendices

Appendix A
Borehole Logs

Notes on Borehole and Test Pit Reports

Soil description :

Each subsurface stratum is described using the following terminology. The relative density of granular soils is determined by the Standard Penetration Index ("N" value), while the consistency of clayey sols is measured by the value of undrained shear strength (Cu).

Classification (Unified system)

Clay	< 0.002 mm		
Silt	0.002 to 0.075 mm		
Sand	0.075 to 4.75 mm	fine	0.075 to 4.25 mm
		medium	0.425 to 2.0 mm
		coarse	2.0 to 4.75 mm
Gravel	4.75 to 75 mm	fine	4.75 to 19 mm
		coarse	19 to 75 mm
Cobbles	75 to 300 mm		
Boulders	>300 mm		

Terminology

"trace"	1-10%
"some"	10-20%
adjective (silty, sandy)	20-35%
"and"	35-50%

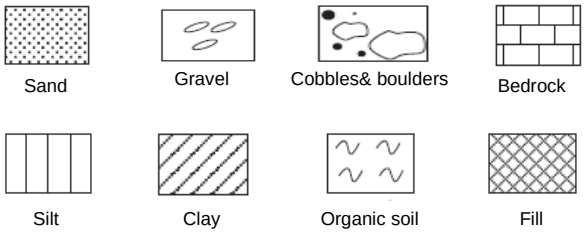
Relative density of granular soils	Standard penetration index "N" value (BLOWS/ft – 300 mm)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Consistency of cohesive soils	Undrained shear strength (Cu) (P.S.F)	(kPa)
Very soft	<250	<12
Soft	250-500	12-25
Firm	500-1000	25-50
Stiff	1000-2000	50-100
Very stiff	2000-4000	100-200
Hard	>4000	>200

Rock quality designation

"RQD" (%) Value	Quality
<25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
>90	Excellent

STRATIGRAPHIC LEGEND



Samples:

Type and Number

The type of sample recovered is shown on the log by the abbreviation listed hereafter. The numbering of samples is sequential for each type of sample.

SS: Split spoon
SSE, GSE, AGE: Environmental sampling
ST: Shelby tube
PS: Piston sample (Osterberg)
AG: Auger
RC: Rock core
GS: Grab sample

Recovery

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil

RQD

The "Rock Quality Designation" or "RQD" value, expressed as percentage, is the ratio of the total length of all core fragments of 4 inches (10 cm) or more to the total length of the run.

IN-SITU TESTS:

N: Standard penetration index
R: Refusal to penetration
N_c: Dynamic cone penetration index
Cu: Undrained shear strength
Pr: Pressure meter
k: Permeability
ABS: Absorption (Packer test)

LABORATORY TESTS:

I_p: Plasticity index
W_l: Liquid limit
Wp: Plastic limit
H: Hydrometer analysis
GSA: Grain size analysis
A: Atterberg limits
w: Water content
γ: Unit weight
C: Consolidation
CS: Swedish fall cone
CHEM: Chemical analysis
O.V.: Organic vapor

REFERENCE No.: 11131634-40 ENCLOSURE No.: 1

BOREHOLE No.: BH1-18
ELEVATION: 78.82 m

BOREHOLE REPORT

Page: 1 of 2

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: S. Ahmad CHECKED BY: W. Russell
DATE (START): 1 October 2018 DATE (FINISH): 2 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery %	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index N
		78.82		GROUND SURFACE						
0	0.13	78.69		BITUMINOUS MATERIAL : 125 mm						
1				GRANULAR MATERIAL : SAND and GRAVEL		GS		--	--	--
2	0.81	78.01								
3	1.0			FILL : SILT, some clay, brown, moist		SS1	83	17	3-3-2-3	5
4										
5										
6						SS2	83	19	2-1-2-1	3
7	2.0									
8				increased clay content		SS3	83	20	1-1-1-1	2
9										
10	3.0									
	3.05	75.77		SILTY CLAY, trace sand, grey, moist		SS4	83	19	0-1-1-2	2
11										
12										
13	4.0									
14										
15										
16	5.0									
17										
18										
19										
20	6.0									
	6.10	72.72		POSSIBLE PEAT, some clay, trace sand, trace organics and rootlets, trace wood fragments, grey, moist		SS5	100	37	1-1-1-2	2
21										
22										
23	7.0									
24										
25										
26	8.0									
27										
28										
29										
30	9.0									
	9.15	69.67		POSSIBLE NATIVE : SILT, some clay, brownish grey, moist, dense		SS6	79	25	12-17-32-50/ 125mm	49
31	9.43	69.39								
32				SHALE, some clay, highly weathered, red						
33	10.0									
34										
35						SS7	100	10	50/ 100mm	100
	10.77	68.05								
36	11.0			SHALE-BEDROCK (QUEENSTON FORMATION), red-shale/siltstone interbedded with occasional layers (approximately 50-150mm thick) of grey/bleuish grey limestone/siltstone, fime grained, medium to thicky bedded, laminated, (fizzed when exposed to muriatic acid, possibly						
37						RC1	93	--	57	--
38										
39										
40	12.0									

Shear test (Cu) △ Field
Sensitivity (S) ☐ Lab
○ Water content (%)
w_p w_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 1

BOREHOLE No.: BH1-18
ELEVATION: 78.82 m

BOREHOLE REPORT

Page: 2 of 2

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: S. Ahmad CHECKED BY: W. Russell
DATE (START): 1 October 2018 DATE (FINISH): 2 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery %	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index N	Shear test (Cu) △ Field, Sensitivity (S) ☐ Lab, ○ Water content (%), w_p w_l Atterberg limits (%), ● "N" Value (blows / 12 in.-30 cm) 10 20 30 40 50 60 70 80 90
		78.82		GROUND SURFACE							
42	13.0			limestone/dolostone		RC2	100	--	51	--	
46	14.0					RC3	100	--	43	--	
49	15.0										
52	16.0					RC4	100	--	72	--	
54	16.54	62.28									
56	17.0			**END OF BOREHOLE :**							
59	18.0			**NOTE :** - End of Borehole at 16.54 m bgs - Rock coring from 10.77 m bgs - bgs denotes 'below ground surface'							
62	19.0										
66	20.0										
69	21.0										
72	22.0										
75	23.0										
79	24.0										
81											

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 2

BOREHOLE No.: BH2-18
ELEVATION: 78.97 m

BOREHOLE REPORT

Page: 1 of 2

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: M. Pley CHECKED BY: W. Russell
DATE (START): 28 September 2018 DATE (FINISH): 28 September 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery %	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index N
		78.97		GROUND SURFACE						
0	0.06	78.91		BITUMINOUS MATERIAL : 60 mm						
1				GRANULAR MATERIAL : SAND and GRAVEL						
3	1.0									
4	1.37	77.60		CLAYEY SILT, trace gravel, greyish brown, moist to wet		GS1		20	--	--
6	2.0									
9										
10	3.0			saturated		GS2		24	--	--
12										
13	4.0									
15				moist		SS1	100	13	8-12-13	25
16	5.0									
18										
19										
20	6.0									
	6.10	72.87		SILTY SAND, trace to some gravel, reddish brown, wet		SS2	100	14	17-21-23-26	44
23	7.0									
25										
	7.77	71.20		NATIVE : CLAYEY SILT, grey, wet, compact		SS3	100	18	11-11-12-13	23
26	8.0									
28										
29										
30	9.0									
	9.15	69.82		SILTY SAND, brown, saturated, compact		SS4	100	23	11-12-12-14	24
31										
33	10.0									
34										
35	10.67	68.30				SS5	67	11	50	50
	10.83	68.14		SHALE, some clay, highly weathered, red						
36	11.0									
37				SHALE-BEDROCK (QUEENSTON FORMATION)						
38										
39	12.0									
40	12.35	66.62				SS6	83	11	50	50

Shear test (Cu) △ Field
Sensitivity (S) □ Lab
○ Water content (%)
W_p W_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 2

BOREHOLE No.: BH2-18
ELEVATION: 78.97 m

BOREHOLE REPORT

Page: 2 of 2

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: M. Pley CHECKED BY: W. Russell
DATE (START): 28 September 2018 DATE (FINISH): 28 September 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index
		78.97		GROUND SURFACE			%			N
42	13.0			**END OF BOREHOLE :**						
43				**NOTE :**						
44				- End of Borehole at 12.35 m bgs						
45				- Water level at 5.94 m bgs upon completion						
46	14.0			- Borehole caved at 12.2 m bgs						
47				- bgs denotes 'below ground surface'						
48										
49	15.0									
50										
51										
52	16.0									
53										
54										
55										
56	17.0									
57										
58										
59	18.0									
60										
61										
62	19.0									
63										
64										
65										
66	20.0									
67										
68										
69	21.0									
70										
71										
72	22.0									
73										
74										
75	23.0									
76										
77										
78										
79	24.0									
80										
81										

Shear test (Cu) △ Field
Sensitivity (S) □ Lab
○ Water content (%)
w_p w_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 3

BOREHOLE No.: BH3-18
ELEVATION: 77.67 m

BOREHOLE REPORT

Page: 1 of 2

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: S. Ahmad CHECKED BY: W. Russell
DATE (START): 3 October 2018 DATE (FINISH): 3 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery %	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index N
		77.67		GROUND SURFACE						
0	0.13	77.54		BITUMINOUS MATERIAL : 125 mm						
1				GRANULAR MATERIAL : SAND and GRAVEL		GS		--	--	--
2										
3	1.0			shale fragments, grey/brown, moist		SS1	79	11	46-35-20-8	55
4										
5				wood pieces (only), trace gravel						
6						SS2	8	17	8-7-3-5	10
7	2.0									
8				wood and clay (cuttings collected from auger)						
9						SS3	0	26	2-1-1-1	2
10	3.0									
	3.05	74.62		FILL :						
11				SILTY CLAY, trace sand, grey, moist		SS4	58	14	1-3-4-4	7
12										
13	4.0									
14										
15										
16	5.0									
17										
18										
19										
20	6.0			wood fragments, organics, dark/grey, saturated		SS5	100	43	1-1-1-2	2
21										
22										
23	7.0									
24										
25										
26	8.0									
27										
28										
29										
30	9.0			trace wood fragments						
31						SS6	100	42	2-2-3-5	5
32										
33	10.0									
34										
35	10.67	67.00								
36	11.0			POSSIBLE PEAT, organics, dark grey		SS7	75	38	0-3-3-4	6
37										
38										
39										
40	12.0									
	12.20	65.47		POSSIBLE NATIVE :						

Shear test (Cu) △ Field
Sensitivity (S) □ Lab
○ Water content (%)
w_p w_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 3

BOREHOLE No.: BH3-18
ELEVATION: 77.67 m

BOREHOLE REPORT

Page: 2 of 2

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: S. Ahmad CHECKED BY: W. Russell
DATE (START): 3 October 2018 DATE (FINISH): 3 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery %	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index N
		77.67		GROUND SURFACE						
42	13.0			SILTY CLAY, grey, moist to wet, very soft		SS8	42	27	0-0-2-3	2
60	18.29	59.38		SHALE-BEDROCK, weathered, reddish grey		SS9	100	16	50/.125mm	100
61	18.42	59.25								

Shear test (Cu) △ Field
Sensitivity (S) □ Lab
○ Water content (%)
w_p w_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

END OF BOREHOLE :

NOTE :
- End of Borehole at 18.42 m bgs
- Water level found at 2.44 m bgs upon completion
- bgs denotes 'below ground surface'

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 4

BOREHOLE No.: BH4-18
ELEVATION: 76.10 m

BOREHOLE REPORT

Page: 1 of 2

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: S. Ahmad CHECKED BY: W. Russell
DATE (START): 3 October 2018 DATE (FINISH): 3 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery %	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index N
0		76.10		GROUND SURFACE						
0	0.10	76.00		BITUMINOUS MATERIAL : 100 mm						
1				GRANULAR MATERIAL : SAND and GRAVEL						
3	1.0									
5	1.52	74.58		FILL : SILTY CLAY, moist to wet						
7	2.0									
10	3.0									
13	4.0									
16	5.0									
20	6.0									
23	7.0									
26	8.0									
29	9.0									
30	9.15	66.95		POSSIBLE NATIVE : SILTY CLAY, grey, wet to saturated						
33	10.0									
36	11.0									
39	12.0									

Shear test (Cu) △ Field
Sensitivity (S) □ Lab
○ Water content (%)
W_p W_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

▼

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 4

BOREHOLE No.: BH4-18
ELEVATION: 76.10 m

BOREHOLE REPORT

Page: 2 of 2

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: S. Ahmad CHECKED BY: W. Russell
DATE (START): 3 October 2018 DATE (FINISH): 3 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index
		76.10		GROUND SURFACE			%			N
	13.0									
	14.0									
	15.0									
	16.0									
	17.0									
	17.53	58.57		SHALE-BEDROCK auger refusal						
	17.60	58.50								
	18.0			**END OF BOREHOLE :** **NOTE :** - End of Borehole at 17.53 m bgs due to inferred bedrock - Water level at 4.57 m bgs upon completion of drilling - Borehole caved at 15.24 m bgs - bgs denotes 'below ground surface'						
	19.0									
	20.0									
	21.0									
	22.0									
	23.0									
	24.0									

Shear test (Cu) △ Field
Sensitivity (S) □ Lab
○ Water content (%)
w_p w_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 5

BOREHOLE No.: BH5-18
ELEVATION: 76.82 m

BOREHOLE REPORT

Page: 1 of 3

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: M. Pley/S. Ahmad CHECKED BY: W. Russell
DATE (START): 28 September 2018 DATE (FINISH): 3 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery %	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index N
		76.82		GROUND SURFACE						
0	0.06	76.76		BITUMINOUS MATERIAL : 60 mm						
1				GRANULAR MATERIAL : SAND and GRAVEL, moist						
2										
3	1.0					GS1		9	--	--
4										
5	1.52	75.30		CLAYEY SILT, trace gravel, organics, low plasticity, dark brown, moist		GS2		15	--	--
6	1.98									
7	2.0	74.84		SANDY SILT, some gravel, reddish brown, moist						
8										
9										
10	3.0			moist to wet		SS1	45	13	2-1-1-2	2
11										
12										
13	4.0									
14										
15				trace gravel, higher clay content, grey layering		SS2	50	13	0-1-1-2	2
16										
17	5.0									
18										
19										
20	6.0									
21	6.10	70.72		SILTY CLAY, grey, moist		SS3	75	43	1-1-2-3	3
22	6.40	70.42		SILTY SAND, wood fragments, grey, saturated						
23	7.0									
24										
25				some clay		SS4	77	34	0-1-1-2	2
26	8.0									
27										
28										
29	9.0									
30				brick fragments		SS5	38	18	0-3-3-3	6
31										
32										
33	10.0									
34										
35										
36	11.0					SS6	69	25	3-3-3-1	6
37										
38										
39	12.0									
40										

Shear test (Cu) △ Field
Sensitivity (S) □ Lab
○ Water content (%)
W_p W_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 5

BOREHOLE No.: BH5-18
ELEVATION: 76.82 m

BOREHOLE REPORT

Page: 2 of 3

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: M. Pley/S. Ahmad CHECKED BY: W. Russell
DATE (START): 28 September 2018 DATE (FINISH): 3 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery %	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index N
		76.82		GROUND SURFACE						
						SS7	44	18	3-5-5-6	10
	13.0									
						SS8	48	17	3-6-6-6	12
	14.0									
	15.0									
	16.0									
	17.0									
	17.38	59.44		SHALE, highly weathered						
	18.0									
						SS9	38	11	7-10-50	60
	18.75	58.07								
	19.0			SHALE-BEDROCK (QUEENSTON FORMATION)		RC1	7	--	0	--
	19.77	57.05								
	20.0			SHALE-BEDROCK (QUEENSTON FORMATION), redstone/siltstone interbedded with occasional layers (approximately 125-500mm thick) of grey/bleuish grey limestone/siltstone, fine grained, medium to thicky bedded, laminated, (fizzed when exposed to muriatic acid, possibly limestone/dolostone		RC2	100	--	72	--
	21.0									
	21.29	55.53								
	22.0			SHALE-BEDROCK (QUEENSTON FORMATION), redstone/siltstone interbedded with occasional layers (approximately 125-500mm thick) of grey/bleuish grey limestone/siltstone, fine grained, medium to thicky bedded, laminated, (fizzed when exposed to muriatic acid, possibly limestone/dolostone		RC3	100	--	71	--
	22.82	54.00								
	23.0			**END OF BOREHOLE :**						
	24.0			**NOTE :** - End of Borehole at 22.82 m bgs						

Shear test (Cu) △ Field ☐ Lab
Sensitivity (S)
○ Water content (%)
w_p w_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

REFERENCE No.: 11131634-40 ENCLOSURE No.: 5

BOREHOLE No.: BH5-18
ELEVATION: 76.82 m

BOREHOLE REPORT

Page: 3 of 3

CLIENT: Rankin Construction Inc.
PROJECT: Port Dalhousie Legion Redevelopment.
LOCATION: 57 Lakeport Road, St. Catharines, Ontario
DESCRIBED BY: M. Pley/S. Ahmad CHECKED BY: W. Russell
DATE (START): 28 September 2018 DATE (FINISH): 3 October 2018

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Moisture Content	Blows per 6 in. / 15 cm or RQD	Penetraion Index
		76.82		GROUND SURFACE			%			N
83				- Rock coring from 18.75 m bgs - Water level at 3.05 m bgs upon completion of drilling - bgs denotes 'below ground surface'						
84										
85	26.0									
86										
87										
88	27.0									
89										
90										
91										
92	28.0									
93										
94										
95	29.0									
96										
97										
98	30.0									
99										
100										
101										
102	31.0									
103										
104										
105	32.0									
106										
107										
108	33.0									
109										
110										
111	34.0									
112										
113										
114										
115	35.0									
116										
117										
118	36.0									
119										
120										
121	37.0									
122										

Shear test (Cu) △ Field
Sensitivity (S) □ Lab
○ Water content (%)
w_p w_l Atterberg limits (%)
● "N" Value (blows / 12 in.-30 cm)

10 20 30 40 50 60 70 80 90

SOIL LOG WITH GRAPH+WELL 11131634.GPJ GHD_Geotechnical 22/11/18

Appendix B
Rock Core Photo Log



Photo 1 - BH1-18: Run 1 10.5 m – 12 m Run 2 12 m – 13.5 m



Photo 2 - BH1-18: Run 3 13.5 m – 15 m Run 4 15 m – 16.5 m

Bedrock Core Photographs

Geotechnical Investigation

57 Lakeport Road, St. Catharines, Ontario



Photo 3 - BH5-18: Run 2 19.8 m – 21.3 m Run 3 21.3 m – 22.8 m

Appendix C
Geotechnical Laboratory Test Results

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 - Method D)

CLIENT:	Rankin Construction Inc.	**LAB No.:**	WLT 57-1
PROJECT/ SITE:	Port Dalhousie Legion Redevelopment / 57 Lakeport Road, St. Catharines, Ontario	**PROJECT No.:**	11131634-40

Borehole No.:	BH 5-18	**Sampled ID:**	Run 2
Depth:	68'1.5"-68'6" (20.76 m - 20.88 m)	**Date Sampled:**	October 3, 2018

Lithological Description: Shale

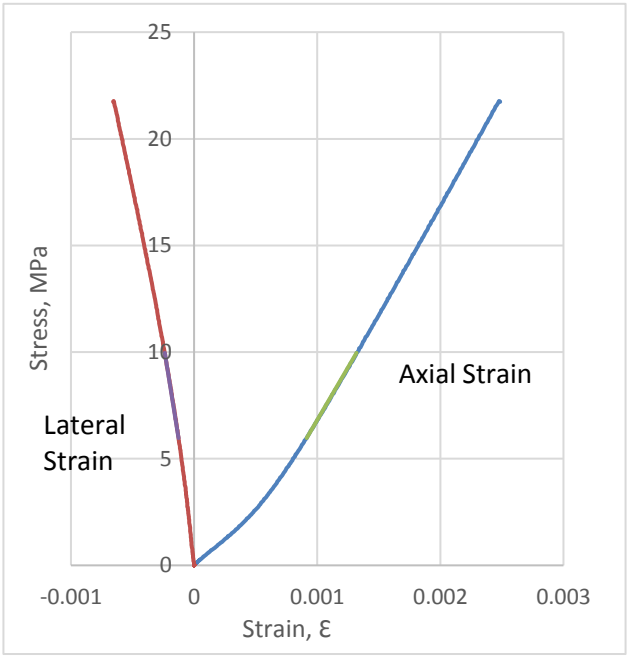
Specimen Parameters	
Diameter, cm	5.04
Length, cm	11.22
Height-to-Diameter Ratio	2.2
Bulk Density, kg/m^3	2628
Moisture Condition	As Received
Moisture Content, %	-

Uniaxial Compressive Strength, MPa	**54.8**
Young's Modulus, *E*, GPa *1	**9.9**
Poisson's Ratio *1	**0.27**

*1: Tangent modulus measured in the range of 6 MPa and 10 MPa







REMARKS:

PERFORMED BY: M. Reddy **DATE:** October 15, 2018

VERIFIED BY: Michael Braverman **DATE:** October 16, 2018

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 - Method D)

CLIENT:	Rankin Contruction Inc.	**LAB No.:**	WLT 57-2
PROJECT/ SITE:	Port Dalhousie Legion Redevelopment / 57 Lakeport Road, St. Catharines, Ontario	**PROJECT No.:**	11131634-40

Borehole No.:	BH 5-18	**Sampled ID:**	Run 3
Depth:	72'11"-74'0.5" (22.23 m - 22.57 m)	**Date Sampled:**	October 3, 2018

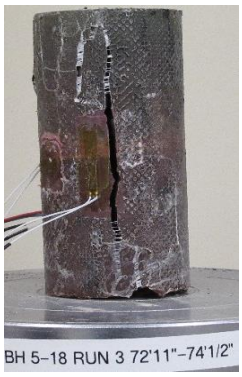
Lithological Description: Shale

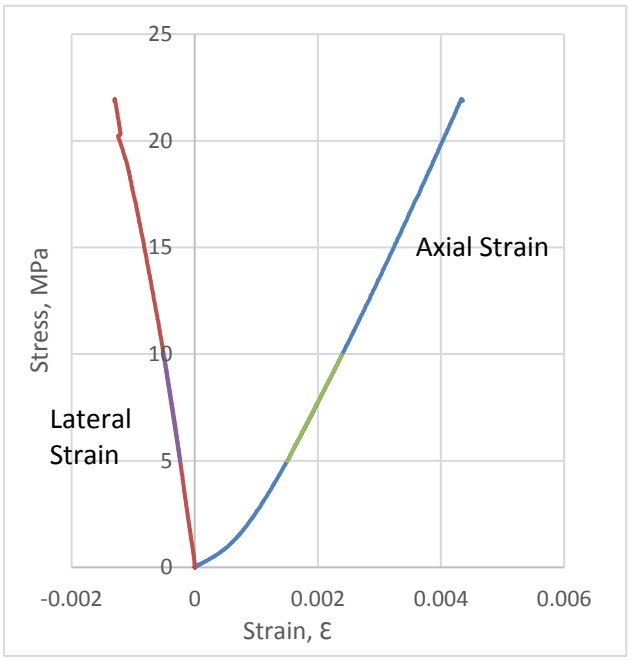
Specimen Parameters	
Diameter, cm	5.03
Length, cm	10.86
Height-to-Diameter Ratio	2.2
Bulk Density, kg/m^3	2632
Moisture Condition	As Received
Moisture Content, %	-

Uniaxial Compressive Strength, MPa	**32.4**
Young's Modulus, *E*, GPa *1	**5.6**
Poisson's Ratio *1	**0.30**

*1: Tangent modulus measured in the range of 5 MPa and 9.9 MPa







REMARKS:

PERFORMED BY:	M. Reddy	**DATE:**	October 15, 2018
VERIFIED BY:	Michael Braverman	**DATE:**	October 16, 2018

Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (ASTM D7012 - Method D)

CLIENT:	Rankin Construction Inc.	**LAB No.:**	WLT 57-3
PROJECT/ SITE:	Port Dalhousie Legion Redevelopment / 57 Lakeport Road, St. Catharines, Ontario	**PROJECT No.:**	11131634-40

Borehole No.:	BH 1-18	**Sampled ID:**	Run 4
Depth:	53'-53'6.5" (16.15 m - 16.32 m)	**Date Sampled:**	October 2, 2018

Lithological Description: Shale

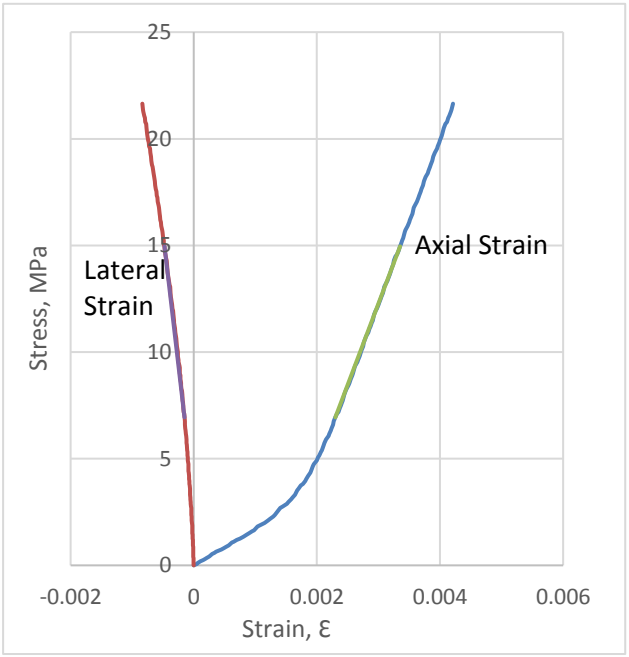
Specimen Parameters	
Diameter, cm	5.02
Length, cm	8.00
Height-to-Diameter Ratio	1.6
Bulk Density, kg/m^3	2683
Moisture Condition	As Received
Moisture Content, %	2.4

Uniaxial Compressive Strength, MPa	**49.9**
Young's Modulus, *E*, GPa *1	**7.6**
Poisson's Ratio *1	**0.31**

*1: Tangent modulus measured in the range of 6.9 MPa and 15 MPa







REMARKS: Not compliant with ASTM standards (Height-to-Diameter Ratio 1.6 < 2.0)

PERFORMED BY:	M.J. Hajizadeh	**DATE:**	October 24, 2018
VERIFIED BY:	Michael Braverman	**DATE:**	October 25, 2018



about GHD

GHD is one of the world's leading professional services companies operating in the global markets of water, energy and resources, environment, property and buildings, and transportation. We provide engineering, environmental, and construction services to private and public sector clients.

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