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# Geotechnical Investigation Report
Proposed Shell Service Station
5 Orchard Drive
Ottawa, Ontario

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Submitted to:

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# Geotechnical Investigation Report
# Proposed Shell Service Station
# 5 Orchard Drive
Ottawa, Ontario

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# 1.0 INTRODUCTION

## 1.1 General

This report presents the results of a geotechnical investigation carried out for the design and construction of a new Shell service station to be located at 5 Orchard Drive in Ottawa, Ontario (refer to Borehole Location Plan, Figure 1). The purpose of the geotechnical investigation was to identify the general subsurface conditions at the site by means of a limited number of boreholes, and based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

## 1.2 Project and Site Description

Plans are being prepared to develop a vacant parcel of land located at the southwest corner of Hazeldean Road and Fringewood Drive in Ottawa (Stittsville), Ontario. Based on available property information from the City of Ottawa, the civic address for the proposed Shell site is 5 Orchard Drive, Ottawa.

Based on the information provided to us, the proposed structures will include a 168 square metre convenience store, a 97 square metre carwash, a pump island on a 240 square metre concrete apron with a 198 square metre canopy, access roadway and parking areas, and two (2) underground fuel storage tanks. It is anticipated that all of the structures will be of slab on grade (i.e. basementless) construction. The founding depth of the fuel storage tanks were not provided to us; however, based on our past experience, it is anticipated that the tanks will be founded at about 4.5 metres below finished grade. Similarly, it is anticipated that the pad footings for the canopy may be founded at depths between 2.5 and 4.5 metres.

# 2.0 SUBSURFACE INVESTIGATION

The fieldwork for this investigation was carried out on June 4th, 2019. At that time, three (3) boreholes were advanced across the property. The boreholes were advanced using a track mounted drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville-Sur-La-Rouge, Quebec. Details of the boreholes are provided below:

- Borehole BH19-1 was advanced to practical refusal of the auger at a depth of about 3.4 metres below ground surface in the area of the convenience store and car wash.

- Borehole BH19-2 was advanced to practical refusal of the auger at a depth of about 3.7 metres below ground surface in the area of the pump island and canopy. The bedrock was then cored from the bottom of the borehole to a depth of about 5.3 metres below ground surface using HQ size coring equipment.

- Borehole MW19-1 was advanced to practical refusal of the auger at a depth of about 2.9 metres below ground surface in the area of the underground fuel storage tanks. The bedrock was then cored from the bottom of the borehole to a depth of about 5.4 metres below ground surface using HQ size coring equipment. A well screen was installed in the borehole to facilitate hydraulic conductivity testing and to measure the stabilized groundwater level.

As part of Shell's health and safety policy, the following precautions were undertaken prior to advancing the boreholes at the site:

- The boreholes were daylighted to depths of about 1.5 and 2.0 metres below ground surface prior to starting the drilling operation.

The fieldwork was observed by members of our engineering staff who directed the drilling and hydro-vacuuming operations, observed the in situ testing and logged the samples and boreholes. Standard penetration tests were carried out within the overburden deposits and samples of the soils encountered were recovered using drive open sampling equipment. At boreholes MW19-1 and BH19-2, the encountered bedrock was cored using HQ size bedrock coring equipment. A well screen was sealed in the bedrock at the location of MW19-1.

A sample of the soil recovered from borehole BH19-1 was sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

Following the borehole drilling work, the soil and bedrock samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content and grain size distribution. A sample of the bedrock was tested for unconfined compressive strength. A hydraulic conductivity test was undertaken within the well screen installed in MW19-1 on June 13, 2019.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The results of the laboratory classification testing on the soil are also provided in Appendix A. A photo of the bedrock core samples recovered is provided on Figure B1 in Appendix B. The results of the hydraulic testing are provided in Appendix C. The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 1.

The borehole locations were selected by AECOM Canada Ltd. (AECOM) and GEMTEC Consulting Engineers and Scientists Limited (GEMTEC), and positioned at the site by GEMTEC personnel relative to existing site features. Elevations were measured using our Trimble R10 GPS equipment and are referenced to geodetic datum CGVD28.

## 3.0 SUBSURFACE CONDITIONS

### 3.1 General

As previously indicated, the subsurface conditions identified in the boreholes are given on the Record of Borehole sheets in Appendix A. The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of drilling, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the test locations may vary from the conditions encountered in the boreholes. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities in the area.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and GEMTEC does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered in the boreholes advanced during this investigation.

### 3.2 Topsoil

The surface grade at the borehole locations consists of dark brown clayey silt topsoil. The thickness of the topsoil soil is about 150 and 200 millimetres at the borehole locations.

The moisture content of the topsoil samples from boreholes BH19-1 and BH19-2 are 31 and 34 percent, respectively.

### 3.3 Silt

A deposit of brown silt with some clay and trace sand was encountered below the topsoil at all of the borehole locations. The silt has a thickness of about 0.8 metres and extends to a depth of about 0.9 metres below surface grade at the borehole locations.

The SPT N values recorded within the silt range from 3 to 5 blows per 0.3 metres of penetration, which reflects a very loose to loose relative density.

The results of a grain size distribution test on a sample of the silt from borehole BH19-1 are provided on the Soils Grading Chart in Appendix A and summarized in Table 3.1.

**Table 3.1 – Summary of Grain Size Distribution Testing (Silt)**

| Location | Sample Number | Sample Depth (metres) | Gravel (%) | Sand (%) | Silt (%) | Clay (%) |
|---|---|---|---|---|---|---|
| BH19-1 | 1B | 0.3 – 0.6 | 0 | 8 | 72 | 20 |

The moisture content of the silt samples from boreholes BH19-1 and BH19-2 range from 26 to 28 percent.

### 3.4 Glacial Till

Glacial till was encountered below the silt at all of the borehole locations at a depth of about 0.9 metres below ground surface. The thickness of the glacial till ranges from about 1.9 to 2.4 metres.

Glacial till is a heterogeneous mixture of all grain sizes. At this site, the glacial till is described as brown to grey brown gravelly silty sand with trace clay, cobbles and boulders.

The SPT N values recorded within the glacial generally range from 7 to 33 blows per 0.3 metres of penetration, which reflects a loose to dense relative density. The SPT tests that encountered practical refusal (i.e. less than 0.3 metres of penetration) reflect the presence of cobbles in the glacial till or a very dense relative density.

The results of a grain size distribution test on a sample of the glacial till from borehole MW19-1 are provided on the Soils Grading Chart in Appendix A and summarized in Table 3.2.

**Table 3.2 – Summary of Grain Size Distribution Testing (Glacial Till)**

| Location | Sample Number | Sample Depth (metres) | Gravel (%) | Sand (%) | Silt (%) | Clay (%) |
|---|---|---|---|---|---|---|
| MW19-1 | 3 | 1.2 – 1.8 | 21 | 48 | 23 | 8 |

The moisture content of the glacial till samples from all of the boreholes range from 10 to 31 percent.

### 3.5 Bedrock

Below the glacial till, fractured and weathered bedrock was encountered at depths of 2.8 to 3.3 metres below ground surface. At boreholes BH19-1 and BH19-2, the bedrock was penetrated 0.1 and 0.9 metres, respectively, with the augering equipment. Auger refusal was encountered on or within the bedrock at all of the borehole locations at depths ranging from about 2.9 to 3.7 metres below ground surface.

At boreholes MW19-1 and BH19-2, the bedrock was cored using HQ sized coring equipment. Borehole MW19-1 was cored from 2.9 to 5.4 metres below ground surface, and borehole BH19-2 was cored from 3.7 to 5.3 metres below ground surface.

The bedrock consists of moderately fractured, slightly weathered, limestone bedrock banded with shale. The solid core recovery (SCR) values range from 59 to 80 percent, and the rock quality designation (RQD) values range from 44 to 80 percent. Based on the RQD values, the bedrock quality is poor, becoming good with depth. Photographs of the collected rock cores are provided in Appendix B.

One (1) bedrock core sample was tested for unconfined compressive strength and the result is summarized in Table 3.3 below.

**Table 3.3 – Unconfined Compressive Strength of Bedrock Core – Borehole 19-102**

| Borehole | Sample No. | Depth (metres) | Unconfined Compressive Strength (MPa) |
|---|---|---|---|
| MW19-1 | RC5 | 3.2 – 3.4 | 146 |

Based on the unconfined compressive strength test results presented in Table 3.3, the bedrock strength may be classified as very strong.

### 3.6 Groundwater Levels

The groundwater level was measured in the well screen at MW19-1 on June 10, 2019, and is summarized in Table 3.4.

**Table 3.4 – Groundwater Level – June 10, 2019**

| Monitoring Well | Ground Surface Elevation (Metres, Geodetic) | Groundwater Depth (metres) | Groundwater Elevation (metres, geodetic datum) |
|---|---|---|---|
| MW19-1 | 104.0 | 1.7 | 102.3 |

It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

### 3.7 Soil Chemistry Relating to Corrosion

The chemical testing results of a soil sample recovered from borehole BH19-1 are provided in Appendix D and summarized in Table 3.5.

**Table 3.5 – Summary of Corrosion Testing - Soil**

| Parameters | Borehole BH19-1<br>SA3 |
|---|---|
| Chloride Content (µg/g dry) | 34 |
| Resistivity (Ohm.m) | 61.9 |
| pH | 7.88 |
| Sulphate Content (µg/g dry) | 7 |

## 4.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

### 4.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from off-site sources are outside the terms of reference for this report.

### 4.2 Overburden Excavation

It is anticipated that the excavation for the proposed building, fuel storage tanks, and pump island canopy will be carried out through the topsoil, and native deposits of silt, glacial till, and bedrock. The sides of the excavations within overburden soils should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the soils at this site can be classified as Type 3 soils. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation side slopes in the overburden.

## 4.3 Bedrock Excavation

Based on the results of the boreholes, limestone bedrock interbedded with shale may be encountered during the excavation of the fuel storage tanks and pump island canopy.

Localized bedrock removal at this site could be carried out using hoe ramming techniques in conjunction with line drilling on close centres. The vibration effects of hoe ramming are usually minor and localized.

It is noted, based on observations during drilling and local experience, that the bedrock may contain horizontal bedding planes and near vertical joints. Therefore, some horizontal and vertical overbreak should be expected. Allowance should be made for additional granular material below the fuel storage tanks and footings for the pump island canopy.

## 4.4 Groundwater Pumping

Based on the grain size distribution results for the glacial till, groundwater inflow from the overburden soil for the construction of the convenience store, car wash and pump island canopy should be controlled by pumping from filtered sumps within the excavation. Suitable detention and filtration will be required before discharging the water to any sewers.

A hydraulic conductivity (falling head) test was undertaken in the monitoring well installed in borehole MW19-1 on June 19, 2019. The well screen is sealed within the bedrock and as such, the testing provided information on the permeability of the bedrock. The results of the hydraulic conductivity testing, which are provided in Appendix C, indicate that there was insufficient recovery of the groundwater level during the test to calculate a hydraulic conductivity value (about 3 centimetres over 30 minutes), which indicates that the bedrock in the area of MW19-1 has low permeability. Therefore, significant groundwater inflow from the bedrock during the construction of the underground fuel storage tanks is not anticipated. Any groundwater inflow from the soil and bedrock should be controlled by pumping from filtered sumps within the excavation.

## 4.5 Site Grade Raise Restrictions

The subsurface conditions at this site consist of very loose to loose silt overlying compact to dense glacial till. Based on this information, there are no grade raise restrictions for the proposed development, from a geotechnical perspective.

## 4.6 Foundation Design

Based on the results of the subsurface investigation, the proposed structures could be founded on spread and pad footings bearing on undisturbed native soil. All topsoil, loose or water-softened soils encountered should be removed from the footing areas.

In areas where the underside of footing level is above the level of the native soil, or where subexcavation is required, the grade below the proposed footing could be raised with granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type I or Type II. The granular material should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. To provide adequate spread of load below the footings, the granular material should extend at least 0.3 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

The spread footing foundations should be sized using the bearing pressures provided in Table 4.1.

**Table 4.1 – Foundation Bearing Pressures**

| Subgrade Material | Geotechnical Reaction at Serviceability Limit State (kilopascals) | Factored Geotechnical Resistance at Ultimate Limit State (kilopascals) |
|---|---|---|
| Native undisturbed silt, or on a pad of engineered fill above native undisturbed silt | 100[1] | 275 |
| Native undisturbed glacial till, or on a pad of engineered fill above native undisturbed glacial till | 250[1] | 500 |
| Competent bedrock | n/a[2] | 1,000[3] |

Notes:

1. Provided that the subgrade surface and engineered fill are prepared as described in this report, the post construction total and differential settlement of the footings at SLS should be less than 25 and 15 millimetres, respectively.
2. The geotechnical reaction at SLS for 25 millimetres of settlement will be greater than the factored resistance at ULS; as such, ULS conditions will govern for footings founded directly on the competent bedrock surface.
3. The above bearing pressure assumes that all soil, and disturbed or loosened bedrock is removed from the bearing surface. Allowance should be made in the contract for concrete fill below the foundations due to vertical overbreak of the bedrock.

## 4.7 Frost Protection of the Foundations

All exterior footings in heated areas of the structure should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) footings that are located in areas that are to be cleared of snow should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation, similarly to the insulation currently in place along the existing structure. An insulation detail could be provided upon request.

If the new foundation and\or concrete slab on grade is insulated in a way that reduces heat loss towards the surrounding soil, the required earth cover over the footings should conform to that of an unheated structure (i.e. 1.8 metres).

### 4.8 Foundation Backfill and Drainage

The native deposits at this site are considered frost susceptible and should not be used as backfill against foundation walls. To avoid frost adhesion and possible heaving, the following options are provided for foundation backfilling:

- Backfill the foundations with imported, free-draining, non-frost susceptible granular material such as that meeting OPSS Granular A or Granular B Type I or II requirements, or

- Provide a suitable bond break to the surfaces of all the foundations and backfill using the fill or native soils. A suitable bond break could consist of at least 2 layers of 6-mil polyethylene sheeting.

Where the backfill will ultimately support areas of hard surfacing (roadways or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

Where areas of hard surfacing (pavement or pathways, etc.) abut the proposed structures, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible material to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the underside of footing level to the underside of the granular subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

Perimeter foundation drainage is not considered necessary for a slab on grade structure at this site, provided that the floor slab level is above the finished exterior ground surface level.

### 4.9 Slab on Grade Support (Heated Areas Only)

For predictable performance of the slab on grade for the proposed structures, the area should be stripped of topsoil to expose the underlying native soil. The subgrade surface should then be proof rolled with a 10 tonne steel drum roller (without vibration) under dry conditions. Any soft areas that are evident from the proof rolling should be subexcavated and replaced with granular material meeting OPSS Granular B Type I or II. The subgrade surfaces and the proof rolling should be observed throughout by geotechnical personnel.

The grade within the proposed building could be raised, where necessary, with granular material meeting OPSS requirements for Granular B Type I or II. The use of Granular B Type II material is preferred under wet conditions. The granular base for the proposed slab on grade should consist of at least 150 millimetres of OPSS Granular A.

The granular materials should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment.

### 4.10 Seismic Design of Proposed Structures

Based on the results of the subsurface investigation, the proposed structures will be founded on or within silt and/or glacial till deposits having a very loose to dense relative density. In accordance with the Ontario Building Code (OBC), Site Class C could be used for the seismic design of the proposed building.

In our opinion, the potential for liquefaction of the overburden soils at this site is negligible.

## 5.0 PROPOSED UNDERGROUND FUEL STORAGE TANKS

### 5.1 Excavation and Groundwater Pumping

It is understood that the service station will contain two (2) underground fuel storage tanks located within the northeast corner of the site.

Based on the investigation results, the excavation for the proposed underground storage tanks will be carried out through topsoil and native deposits of silt and glacial till, and possibly bedrock. Our comments on overburden excavation, bedrock excavation, and groundwater pumping provided in Sections 4.2 to 4.4 apply equally to the fuel storage tanks.

### 5.2 Bedding

The subbedding and bedding should conform to the tank manufacturer’s recommendations for grain size distribution and compaction requirements. All of the topsoil, disturbed soil, and soft or deleterious materials should be removed from the tank footprint.

In areas where subexcavation is required, the grade below the proposed footing could be raised with granular material meeting OPSS requirements for Granular B Type II. The granular material should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. To provide adequate spread of load below the tanks, the granular material should extend at least 0.3 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

## 5.3 Backfill

To prevent frost adhesion and possible heaving, the tanks should be backfilled with a free-draining, non-frost susceptible granular material such as OPSS Granular A, or Granular B Type II. It should be noted that the tank manufacturer's specifications for backfill material supersedes our recommendations.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

Where future landscaped areas will exist next to the proposed tanks and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed tanks, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible soil to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the maximum depth of frost penetration (i.e. 1.8 metres below ground surface). The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

For design purposes, the earth pressure parameters provided in Table 5.1 could be used to calculate the lateral earth pressure on the underground fuel storage tank.

**Table 5.1 – Backfill Earth Pressure Parameters**

| Parameter | OPSS Granular A, Granular B Type II |
|---|---|
| Material Unit Weight, $\gamma$ (kN/m$^3$) | 22 |
| Estimated Friction Angle (degrees) | 36 |
| "Active" Earth Pressure Coefficient, $K_a$, assuming horizontal backfill behind the structure | 0.26 |
| "Passive" Earth Pressure Coefficient, $K_p$, assuming horizontal backfill behind the structure | 3.85 |
| "At Rest" Earth Pressure Coefficient, $K_o$, assuming horizontal backfill behind the structure | 0.41 |

The lateral pressures due to compaction should be considered in the design. The magnitude of the compaction surcharge pressure depends on the mass and type of compaction equipment. For light, hand operated compaction equipment having a mass of approximately 400 kilograms, the surcharge pressure can be taken as 16 kilopascals. The surcharge pressure should be increased if heavier equipment is used.

### 5.4 Buoyant Uplift of Tanks

The groundwater levels could be higher than those measured during our investigation due to both seasonal fluctuations and surface water seepage into the granular backfill material, therefore, the design and installation of the tanks should consider the tank manufacturer's recommendations for managing hydrostatic pressures and buoyant uplift. As a conservative design approach, we recommend that the ground water level be assumed near ground surface for buoyancy computations.

## 6.0 SITE SERVICES

### 6.1 Overburden Excavation

Based on the investigation results, it is anticipated that the excavation for services will be carried out through topsoil and native deposits of silt and glacial till. The planned depth of the services was not known at the time the report was written.

In the overburden, the excavation for flexible service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.010 for Type 3 Soil. The excavation for rigid service pipes should be in accordance with OPSD 802.031 for Type 3 soil.

The excavations for the services should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the soils at this site can be classified as Type 3 and allowance should be made for 1 horizontal to 1 vertical side slopes extending upwards from the base of the excavation. Alternatively, the excavations could be carried out near vertically within a tightly fitting, braced steel trench box designed specifically for this purpose.

Additional comments on overburden excavation are provided in Sections 4.2.

### 6.2 Bedrock Excavation

Depending on the invert of the new sewer and watermain, excavation of the bedrock may be required.

In bedrock, the excavation for flexible service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.013 for bedrock. The excavation for rigid service pipes should be in accordance with OPSD 802.033 for bedrock.

Our comments on bedrock excavation provided in Section 4.3 apply equally to the excavation for site services.

### 6.3 Groundwater Pumping and Management

Groundwater pumping and management guidelines are provided in Section 4.4 of this report. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services.

### 6.4 Pipe Bedding

The bedding for the new sewers should be in accordance with OPSD 802.010 and 802.013 for flexible pipes in earth and bedrock excavation, respectively, and OPSD 802.031 and OPSD 802.033 for rigid pipes in earth and bedrock excavation, respectively. The pipe bedding material should consist of at least 150 millimetres of well graded crushed stone meeting OPSS requirements for Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used in the service trench be composed of virgin (i.e., not recycled) material only.

In areas where the subgrade is disturbed or where unsuitable material (such as existing fill material) exists below the pipe subgrade level, the disturbed/unsuitable material should be removed and replaced with a subbedding layer of compacted granular material, such as that meeting OPSS Granular B Type I or Type II (50 or 100 millimetre minus crushed stone). To provide adequate support for the pipes in the long term in areas where subexcavation of overburden material is required below design subgrade level, the excavations should be sized to allow a 1 horizontal to 2 vertical spread of granular material down and out from the bottom of the pipe.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The subbedding, bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value.

### 6.5 Trench Backfill

In areas where the service trench will be located below or in close proximity to existing or future areas of hard surfacing (pavement, sidewalk, etc.), acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent hard surfaced area. The depth of frost penetration in exposed areas can normally be taken as 1.8 metres below finished grade. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I or Type II.

It is anticipated that most of the inorganic overburden materials encountered during the subsurface investigation will be acceptable for reuse as trench backfill. Any topsoil or organic soil should be wasted from the trench.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, sidewalks, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced to 90 percent of the standard Proctor dry density in areas where the trench backfill is not located below or in close proximity to existing or future roadways, parking areas, sidewalks, etc. (i.e. in landscaped areas) and provided that some settlement above the trench is acceptable.

Depending on the weather conditions at the time of construction, some wetting of materials could occur. As such, the specified densities may not be possible to achieve and, consequently, some settlement of these backfill materials should be expected. Consideration could be given to implementing one or a combination of the following measures to reduce post construction settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to compaction;
- Reuse any wet materials in the lower part of the trenches and make provision to defer final placement of the final lift of the asphaltic concrete for 3 months, or longer, to allow some of the trench backfill settlement to occur and thereby improve the final pavement appearance.
- Avoid reusing any wet material within the trench. If additional material is required for trench backfill, consideration could be given to using imported relatively dry earth fill material, or imported OPSS Select Subgrade Material below the zone of frost penetration.

### 6.6 Seepage Barriers

To prevent the granular bedding in the services trench from acting as a “French Drain” and thereby promoting migration of potential contaminants off the property, seepage barriers should be installed along the service trenches just inside the property lines. The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround to within the native backfill materials, and horizontally across the full width of the service trench excavation. The seepage barriers could consist of 1.5 metre wide dykes of compacted weathered silty clay. The weathered silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. It is noted that silty clay will need to be imported to site. Alternatively, consideration could be given to installing an anti-seep collar or mixing OPSS Granular A with bentonite (as per OPSS 1205). The locations of the seepage barriers could be provided at the final design stage.

# 7.0 ACCESS ROADWAY AND PARKING AREAS

## 7.1 Subgrade Preparation

In preparation for the construction of the access roadway and parking areas at this site, all surficial topsoil, and any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface. Any subexcavated areas could be filled with compacted earth borrow or imported granular material. The Granular B Type I, II, Select Subgrade Material or earth borrow should be placed in maximum 300 millimetres thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment.

The subgrade surfaces should be proof rolled with a large steel drum roller (under dry conditions) and shaped and crowned to promote drainage of the granular materials.

## 7.2 Flexible Pavement Structures for the Parking Areas and Access Roadway

It is suggested that parking and roadway areas be constructed using the following minimum pavement structure:

- 90 millimetres asphaltic concrete, over
- 150 millimetres of OPSS Granular A base, over
- 450 millimetres of OPSS Granular B Type I or II subbase

The 90 millimetres asphaltic concrete surface should consist of 40 millimetres of Superpave 12.5 (Traffic Level B) over 50 millimetres of Superpave 12.5 (Traffic Level B). Performance grade PG 58-34 asphaltic concrete should be specified.

This pavement structure is suitable for both light and heavy-duty vehicle access. If required, a pavement structure suitable for light-duty areas only (e.g., parking areas that will not be used by heavy trucks) could be provided as the design progresses.

Where the new pavement will abut existing pavement, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter to match the depths of the granular material(s) exposed in the existing pavement.

If the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the subbase material, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

### 7.3 Compaction Requirements

All imported granular materials should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 98 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

## 8.0 ADDITIONAL CONSIDERATIONS

### 8.1 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the soil sample from borehole BH19-1 is 7 micrograms per gram. According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate in the soil can be classified as low. For low exposure conditions, any concrete that will be in contact with the native soil or groundwater should be batched with General Use (formerly Type 10) cement. The design of any concrete should take into consideration freeze thaw effects and the presence of chlorides.

Based on the resistivity and pH of the soil samples, the soil can be classified as non-aggressive towards unprotected steel. The manufacturer of any buried steel elements that will be in contact with the soil and groundwater should be consulted to determine the durability of the product used. It is noted that the corrosivity of the soil and groundwater could vary throughout the year due to the application sodium chloride for de-icing.

### 8.2 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, hoe ramming, etc.) will cause ground vibration on and off of the site. The vibrations will attenuate with distance from the source, but may be felt at nearby structures. We recommend that preconstruction surveys be carried out on the adjacent structures and that vibration monitoring be carried out during the construction so that any damage claims can be addressed in a fair manner.

### 8.3 Winter Construction

In the event that construction is required during freezing temperatures, the soil below the proposed foundations and slabs should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any open excavations should be opened for as short a time as practicable. The materials on the sides of the excavation should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

Provision must be made to prevent freezing of any soil below the level of any existing structures or services. Freezing of the soil could result in damage to structures or services.

## 8.4 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

## 8.5 Design Review and Construction Observation

The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the final design drawings be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the site services and roadways should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Luc Bouchard

Luc Bouchard, P.Eng., ing.

LICENSED PROFESSIONAL ENGINEER
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03 Jul 2019
PROVINCE OF ONTARIO

Johnathan A. Cholewa, Ph.D.,
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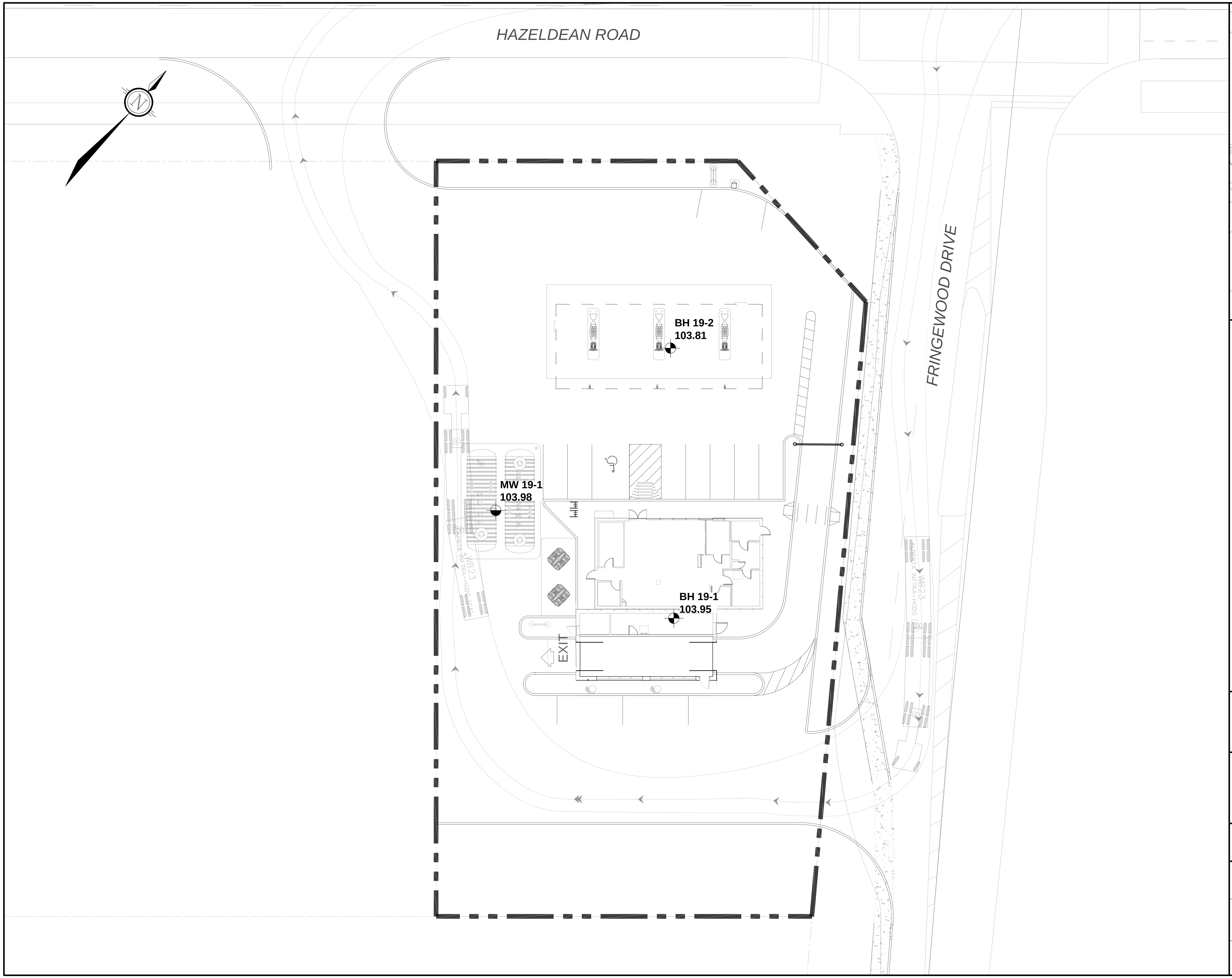
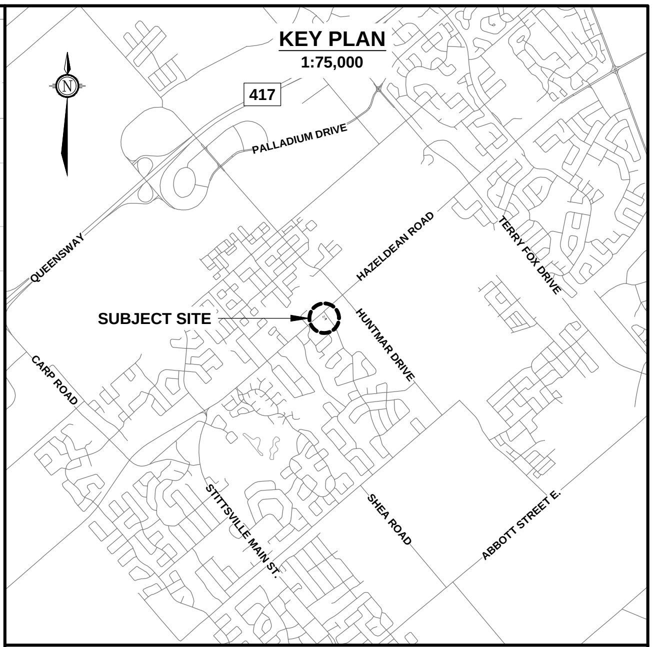


## KEY PLAN

1:75,000



**LEGEND**

⊕ **BOREHOLE LOCATION IN PLAN**
(current investigation by GEMTEC)

⊖ **MONITORING WELL LOCATION IN PLAN**
(current investigation by GEMTEC)

**BH/MW #** ◄ BOREHOLE/MONITORING WELL ID
**XX.XX** ◄ GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

NOTE:

THIS PLAN WAS PREPARED BY USING THE PROPOSED SITE PLAN BY OTHERS

Scale 1:400

0 8 16 24m

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ottawa@gemtec.ca

Client: AECOM CANADA LTD.
Project: 63993.69
Location: PROPOSED SHELL SERVICE STATION
5 ORCHARD DRIVE, OTTAWA, ON
Drwn by: P.C.
Chkd by: L.B.

BOREHOLE LOCATION PLAN

Date: JULY 2019
Rev.: 0

**FIGURE 1**

# APPENDIX A

Record of Borehole Sheets
Results of Laboratory Classification Testing
List of Abbreviations and Terminology
Lithological and Geotechnical Rock Description Terminology

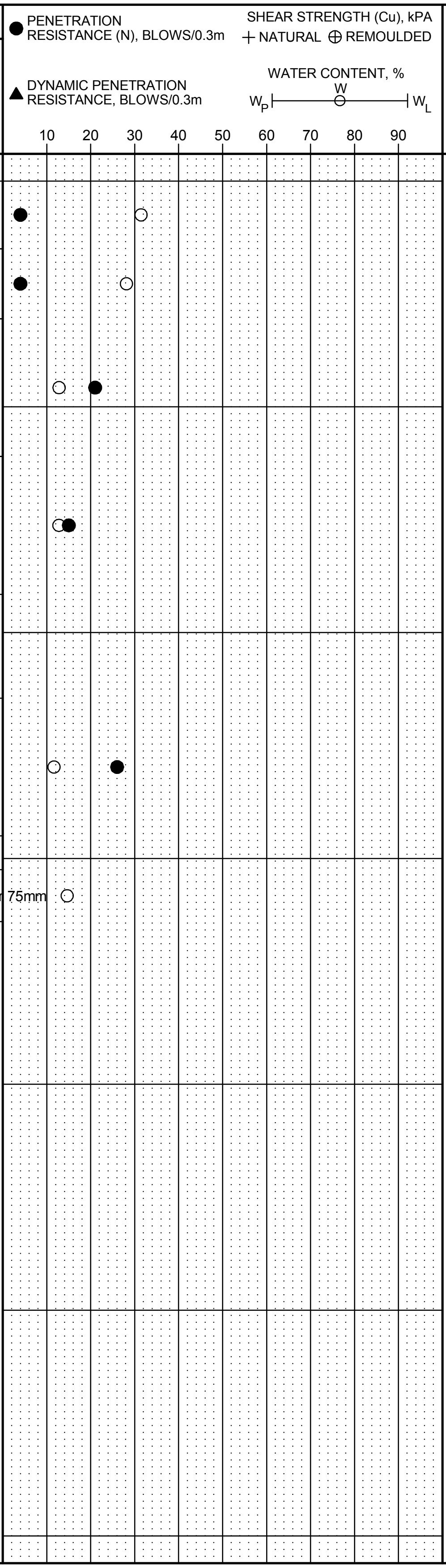
# RECORD OF BOREHOLE 19-1

CLIENT: AECOM Canada Ltd.
PROJECT: Geotechnical Investigation
JOB#: 63993.69
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 4 2019

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE DESCRIPTION | ELEV. DEPTH (m) |
|---|---|---|---|
| 0 | | Ground Surface | 103.95 |
| | Hydrovacuum | Dark brown clayey silt (TOPSOIL) | 103.80<br>0.15 |
| | | Very loose, brown SILT, some clay, trace sand | 103.05<br>0.90 |
| 1 | | Compact, gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL) | |
| 2 | Power Auger<br>Hollow Stem Auger (210mm OD) | | |
| 3 | | | 100.67 |
| | | Fractured and weathered BEDROCK | 100.59<br>3.36 |
| | | Auger Refusal<br>End of Borehole | |
| 4 | | | |
| 5 | | | |
| 6 | | | |

| SAMPLES NUMBER | TYPE | RECOVERY, mm | BLOWS/0.3m | ADDITIONAL LAB. TESTING |
|---|---|---|---|---|
| 1a | SS | 350 | 4 | |
| 1b | SS | 350 | 4 | MH |
| 2 | SS | 450 | 21 | |
| 3 | SS | 450 | 15 | |
| 4 | SS | 400 | 26 | |
| 5 | SS | 50 | 50 for 75mm | |



PIEZOMETER OR STANDPIPE INSTALLATION

Backfilled with auger cuttings

GEO - BOREHOLE LOG 63993.69_GINT LOGS BOREHOLES.GPJ GEMTEC 2018.GDT 28/6/19

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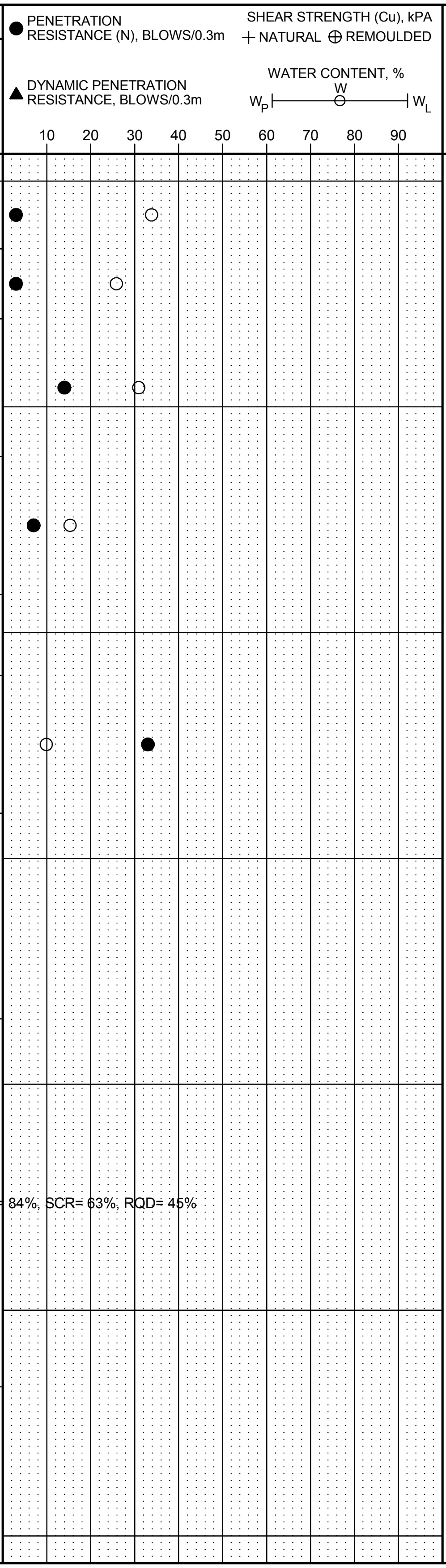
LOGGED: ML
CHECKED: LB

# RECORD OF BOREHOLE 19-2

CLIENT: AECOM Canada Ltd.
PROJECT: Geotechnical Investigation
JOB#: 63993.69
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 4 2019

| BORING METHOD | SOIL PROFILE: DESCRIPTION | ELEV. DEPTH (m) | SAMPLES: NUMBER | TYPE | RECOVERY, mm | BLOWS/0.3m | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|
| | Ground Surface | 103.81 | | | | | | |
| Hydrovacuum | Dark brown clayey silt (TOPSOIL) | 103.61<br>0.20 | 1 | SS | 350 | 3 | | Backfilled with auger cuttings |
| | Very loose to compact, brown SILT, some clay, trace sand | 102.91<br>0.90 | 1<br>2 | SS<br>SS | 350<br>450 | 3<br>14 | | |
| Power Auger<br>Hollow Stem Auger (210mm OD) | Loose to dense, brown, gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL) | 101.01<br>2.80 | 3<br>4 | SS<br>SS | 450<br>400 | 7<br>33 | | |
| | Fractured and weathered BEDROCK | 100.10<br>3.71 | | | | | | |
| Diamond Rotary Core<br>HQ (89mm OD) | Moderately fractured, slightly weathered LIMESTONE BEDROCK, banded with shale | 98.47<br>5.34 | 5 | RC | 1630 | TCR= 84%, SCR= 63%, RQD= 45% | | |
| | End of borehole | | | | | | | |



LOGGED: ML
CHECKED: LB

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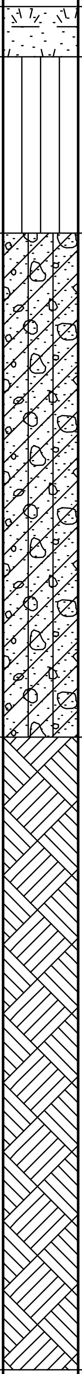
# RECORD OF BOREHOLE MW19-1

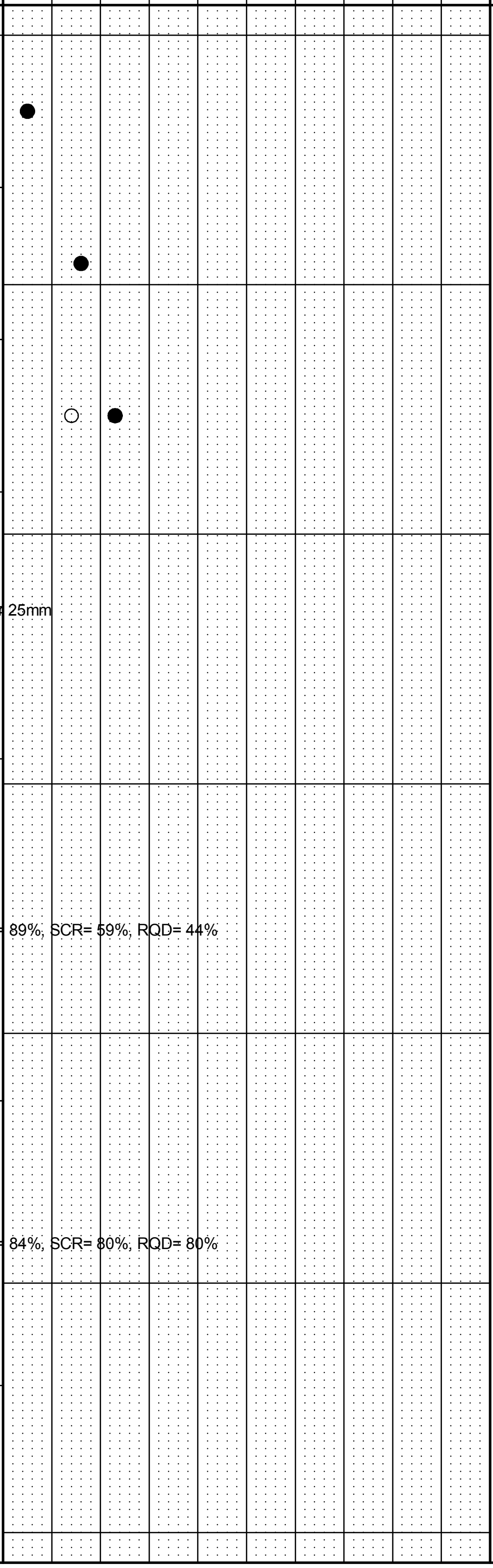
CLIENT: AECOM Canada Ltd.
PROJECT: Geotechnical Investigation
JOB#: 63993.69
LOCATION: See Borehole Location Plan, Figure 1

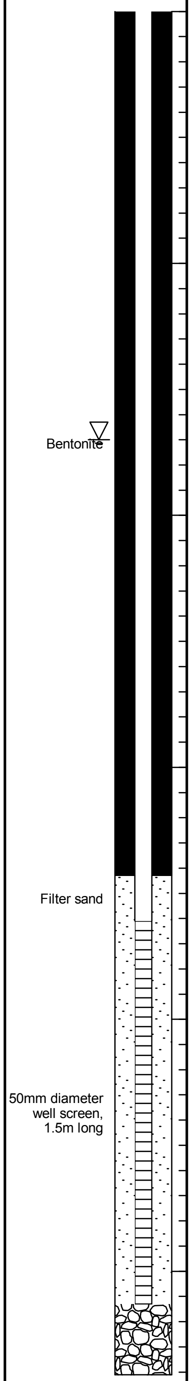
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 4 2019

| BORING METHOD | DESCRIPTION | ELEV. DEPTH (m) |
|---|---|---|
| | Ground Surface | 103.98 |
| Hydrovacuum | Dark brown clayey silt (TOPSOIL) | 103.78<br>0.20 |
| | Loose to compact, brown SILT, some clay, trace sand | 103.08<br>0.90 |
| Power Auger<br>Hollow Stem Auger (210mm OD) | Compact, brown gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL) | 101.08<br>2.90 |
| Diamond Rotary Core<br>HQ (89mm OD) | Moderately fractured, slightly weathered LIMESTONE BEDROCK, banded with shale | 98.57<br>5.41 |
| | End of borehole | |

| NUMBER | TYPE | RECOVERY, mm | BLOWS/0.3m | ADDITIONAL LAB. TESTING |
|---|---|---|---|---|
| 1 | SS | 350 | 5 | |
| 2 | SS | 350 | 16 | |
| 3 | SS | 450 | 23 | MH |
| 5 | SS | 25 | 50 for 25mm | |
| 5 | RC | 1370 | TCR= 89%, SCR= 59%, RQD= 44% | UC= 146 MPa |
| 6 | RC | 1140 | TCR= 84%, SCR= 80%, RQD= 80% | |







GROUNDWATER OBSERVATIONS

| DATE | DEPTH (m) | ELEV. (m) |
|---|---|---|
| 19/06/10 | 1.7 ▽ | 102.3 |

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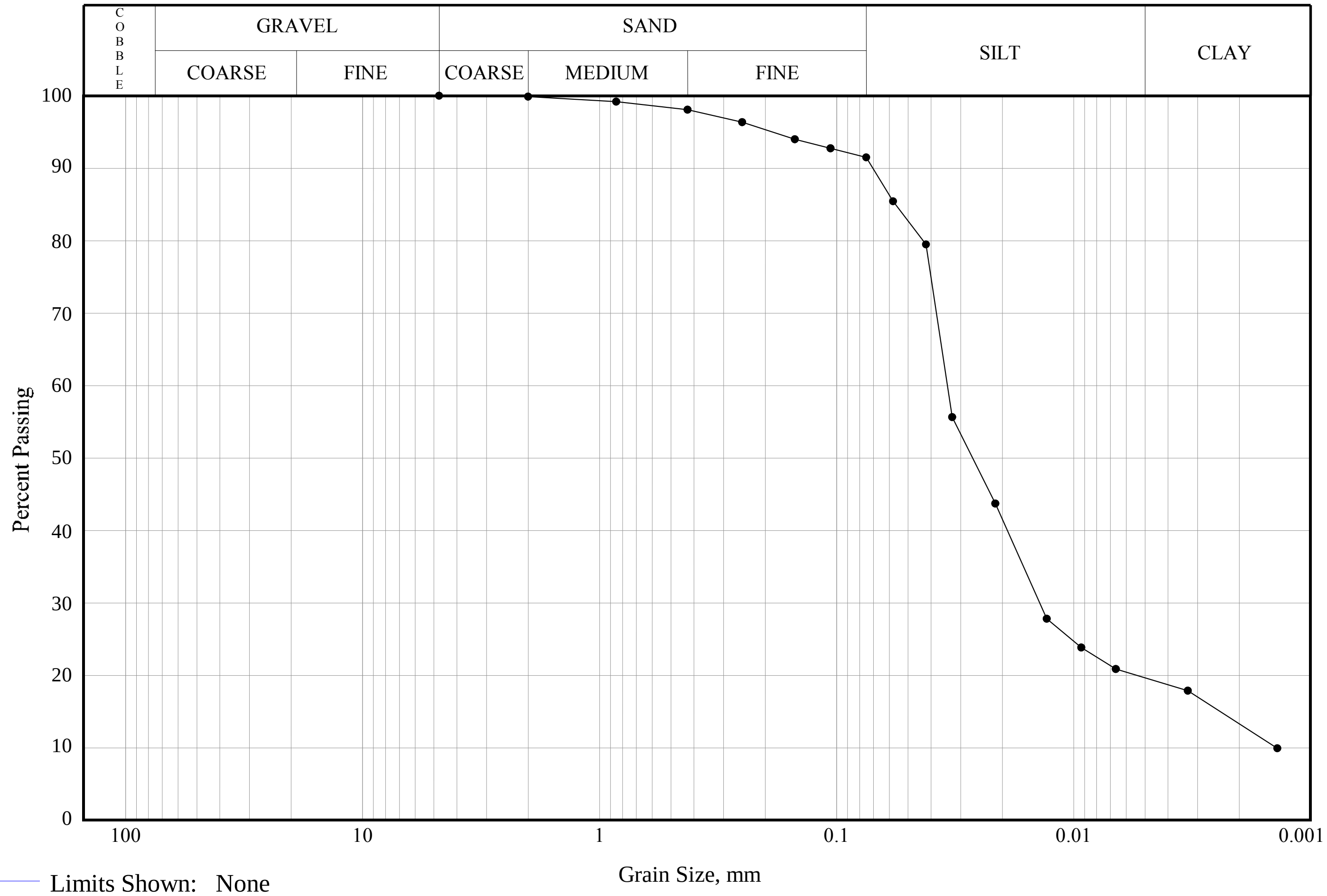
LOGGED: ML
CHECKED: LB

| | | |
|---|---|---|
| GEMTEC<br>Consulting Engineers and Scientists | Client: AECOM Canada Ltd<br>Project: Geotechnical Investigation - 5 Orchard Drive, Ottawa<br>Project #: 6399369 | Soils Grading Chart |



Limits Shown: None

| Line Symbol | Sample | Borehole/ Test Pit | Sample Number | Depth | % Cob.+ Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|---|---|---|
| ● | Silt | 19-1 | 01b | 0.30-0.61 | 0.0 | 8.5 | 71.8 | 19.7 |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

| Line Symbol | CanFEM Classification | USCS Symbol | $D_{10}$ | $D_{15}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $D_{85}$ | % 5-75μm |
|---|---|---|---|---|---|---|---|---|---|
| ● | Silt , some clay , trace sand | N/A | 0.00 | 0.00 | 0.01 | 0.03 | 0.03 | 0.06 | 71.8 |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |

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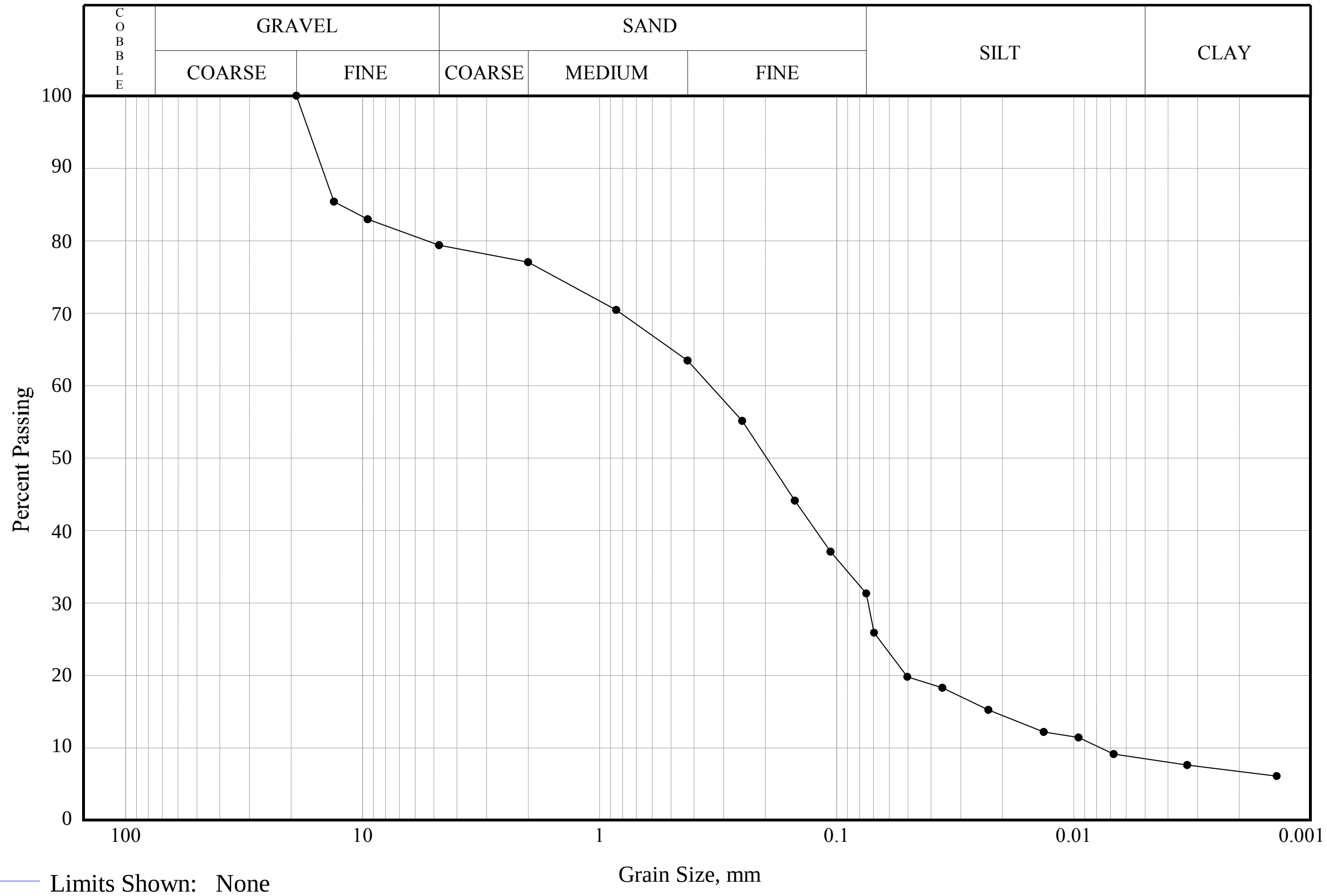
Client: AECOM Canada Ltd
Project: Geotechnical Investigation - 5 Orchard Drive, Ottawa
Project #: 6399369

# Soils Grading Chart



Limits Shown: None

| Line Symbol | Sample | Borehole/ Test Pit | Sample Number | Depth | % Cob.+ Gravel | % Sand | % Silt | % Clay |
|---|---|---|---|---|---|---|---|---|
| —●— | Glacial Till | 19-1MW | 03 | 1.22-1.82 | 20.6 | 48.1 | 22.8 | 8.5 |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

| Line Symbol | CanFEM Classification | USCS Symbol | $D_{10}$ | $D_{15}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $D_{85}$ | % 5-75μm |
|---|---|---|---|---|---|---|---|---|---|
| —●— | Gravelly silty sand , trace clay | N/A | 0.01 | 0.02 | 0.07 | 0.20 | 0.34 | 12.53 | 22.8 |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |

# ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

| SAMPLE TYPES | |
|---|---|
| AS | Auger sample |
| CA | Casing sample |
| CS | Chunk sample |
| BS | Borros piston sample |
| GS | Grab sample |
| MS | Manual sample |
| RC | Rock core |
| SS | Split spoon sampler |
| ST | Slotted tube |
| TO | Thin-walled open shelby tube |
| TP | Thin-walled piston shelby tube |
| WS | Wash sample |

| SOIL TESTS | |
|---|---|
| w | Water content |
| PL, $w_p$ | Plastic limit |
| LL, $w_L$ | Liquid limit |
| C | Consolidation (oedometer) test |
| $D_R$ | Relative density |
| DS | Direct shear test |
| $G_S$ | Specific gravity |
| M | Sieve analysis for particle size |
| MH | Combined sieve and hydrometer (H) analysis |
| MPC | Modified Proctor compaction test |
| SPC | Standard Proctor compaction test |
| OC | Organic content test |
| UC | Unconfined compression test |
| γ | Unit weight |

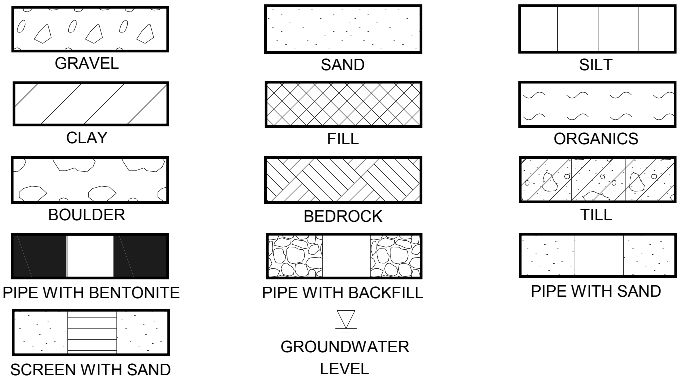
## PENETRATION RESISTANCE

**Standard Penetration Resistance, N**
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

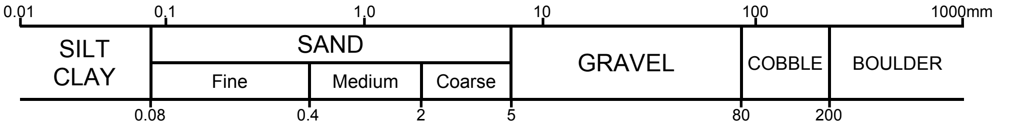
**Dynamic Penetration Resistance**
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

| | |
|---|---|
| WH | Sampler advanced by static weight of hammer and drill rods |
| WR | Sampler advanced by static weight of drill rods |
| PH | Sampler advanced by hydraulic pressure from drill rig |
| PM | Sampler advanced by manual pressure |

| COHESIONLESS SOIL Compactness | | COHESIVE SOIL Consistency | |
|---|---|---|---|
| SPT N-Values | Description | Cu, kPa | Description |
| 0-4 | Very Loose | 0-12 | Very Soft |
| 4-10 | Loose | 12-25 | Soft |
| 10-30 | Compact | 25-50 | Firm |
| 30-50 | Dense | 50-100 | Stiff |
| >50 | Very Dense | 100-200 | Very Stiff |
| | | >200 | Hard |



**GRAIN SIZE**



**DESCRIPTIVE TERMINOLOGY**

(Based on the CANFEM 4th Edition)

| 0 | 10 | 20 | 35 |
|---|---|---|---|
| TRACE | SOME | ADJECTIVE | noun > 35% and main fraction |
| trace clay, etc | some gravel, etc. | silty, etc. | sand and gravel, etc. |

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# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

| WEATHERING STATE | |
|---|---|
| Fresh | No visible sign of rock material weathering |
| Faintly weathered | Weathering limited to the surface of major discontinuities |
| Slightly weathered | Penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material |
| Moderately weathered | Weathering extends throughout the rock mass but the rock material is not friable |
| Completely weathered | Rock is wholly decomposed and in a friable condition but the rock and structure are preserved |

| CORE CONDITION |
|---|
| **Total Core Recovery (TCR)**<br>The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run |
| **Solid Core Recovery (SCR)**<br>The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run. |
| **Rock Quality Designation (RQD)**<br>The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completed broken core to 100% for core in solid segments. |

| BEDDING THICKNESS | |
|---|---|
| **Description** | **Thickness** |
| Thinly laminated | < 6 mm |
| Laminated | 6 - 20 mm |
| Very thinly bedded | 20 - 60 mm |
| Thinly bedded | 60 - 200 mm |
| Medium bedded | 200 - 600 mm |
| Thickly bedded | 600 - 2000 mm |
| Very thickly bedded | 2000 - 6000 mm |

| DISCONTINUITY SPACING | |
|---|---|
| **Description** | **Spacing** |
| Very close | 20 - 60 mm |
| Close | 60 - 200 mm |
| Moderate | 200 - 600 mm |
| Wide | 600 -2000 mm |
| Very wide | 2000 - 6000 mm |

| ROCK QUALITY | |
|---|---|
| **RQD** | **Overall Quality** |
| 0 - 25 | Very poor |
| 25 - 50 | Poor |
| 50 - 75 | Fair |
| 75 - 90 | Good |
| 90 - 100 | Excellent |

| ROCK COMPRESSIVE STRENGTH | |
|---|---|
| **Comp. Strength, MPa** | **Description** |
| 1 - 5 | Very weak |
| 5 - 25 | Weak |
| 25 - 50 | Moderate |
| 50 - 100 | Strong |
| 100 - 250 | Very strong |

# APPENDIX B

## Rock Core Photo – Figure B1

Report to: AECOM Canada Ltd.
Project: 63993.69 (July 3, 2019)

**MW19-1**
**BORING DATE: June 4, 2019**
**DEPTH: 2.90 to 5.41 mbgs**

**BH19-2**
**BORING DATE: June 4, 2019**
**DEPTH: 3.71 to 5.34 mbgs**

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Project

GEOTECHNICAL INVESTIGATION
PROPOSED SHELL SERVICE STATION
5 ORCHARD DRIVE, OTTAWA, ON

**FIGURE B1**

File No.
63993.69

ROCK CORE PHOTOGRAPHS

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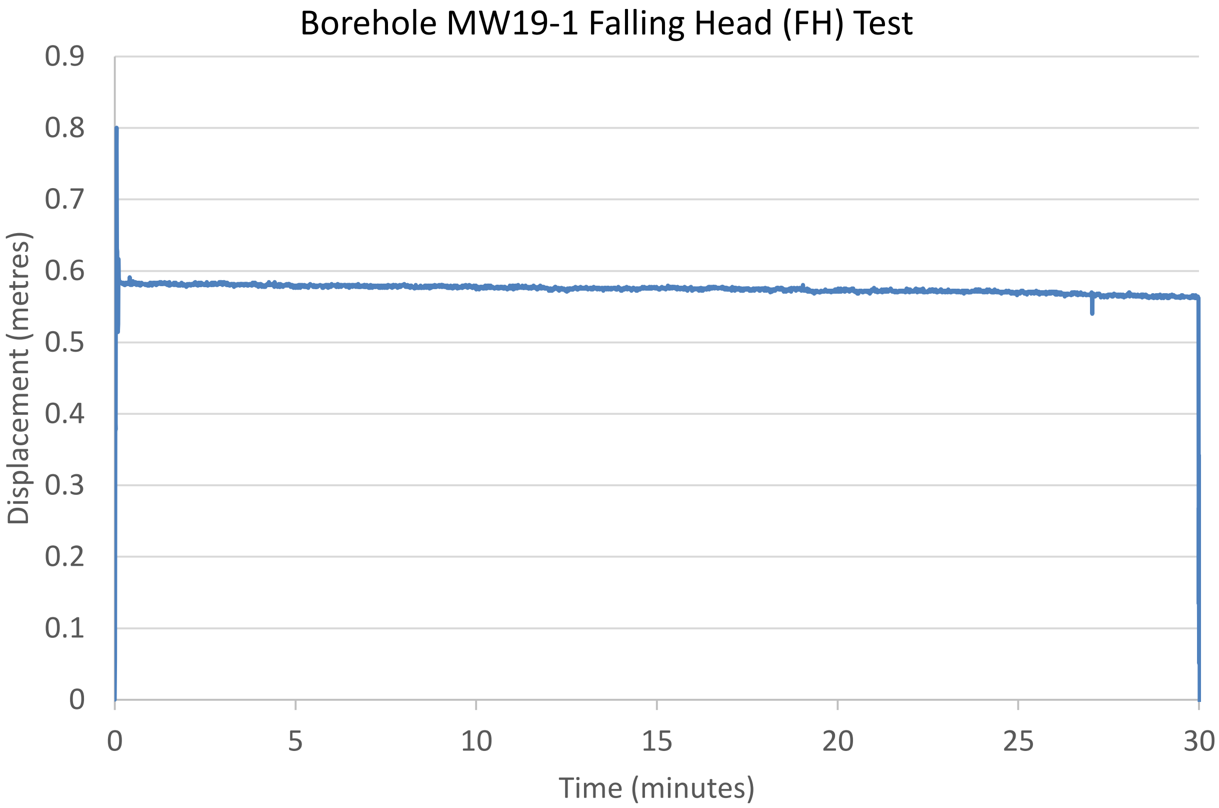
# APPENDIX C

## Hydraulic Testing Results

Report to: AECOM Canada Ltd.
Project: 63993.69 (July 3, 2019)

# Hydraulic Testing

**FIGURE C1**



Notes:

1. Static water level 1.8 metres below ground surface as measured on June 13, 2019.
2. Insufficient recovery, hydraulic conductivity not calculated.

Well Data:

Displacement observed (slug size): 0.79 metres (0.60 m)
Well Depth: 5.10 metres
Screen Length: 1.52 metres
Well Radius: 0.085 metres

Aquifer Data

Saturated Thickness: 3.3 metres
Anisotropy Ratio (Kz/Kr): 1
Aquifer Model: Confined
Static Water Level: 1.8 metres bgs

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

Date: June 2019

Project: 63993.69

# APPENDIX D

## Chemical Test Results on Soil Sample
## Corrosion of Buried Concrete and Steel
## Paracel Laboratories Ltd. Order No. 1924207

Certificate of Analysis

**Client: GEMTEC Consulting Engineers and Scientists Limited**

**Client PO:**

Report Date: 17-Jun-2019

Order Date: 11-Jun-2019

**Project Description: 63993.69**

| | **Client ID:** | 19-1 SA3 | - | - | - |
|---|---|---|---|---|---|
| | **Sample Date:** | 04-Jun-19 09:00 | - | - | - |
| | **Sample ID:** | 1924207-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 88.3 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 7.88 | - | - | - |
| Resistivity | 0.10 Ohm.m | 61.9 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 34 | - | - | - |
| Sulphate | 5 ug/g dry | 7 | - | - | - |

experience • knowledge • integrity

| | |
|---|---|
| civil | civil |
| geotechnical | géotechnique |
| environmental | environnementale |
| field services | surveillance de chantier |
| materials testing | service de laboratoire des matériaux |

expérience • connaissance • intégrité