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Subsurface Investigation
Proposed Service Station
106 Eagleson Road

Ottawa, Ontario

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Submitted to:

AECOM
3292 Production Way
Burnaby, BC
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Subsurface Investigation
Proposed Service Station
106 Eagleson Road
Ottawa, Ontario

July 28, 2025
Project: 100041.032

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1.0 INTRODUCTION

This report presents the results of a subsurface investigation carried out in support of the replacement of an automotive service station at 106 Eagleson Road in Ottawa, Ontario (hereinafter referred to as ‘the Site’).

The purpose of the investigation was to identify the general subsurface conditions at the Site by means of a limited number of boreholes and, based on the information obtained, to provide engineering guidelines and recommendations on the geotechnical design aspects of the project, including construction considerations, that could influence development decisions.

The investigation was carried out in general accordance with our proposal dated April 24, 2025.

This report is subject to the ‘Conditions and Limitations of this Report’ which follows the text of the report and which are considered an integral part of the report.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

Based on information provided to GEMTEC, the following details are understood about the project:

- The Site is currently developed as a service station with 4 fueling pumps, a ‘kiosk’ style convenience store and a carwash.
- The proposed development will consist of:
    - Removal of the existing convenience store and fuel pumps. The existing carwash will remain;
    - Construction of a new building that will hold a 211 square metre convenience store and a 149 square metre quick service restaurant. The building will be one storey above grade and have a slab at grade (no below grade level);
    - New fuel pump islands (6 total) and canopy;
    - New underground storage tanks;
    - Expanded and/or reconstructed asphaltic concrete paved laneways and parking areas; and,
    - Site services (i.e., municipal water, sewage, and stormwater connections).
- The planned maximum depths of excavations for services and underground storage tanks below existing ground surface are:
    - 3 metres for site service trenches; and,
    - 5 metres for underground storage tanks.
- The conceptual foundations for the pump island canopy will consist of spread footings.

2.2 Site Description

The following is known about the Site:

- The site is bordered to the north by undeveloped forested lands and Katimavik Road beyond, to the east by Eagleson Road and agricultural lands beyond, to the south by commercial and residential development, and to the west by residential development;
- The site has a total area of about 1.9 acres; and,
- The site is mostly cleared of vegetation and developed with the aforementioned structures and asphaltic concrete paved areas, with some grassed areas.

The location of the Site is shown on Figure 1 in Appendix A.

3.0 REVIEW OF AVAILABLE INFORMATION

3.1 Geological Mapping

Surficial geology maps indicate that the site is underlain by deposits of silt and clay. Bedrock geology maps indicate that the overburden is underlain by sandstone of the Nepean Formation at depths ranging between about 0 to 5 metres below ground surface.

4.0 METHODOLOGY

4.1 Geotechnical Investigation

The fieldwork for this investigation was carried out on July 3 and July 4, 2025, at which time three boreholes (numbered 25-01 to 25-03, inclusive) were advanced at the Site. The approximate locations of the boreholes are shown on the Site Plan, Figure 1 (see Appendix A).

The boreholes were advanced using both a hydrovac truck (supplied and operated by Badger Daylighting of Ottawa, Ontario) and a truck mounted drill rig (supplied and operated by George Downing Estate Drilling Limited of Hawkesbury, Ontario). The boreholes were advanced from ground surface to depth of between about 1.4 and 1.7 metres using hydro-excavation (hydrovaccuming) techniques after which the boreholes were advanced using hollow stem augering techniques. Standard penetration tests were carried out within the overburden and samples of the soils encountered were recovered using drive open sampling equipment. At all test hole locations, refusal to augering was encountered. Within two of the boreholes (i.e, 25-02 and 25-03), the bedrock was cored an additional 3 to 4 metres, using rotary diamond drilling techniques while retrieving HQ sized bedrock core.

The fieldwork was observed by a member of our engineering staff who directed the drilling operations, observed the in-situ testing and logged the samples and boreholes.

Following the fieldwork, 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, Atterberg Limits, and grain size distribution. Bedrock core samples were tested for uniaxial compressive strength. Two samples of the overburden soils were sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

The borehole locations were selected by GEMTEC, in consultation with AECOM, based on the details of the proposed development and existing site features. The ground surface elevations at the borehole locations were surveyed using a GPS instrument and referenced to geodetic vertical network CGVD28.

4.2 Hydrogeological Investigation

Monitoring wells were installed in boreholes 25-02 and 25-03. Each monitoring well consisted of a 50-millimetre diameter screened PVC pipe installed within a surround of filter sand. Above the surround of filter sand, bentonite was used to seal the well screen from overlying materials and the wells were finished with flush-mounted caps. Well depths and screen lengths were chosen based on the subsurface material types and inferred groundwater levels encountered during drilling. Details of the well installations are provided on the Borehole Logs in Appendix B.

4.2.1 Water Level Measurements

Groundwater levels were measured in the monitoring wells on July 14, 2025. Preceding the groundwater level measurements, monitoring wells were developed using a foot valve and tubing to purge the water column volume three times or until the well was purged dry. The measured levels are presented later in this report.

4.2.2 Hydraulic Conductivity Testing

Hydraulic testing consisting of rising and falling head testing was carried out on the July 14, 2025, to estimate the hydraulic conductivity of the screened overburden and potential dewatering volumes to be expected during construction. A summary of the testing performed is provided in Table 4.1, the results are discussed later in this document.

Falling head testing involved introducing an instantaneous pressure increase to the water column within the well screen (equal to the volume of the slug) and monitoring the dissipation of the water level over time using a groundwater data logging pressure transducer together with an electric water level tape. Rising head testing involved introducing an instantaneous pressure decrease to the water column within the well screen (equal to the volume of the slug) and monitoring the recovery of the water level over time using a groundwater data logging pressure transducer together with an electric water level tape. Manual measurements were also taken for 5 minutes following the introduction and removal of the slug.

Table 4.1 – Summary of Hydraulic Testing Program

Borehole ID.	Screened Material	Test Completed	
		Falling Head Test	Rising Head Test
25-02	Native Silty Clay and Glacial Till	Y	N[1]
25-03	Sandstone Bedrock	Y	Y

Notes: 1. Rising head testing was not attempted in borehole 25-02 due to slow response during initial falling head test.

4.2.3 Groundwater Quality Sampling

Recovery of a groundwater sample for comparison to the City of Ottawa storm and sanitary sewer use bylaws was not within GEMTEC's scope of work.

5.0 SUBSURFACE CONDITIONS

5.1 General

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix B. The results of the soil classification testing are provided in Appendix C and on the Record of Borehole sheets. Photographs of bedrock core samples are provided in Appendix D. The results of the hydraulic conductivity are provided in Appendix E. The results of the chemical testing on the soil samples are provided in Appendix F.

5.2 Subsurface Conditions

5.2.1 Overview

In general, the subsurface conditions observed at the borehole locations consist mainly of a surficial layer of topsoil or asphaltic pavement and structure overlying fill and/or native deposits of clay and glacial till. The bedrock surface was encountered, where confirmed by coring, at depths of 4.3 and 4.6 metres below ground surface. The groundwater levels in monitoring wells 25-02 and 25-03 were measured at 3.1 and 2.7 metres below ground surface, respectively, on July 14, 2025. A summary of the subsurface conditions encountered in the boreholes is provided in Table 5.1. Where no bedrock coring was carried out, the refusal may be due to the presence of cobbles and/or boulders or the bedrock surface.

Table 5.1 – Summary of Subsurface Conditions, GEMTEC (2025)

Test Hole No.	Depth from Ground Surface to Bottom of Layer (metres)					
	Topsoil/Existing Pavement Structure/Fill	**Native Clay**	**Glacial Till**	**Cored Bedrock Surface / Auger Refusal**	**Bedrock (core end depth)**	**Groundwater Depth below Ground Surface (m)**
25-01	2.3	2.8	3.8	3.8	n/a	n/a
25-02	2.3	4.3	4.6	4.6	7.7	3.1
25-03	2.1	4.2	4.3	4.3	8.3	2.7

Notes:
1. n/a – not applicable (stratigraphy not logged, no monitoring well was installed, auger refusal did not occur or no bedrock coring at noted location)

The subsurface conditions encountered in the boreholes are described in further detail in the following sections.

5.2.2 Topsoil

Topsoil was encountered extending from ground surface at boreholes 25-01 and 25-02 with thicknesses of 230 and 280 millimetres, respectively. The topsoil generally consists of brown silty sand with organic material and varying amounts of gravel.

A buried layer of silty sand topsoil, about 0.6 m in thickness, was encountered below the fill in borehole 25-03. Standard penetration testing carried out in the layer gave an N value of 28 blows per 0.3 metres of penetration, which would suggest a compact relative density. The measured water content of one sample of the layer was about 102 percent.

5.2.3 Existing Pavement Structure

Borehole 25-03 was advanced from ground surface through the existing asphaltic concrete pavement, which has a thickness of about 110 millimetres.

Below the asphalt concrete, a layer of base/subbase material was encountered, which extends to a depth of about 1.5 metres below ground surface. This layer can generally be described as a grey brown to brown sand and gravel. Cobbles were noted in the upper depth of the layer and an increased quantity of cobbles and boulders was noted in the lower depth of the layer (below about 1 metre depth).

Grain size distribution testing was undertaken on one sample of the base/subbase material. The results are provided in Appendix C and are summarized in Table 5.2.

Table 5.2 – Summary of Grain Size Distribution Tests (Base/Subbase)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
25-03	1	0.1 to 0.5	41	50	9	

The measured water content of two base/subbase samples was about 4 and 5 percent. The water content testing was carried out in general conformance with ASTM D4959.

5.2.4 Fill

Fill was encountered below the topsoil at boreholes 25-01 and 25-02, where it extends to a depth of 2.3 metres below ground surface (elevations 99.6 and 100.2 metres, respectively). The fill composition is variable but can generally be described as a silt and sand mixture, to sand and gravel mixture, with varying quantities of cobbles and possibly boulders.

Standard penetration tests carried out in the fill gave N values of 9 and 93 blows per 0.3 metres of penetration, which would suggest a loose to very dense relative density. However, it is possible that the higher value is due to the presence of cobbles and/or boulders.

Grain size distribution testing was undertaken on one sample of the fill material. The results are provided in Appendix C and are summarized in Table 5.3.

Table 5.3 – Summary of Grain Size Distribution Tests (Fill)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
25-02	4	0.9 to 1.4	43	35	22	

The measured water content of two fill samples was about 6 and 13 percent. The water content testing was carried out in general conformance with ASTM D4959.

5.2.5 Clay

A native deposit of grey brown, clay was encountered below the fill in boreholes 25-01 and 25-02, and below the former topsoil layer in borehole 25-03, at depths ranging between about 2.1 and 2.3 metres below ground surface. The clay has been weathered over time to form a crust.

Standard penetration tests carried out in the deposit gave N values of between about 4 and 12 blows per 0.3 metres of penetration (clay with a blow count of greater than about 2 will generally have a shear strength of greater than 100 kilopascals, which is not measurable using a standard MTO N-vane). This indicates the weathered crust has a stiff to very stiff consistency.

Grain size distribution testing was undertaken on one sample of the clay. The results are provided in Appendix C and are summarized in Table 5.4.

Table 5.4 – Summary of Grain Size Distribution Tests (Clay)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
25-02	7	3.1 to 3.7	1	9	45	45

Atterberg limit testing was carried out on one sample of the clay and the results are provided in Appendix C and summarized in Table 5.5.

Table 5.5 – Summary of Atterberg Limit Test Results (Clay)

Borehole / Sample No.	Water Content (%)	Liquid Limits (%)	Plastic Limits (%)	Plasticity Index
24-01 / 6	46.1	44.5	22.8	22

The measured water content of seven samples of clay ranged from about 36 and 55 percent. The water content testing was carried out in general conformance with ASTM D4959.

5.2.6 Glacial Till

Native deposits of glacial till were encountered below the silty clay in all the boreholes, at depths ranging between about 2.8 and 4.3 metres below ground surface (i.e., between about elevations 99.1 and 97.9 metres).

Glacial till is generally considered to be a heterogeneous mixture of all grain sizes and often contains cobbles and boulders. At this site, the deposit can generally be described as grey silt and sand with varying amounts of gravel and clay. Cobbles and boulders should also be expected in the glacial till deposit.

Standard penetration testing carried out within the glacial till gave an N value of 38 blows per 0.3 metres of penetration, which reflects a dense relative density.

Grain size distribution testing was undertaken on one sample of the glacial till. The results are provided in Appendix C and are summarized in Table 5.6.

Table 5.6 – Summary of Grain Size Distribution Tests (Glacial Till)

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
25-01	7	3.1 to 3.7	5	50	33	12

The measured water content of two samples of the glacial till was about 8 and 32 percent. The water content testing was carried out in general conformance with ASTM D4959.

5.2.7 Refusal and Bedrock

At borehole 25-01, auger refusal was encountered at a depth of about 3.8 metres below ground surface (elevation 98.1 metres). Auger refusal may indicate the surface of the bedrock or augers and boulders within the till above the bedrock surface.

At boreholes 25-02 and 25-03, the bedrock was encountered at depths of 4.6 and 4.3 metres below ground surface (elevations 97.9 and 97.2 metres, respectively) and was proven by coring, for lengths between about 3.1 and 4.0 metres, using HQ sized drilling equipment. The retrieved bedrock cores consist of fresh sandstone, as presented on the Record of Borehole sheets in Appendix B and photographs of the core samples of the bedrock in Appendix D.

The Rock Quality Designation (RQD) values generally ranged from about 0 to 94 percent, indicating a rock mass of very poor to excellent quality.

Laboratory Uniaxial Compression Strength (UCS) tests were carried out on two bedrock core samples and the measured UCS values were 77 and 105 MPa, indicating the bedrock is strong to very strong. The results of the UCS testing are provided in Appendix C.

5.2.8 Groundwater Levels

A monitoring well was installed in each of boreholes 25-02 and 25-03 to measure stabilized groundwater conditions and to facilitate hydraulic testing.

Table 5.7 summarizes the groundwater level data that was collected on July 14, 2025. 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.

Table 5.7 – Summary of Groundwater Levels

Borehole	Ground Surface Elevation (masl)	Groundwater Depth (mbgs)	Groundwater Elevation (masl)
25-02 (Overburden)	102.5	3.1	99.4

Borehole	Ground Surface Elevation (masl)	Groundwater Depth (mbgs)	Groundwater Elevation (masl)
25-03 (Bedrock)	102.1	2.7	99.4

Notes: masl = metres above mean sea level in CGVD28 ; mbgs = metres below existing ground surface

5.2.9 Hydraulic Conductivity

The results of the hydraulic testing completed in boreholes 25-02 and 25-03 are summarised in Table 5.8 and analyses are provided in Appendix E.

Estimates of hydraulic conductivity were calculated from the results of the falling/rising head tests using the Hvorslev analysis (refer to Table 5.8 below). The displacement volume of the slug was used in the analysis for all boreholes tested.

With regard to the calculated values, the following should be noted:

- The falling head test in borehole 25-02 could not be analysed completely, due to the slow response of the water level. The literature value for silty clay ranges from 10^{-9} to 10^{-5} metres per second (Freeze and Cherry, 1979).
- The estimated hydraulic conductivity for the sandstone bedrock at borehole 25-03 is within the literature range for the noted bedrock, which ranges from 10^{-7} to 10^{-3} metres per second (Freeze and Cherry, 1979)

It should be noted that well-executed slug tests yield theoretical minimum values for the horizontal hydraulic conductivity of the tested aquifer due to inefficiencies associated with typical well construction, installation, and development (Butler, 1998); therefore, higher hydraulic conductivity values presented in Table 5.8 may be applicable to these units.

The results of the testing represent the conditions in the vicinity of the well screen and as distance increases from the well screen the conditions may vary from those reported. Conditions between wells have not been established, therefore variability in hydraulic conductivity within the site should be anticipated.

Table 5.8 – Summary of Hydraulic Testing Results

Borehole ID	Screened Material	Test Type	Static Groundwater Depth (metres)	Displacement (metres)	Recovery Time (minutes)	Recovery (percent)
25-02	Silty Clay / Glacial Till	Falling Head	2.9	0.45	20	25

Borehole ID	Screened Material	Test Type	Static Groundwater Depth (metres)	Displacement (metres)	Recovery Time (minutes)	Recovery (percent)
25-03	Bedrock (Sandstone)	Falling Head	2.6	0.6	2.5	95
25-03	Bedrock (Sandstone)	Rising Head	2.6	0.6	2.0	95

Table 5.9 – Calculated Hydraulic Conductivities

Borehole ID	Screened Geological Unit	Calculated Hydraulic Conductivity, k (metres per second)	
		Falling Head Test	Rising Head Test
25-02	Native Silty Clay / Glacial Till	< 4 x10^{-7}	-
25-03	Bedrock (Sandstone)	8.9 x 10^{-5}	9.4 x 10^{-5}

5.3 Soil Chemistry Relating to Corrosion

The results of chemical testing on two samples of the native overburden soils recovered from the boreholes are provided in Appendix F and are summarized in Table 5.10 below.

Table 5.10 – Summary of Corrosion Testing

Parameter	Borehole 25-01 Sample No. 5 1.7 to 2.3 m	Borehole 25-02 Sample No. 4 0.9 to 1.4 m
Chloride Content (ug/g)	130	72
Resistivity (Ohm.m)	26.1	26.7
pH	7.28	7.54
Sulphate Content (ug/g)	69	19

6.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

6.1 Suitability of the Site for Proposed Development

Based on the results of this investigation it is GEMTEC's opinion that the Site is considered suitable for the proposed development, from a geotechnical perspective, noting that only concepts for the structures were available at the time of reporting.

Notable items identified during this investigation that may influence design and construction during development at the Site include the presence of relatively shallow and, at depth, very poor-quality bedrock and a relatively shallow groundwater level.

6.2 Seismic Site Designation

Based on the results of the subsurface investigation, the proposed structures will be founded on or within native deposits of silty clay and/or glacial till deposits having a compact to very 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 GEMTEC's opinion, the soils at this site are not considered to be liquefiable during the design earthquake event.

6.3 Grade Raise Restrictions

Based on the results of the investigation, it is GEMTEC's opinion that there is no practical limit to the thickness of grade raise fill that may be placed on the Site but thicknesses greater than 4 m should be reviewed by GEMTEC.

6.4 Excavation

6.4.1 General

The excavations for the proposed structures will likely be carried out through the topsoil, fill material (including existing pavement structure), native silty clay deposits, glacial till, and possibly into bedrock. The noted soils are anticipated to be readily excavatable using conventional hydraulic excavation equipment in general.

The sides of the excavations 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 sandy fill and the glacial till (when above the groundwater level,) can be classified as Type 3. Accordingly, for excavations through these soils under the noted conditions, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, extending upwards from the bottom of the excavation. In the instances where excavation in the sandy fill or glacial till will extend below the groundwater level, these soils are classified as Type 4 soils, and therefore allowance should be made for excavation side slopes of 3 horizontal to 1 vertical, or flatter, extending upwards from the bottom of the excavation. The native silty clay (regardless of whether above or below the groundwater level,) can be classified as Type 2. Accordingly, for excavations through these soils under the noted conditions, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, extending upwards from a point of 1.2 metres or less from the bottom of the excavation.

Deposits of silty clay and/or glacial till will be encountered at subgrade level of the structures and along the proposed service alignments. These deposits, when wetted or saturated, are susceptible to weakening under vibration and/or repeated loading. Appropriate excavation techniques may be required, as well as subexcavation and replacement of any disturbed material if excavation occurs in the noted condition.

Cobbles and boulders should be anticipated within the fill material and the glacial till. As such, where excavation within these materials is anticipated, an allowance should be made for removal of boulders during excavation. This may include removal of boulders that partially intrude into the sides of the excavation that must be removed to render the excavation safe or during the advancement of a trench box (if used), which may result in a wider excavation than anticipated. Further, additional backfill and/or bedding material may be required to fill any voids left from the removal of boulders.

Given the relatively shallow depth of overburden soils, as well as the relative level of the groundwater within the overburden, it is GEMTEC’s opinion that the excavations will have an adequate factor of safety (i.e., greater than 2) against basal instability.

6.4.2 Bedrock Excavation

It is expected that some bedrock excavation will be required for foundations and site services.

The bedrock at the project site consists of strong to very strong sandstone. The RQD of the bedrock is very poor to good, although it is more generally good to very good.

Depending on the depth of the excavations into the bedrock, it is anticipated that limited bedrock removal could be carried out using hoe ramming techniques. Line drilling on close centres ahead of the hoe ramming may be required.

If large volumes of deeper rock are to be removed, blasting may be quicker and more cost effective.

Provided that good bedrock excavation techniques are used, the bedrock could be excavated using near vertical side walls. Any loose bedrock should be scaled from the sides of the excavation.

The vibration effects of hoe ramming are usually minor and localized. Monitoring of the hoe ramming could be carried out, at least initially, to measure the vibrations to ensure that they are below the acceptable threshold value.

Any blasting should be carried out under the supervision of a blasting specialist engineer. As a guideline for blasting, the peak vibration limits suggested at the nearest structure or service are provided in Table 6.1.

Table 6.1 – Peak Vibration Limits

Frequency of Vibration (Hz)	Vibration Limits (millimetres/second)
<10	5
10 to 40	5 to 50 (interpolated)
>40	50

It is pointed out that the limits provided in Table 6.1, although conservative, were established to prevent damage to existing buildings and services in good condition; more stringent criteria may be required to prevent damage to freshly placed (uncured) concrete or vibration sensitive equipment or utilities. Monitoring of the blasting should be carried out to ensure that the blasting meets the limiting vibration criteria. Pre-construction condition surveys of nearby structures and existing buried services are considered essential. The effects due to vibration from blasting can be controlled by limiting the size and amount of charge, using delayed detonation techniques, and the like. To reduce the effects of vibration on nearby services, we suggest that the separation distance between any blasting and existing underground services be at least 6 metres. Any bedrock removal within these limits could be carried out using hoe ramming techniques in conjunction with line drilling on close centres. It is noted that the cost of bedrock removal generally increases the closer the bedrock removal is to any existing structures or services.

The bedrock may contain numerous irregular discontinuities. As such, significant overbreak and underbreak should be expected in any bedrock removal. The bedrock below founding level will may break unevenly below the design depth of the footings which may necessitate thickening of the footings, lowering of the footings, and/or grade raise methods below the footings (see following sections for further information).

6.4.3 Excavation Adjacent to Existing Services

For excavations planned adjacent to existing services, it is recommended that the excavations do not encroach within a line extending downwards and outwards at an inclination of 1 vertical to 2 horizontal from the base of the existing services (or edge of structural foundations). Where this is not possible, a detailed assessment by the geotechnical engineer is recommended.

It is recommended that an assessment be carried out in regard to the position of any services that are sensitive to movement within the above limits and the tolerable limits of movement for the services be established in advance of construction. The contractor should be required to clearly identify the proposed method of maintaining movements below the tolerable limits and the actions to be carried out should movements approach the limits.

6.5 Reuse of Excavated Materials

In general, the native overburden soils are considered to be frost susceptible and therefore not suitable for use in applications where non-frost susceptible, granular materials (i.e., OPSS Granular A or B Type I and II) are typically recommended. However, the native overburden soils could potentially be reused for applications such as grade raise fill in appropriate circumstances (as discussed in further detail in the following sections).

While the reuse of these materials is feasible, the characteristics of the soils (e.g., sensitivity to disturbance and change in moisture) and their condition at the time of reuse (i.e. moisture content) may make reuse challenging. A geotechnical professional should be consulted during the construction phases to further assess the feasibility of reusing the materials. Further assessment may include additional gradation analyses as well as testing to determine the Standard Proctor maximum dry density value of the materials to ensure that compaction testing results on the materials are representative. Further, measures (i.e., drying) may be required prior to use to ensure a feasible working condition.

The native soils at the site are not suitable for reuse as engineered fill for structures.

6.6 Groundwater Management

Excavations may extend through the native silty clay and glacial till and into sandstone bedrock.

Groundwater inflow from the overburden and bedrock can likely be managed using typical construction dewatering techniques, however, the inflow from the sandstone is likely to be significant and require considerably increased effort.

The noted groundwater levels may not represent the seasonal high groundwater level, nor future conditions, as the groundwater level will fluctuate seasonally and during periods of notable precipitation, as well as possibly due to construction activities in the area of the project.

6.6.1 Water Taking Permitting and Approvals

As part of the upcoming changes to *Ontario Regulation 387/04* under the *Ontario Water Resources Act* (effective July 1, 2025) all groundwater takings over 50,000 litres per day for construction dewatering will be subject to Environmental Activity and Sector Registry (EASR). A Category 3 Permit To Take Water (PTTW) is no longer required for groundwater takings over 400,000 litres per day for construction dewatering.

EASR registration must be supported by a Water Taking and Discharge Plan report prepared by a Qualified Professional. It is anticipated that an EASR will be required to support services installations at the site.

6.6.2 Groundwater Quality

Recovery of a groundwater sample for comparison to the City of Ottawa storm and sanitary sewer use bylaws was not within GEMTEC's scope of work.

Water quality sampling should be carried as part of the EASR process to assess if groundwater discharge will meet the City of Ottawa Sewer Use by-law requirements for discharge to a sanitary or combined sewer – noting that given the site usage, it is not anticipated that groundwater cannot be discharged to a storm sewer. Should exceedances be observed, it may be necessary to treat the groundwater or dispose of it at an alternative suitable location.

6.7 Proposed Building and Pump Islands

Based on the available, conceptual information, the final grades at the development will be similar to existing.

The design frost penetration for shallow foundations at heated structures is 1.5 m and for isolated, unheated foundations the design frost penetration depth is 1.8 m. The native soils (and potentially the bedrock) at this site are frost susceptible and foundations should be provided with earth cover equal to at least the design frost penetration depth.

In addition, the bedrock at this site is variable in quality and depth. For settlement compatibility, foundations should be supported entirely on one medium (i.e., soil or rock). If it is desired to support the structures on bedrock, then mass concrete should be used where the bedrock surface is below the underside of footing elevation to raise the founding grade. Alternatively, if the foundations are supported on native soil, it is recommended that a minimum of 0.3 metres of suitable native soil (or engineered fill) remain between the underside of the foundations and the bedrock surface. If this is not achievable (where less than 0.3 m of soil or engineered fill will remain) if may be necessary to subexcavate the native soil to expose the bedrock and use mass concrete as described to raise the founding grade.

The above should be considered general guidelines to be used for preliminary planning and the details should be reviewed as the design progresses.

6.7.1 Subgrade Preparation and Placement of Engineered Fill

Any existing topsoil, organic material, fill, disturbed soil, and/or loosened bedrock should be removed from below the proposed structures.

Imported granular material (engineered fill) should be used to raise the grade in areas where the proposed founding level is above the level of the native soil or where subexcavation of material is required below proposed founding level. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II or Granular A and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.3 metres beyond the footings and then down and out from the edges of the footings at 1 horizontal to 1 vertical, or flatter. The excavations should be sized to accommodate this fill placement.

6.7.2 Spread Footing Design

Based on the results of the current investigation, the proposed structures could be founded on conventional footings bearing on or within native, undisturbed clay and/or glacial till, or bedrock, or on a pad of compacted engineered fill above the native soil deposits and/or bedrock.

Spread footing foundations should be sized using the preliminary net geotechnical reactions at Serviceability Limit State (SLS) and factored net geotechnical resistances at Ultimate Limit States (ULS) provided in Table 6.2.

Table 6.2 – Foundation Bearing Values

Subgrade Material	Net Geotechnical Reaction at Serviceability Limit State[1] (kilopascals)	Factored Net Geotechnical Resistance at Ultimate Limit State[2] (kilopascals)
Native Undisturbed Clay and/or Glacial Till (or compacted engineered fill overlying native soil)	150	250
Compacted Engineered Fill (overlying bedrock)	150	250
Sound Bedrock	-	1000

Notes:

1. Provided that all loose or disturbed soil and bedrock is removed from the bearing surfaces and the engineered fill material is prepared as described above, the post construction total and differential settlement of the footings at SLS should be less than 25 and 20 millimetres respectively. Settlement of footings founded on sound bedrock is considered to be negligible.
2. Determined using a bearing resistance factor of 0.5 (Canadian Foundation Engineering Manual, Table 6.2, 5th edition, CGS, 2023).
3. The use of insulation below foundations/structures may affect the provided allowable bearing values. It is recommended that the geotechnical engineer be consulted where insulation will be placed below the foundations.

To reduce the potential for cracking in footings, foundation walls, and concrete slabs on grade, where the footings transition between different subgrade materials, the structures could potentially be reinforced within the transition areas, as recommended by a structural engineer.

In regard to footing design for the proposed canopy, consideration should be given to ensuring that the eccentric loading is minimized to prevent large moments on the foundations or potential uplift. The foundation eccentricity should not exceed 1/6 of the foundation width or length (i.e., B/6 or L/6). If the loading eccentricity exceeds this dimension, uplift restraint may need to be provided.

The preliminary bearing capacities provided by GEMTEC are guidelines for the structural engineer to design the appropriate foundation. The resulting foundation design should be confirmed with GEMTEC to verify that the proposed size, shape, depth, and inclination factors will not affect the bearing capacity of the foundation.

6.7.3 Caisson Foundations for Canopy

As noted above, for predictable performance of structures uncontrolled fill material is typically removed from below foundation support zones such that the foundations are supported by native (undisturbed) soil and/or bedrock or controlled engineered fill placed over the undisturbed soils and/or bedrock. At the Site, adopting this approach and supporting the canopy on conventional spread footings would require excavation and replacement of the fill material which would increase the size of the excavated area, may require increased dewatering effort, and would have to take into consideration the handling and disposal of potentially contaminated material. In this regard, a foundation alternative such as socketed caissons could be considered.

To avoid potential issues with differential settlement, the concrete caissons should be advanced to the surface of, or within, the bedrock. For preliminary design purposes the unit bearing resistance at ULS for the sound bedrock could be taken as 1,000 kilopascals. Settlement of the caissons should be minimal provided that the base of the caisson excavation is cleaned of loosened material including fractured bedrock. A resistance factor of 0.4 has been applied to the bearing resistance.

Caissons socketed in bedrock can resist lateral and uplift forces (e.g., from wind, earthquakes, or lateral soil pressure). The capacity of the socket depends significantly on the quality of the bedrock and the strength of the concrete used. As a design example, assuming 35 MPa concrete and a socket advanced within sound bedrock, a unit shear resistance of 500 kPa could be used between the bedrock and the concrete. A resistance factor of 0.3 has been applied to the shearing resistance.

Considerations associated with the installation concrete caissons include:

- Dense and very dense zones within the fill material (whether from debris within the fill or the presence of cobbles and boulders or other debris), and the presence of cobbles and boulders within the glacial till may increase the difficulty of augering for caissons or may require additional equipment or techniques to achieve the required caisson lengths.
- Ensuring proper cleaning and preparation of the bedrock surface for end-bearing can be challenging if the rock surface is irregular or contains fractures. Additional challenges may be encountered resulting from groundwater inflow. Casing and/or tremie techniques may be required.
- If the borehole is not adequately cleaned or if the concrete placement is not properly controlled, defects such as voids or weak zones may compromise the foundation's performance.
- Unstable or loose fill material may require temporary casing or drilling fluids (e.g., bentonite or polymer slurry) to maintain borehole stability, increasing construction costs and complexity.
- Potential for negative skin friction to occur.

Based on the thickness of the fill material and the assumed presence of numerous cobbles and boulders, larger piling equipment may be required (cranes), and access may be an issue.

6.7.4 Foundation Backfill

6.7.4.1 Foundation Wall (Frost Wall) Backfill

To avoid adfreeze and possible jacking (heaving) of the foundation walls, the interior and exterior of the foundation walls should be backfilled with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II.

6.7.4.2 Isolated Pier Backfill

The backfill against isolated piers should consist of free draining, non-frost susceptible material, such as sand/sand and gravel meeting OPSS Granular B Type I or II requirements.

Other measures to prevent frost jacking of these foundation elements could be provided, if required.

6.7.4.3 Backfill below Hardscape and Softscape Areas

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 for the selected material using suitable compaction equipment. A frost taper with a minimum slope of 5 horizontal to 1 vertical should also be provided under paved surfaces to reduce the magnitude of differential frost induced heaving of the paved surface.

Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable (i.e. soft landscaping), the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value for the selected material.

6.7.5 Slab on Grade Support (Heated Areas Only)

To prevent long term settlement of the floor slab, all organic material, topsoil layers and any existing fill material, should be removed from below the proposed slab to expose the native overburden deposits.

The subgrade surface should then be proof rolled with suitable vibratory compaction equipment under dry conditions and any noted soft or disturbed areas should be sub-excavated, subject to inspection of the geotechnical engineer. Any soft areas evident from the proof rolling should be subexcavated and replaced with compacted engineered fill meeting the OPSS requirements for OPSS Granular B Type II.

The granular base for the proposed slab on grade should consist of at least 200 millimetres of OPSS Granular A. Due to the presence of sandy soils at the site, clear stone should not be used unless additional measures are carried out. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the floor slab be composed of virgin material only for environmental reasons.

All imported granular materials placed below the proposed floor slab should be compacted in maximum 200-millimetre-thick lifts to at least 95 percent of the material's standard Proctor maximum dry density value using suitable vibratory compaction equipment.

The floor slabs should be wet cured to minimize shrinkage cracking and slab curling. The slab could be saw cut to about 1/3 the thickness of the slab as soon as curing of the concrete permits, in order to minimize shrinkage cracks.

Proper moisture protection with a vapour retarder should be used for floor slabs where the floor will be covered by moisture sensitive flooring materials or where moisture sensitive equipment, products or environments will exist. The "Guide for Concrete Floor and Slab Construction", ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the floor slabs.

Perimeter foundation drainage is not typically required for slab at grade structures. Underfloor drainage is not considered necessary provided that the floor slabs are above the finished exterior ground surface level.

6.7.6 Frost Protection

6.7.6.1 Foundations

All exterior footings, adjacent to heated areas of the buildings, should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) footings, such as for piers, should be provided with at least 1.8 metres of earth cover for frost protection purposes. If the foundation walls/floor slabs are insulated in a manner that will reduce heat transfer (loss) to the surrounding soil at founding (footing) level, the frost protection requirements for exterior footings should then conform to that required for foundations for an unheated space/isolated footings (i.e., 1.8 metres).

Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Where insulation will be placed below the foundations, the provided bearing pressure values for foundations may require adjustment.

6.7.6.2 Slab on Grade

It is assumed that the proposed building will be continuously heated and as such the use of insulation below the floor slab is not considered necessary. Slabs in non-heated areas may required additional measures and this should be reviewed by a geotechnical professional.

6.8 Proposed Site Services

The details of the proposed service installations (i.e. sewers, water service) were not available at the time of preparation of this report and therefore this aspect of the works has not been addressed in detail. Further guidance can be provided as the design develops further.

6.8.1 Excavation

As general guidance, the commentary provided in Section 6.4 should be considered.

The sides of the excavations within overburden soils should be sloped as discussed in the noted section. As an alternative or where space constraints dictate, the service installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

Excavation of the glacial till overburden above the groundwater level should not present significant constraints. Below the groundwater level, sloughing of the deposits into the excavation should be anticipated as well as possible disturbance to the soils in the bottom of the excavation. These deposits are susceptible to weakening under vibration and/or repeated loading and it is suggested that final trimming to subgrade level be carried out using a hydraulic shovel equipped with a flat blade bucket. We recommend that a contingency allowance be made in the contract for a 300 millimetre thick subbedding layer of OPSS Granular B Type II granular material in the event that the subgrade soils are disturbed during construction.

If bedrock excavation is required for proposed site services, bedrock excavation should be carried out in accordance with the recommendations provided in Section 6.4.2 above.

As the servicing plans are advanced, they should be reviewed in advance of construction to assess for conformance to the recommendations provided herein.

6.8.2 Groundwater Management

As general guidance, the commentary provided in Section 6.6 should be reviewed.

6.8.3 Pipe Bedding and Cover

The bedding for the sewers and watermains should be in accordance with OPSD 802.010/802.013 and 802.031/802.033 for flexible and rigid pipes, respectively. The pipe bedding 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 and Granular B Type II material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used in the service trenches be composed of virgin (i.e., not recycled) material only.

Allowance should be made for subexcavation of any existing fill, organic deposits, or disturbed material encountered at subgrade level.

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 use of clear crushed stone should not be permitted for the installation of site services as the migration of the native soil into the voids in the clear crushed stone could result in settlement of the pipe.

Where boulders are encountered at subgrade level, additional bedding material may be required to fill any voids left following the removal of boulders. Further, where bedrock removal is required, the use of additional granular bedding should be anticipated, since vertical overbreak of the bedrock will naturally occur along the bedding planes in the bedrock.

The bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

6.8.4 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, 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 section of roadway. 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. The depth of frost penetration in areas that are kept clear of snow and where trench backfill consists of broadly graded shattered rock fill or earth fill is expected to be about 1.8 metres. It is our experience, however, that the frost penetration can be as much as 2.4 metres when the trench backfill consists solely of relatively open graded rock fill. Where cover requirements are not practicable, the pipes could be protected from frost using a combination of earth cover and insulation. Further details regarding insulation could be provided, if required.

Based on the results of the soil classification testing on the inorganic overburden material samples, most of the overburden soil will be acceptable for reuse as trench backfill, however additional assessment by a geotechnical professional at the time of construction is recommended.

At the time of construction, the silty clay and glacial till deposits from the excavations could have moisture contents above optimum for compaction. Furthermore, most of the overburden deposits at this site are sensitive to changes in moisture content. Unless these materials are allowed to dry, the specified densities will not likely be possible to achieve and, as a consequence, some settlement of these backfill materials could occur. 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 paving of any roadways for 6 months, or longer, to allow some the trench backfill settlement to occur and thereby improve the final roadway appearance.
- Reuse any wet materials outside hard surfaced areas and where post construction settlement is less of a concern (such as landscaped areas).

Topsoil or other organic material and boulders from the glacial till should be wasted from the trench.

If blast rock is used as backfill within the service trench, it should be mostly 300 millimetres, or smaller, in size and should be well graded. To prevent ingress of fine material into voids in the blast rock, the upper surface of the blast rock should be covered with a thin layer of well graded crushed stone (e.g. OPSS Granular B Type II).

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, curbs, driveways, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. Rock fill should be placed in maximum 500 millimetre thick lifts and compacted with a large drum roller, the haulage and spreading equipment, or a combination of both. The specified density for compaction of the backfill materials may be reduced where the trench backfill is not located below or in close proximity to existing or future areas of hard surfacing and/or structures.

Some of the soils that exist at this site are highly frost susceptible and are prone to significant ice lensing. In order to carry out the work during freezing temperatures and maintain adequate performance of the trench backfill as a roadway subgrade, the service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The sides of the trenches 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.

6.8.5 Post-Construction Settlement

The design of the services should consider some differential settlement between the various areas of the site. The amount of differential settlement is expected to be less than 25 millimetres.

6.9 Proposed Roadways and Parking Areas

The final details of any proposed roadways and parking areas were not available at the time of preparation of this report and therefore these aspects of the works have not been described in detail. Preliminary guidance is provided below and further details can be provided upon request as the design develops.

6.9.1 Subgrade Preparation

In preparation for roadway construction at this site, all surficial topsoil, fill material and any soft, wet or deleterious materials should be removed from the proposed roadway areas.

Prior to placing granular material for the internal roads, the exposed subgrade should be inspected and approved by geotechnical personnel. Any soft areas should be subexcavated and replaced with suitable (dry) earth borrow or well shattered and graded rock fill material that is frost compatible with the materials exposed on the sides of the area of subexcavation.

Similarly, should it be necessary to raise the roadway grades at this site, material which meets OPSS specifications for Select Subgrade Material, earth borrow or well shattered and graded rock fill material may be used.

The select subgrade material or earth borrow 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. Rock fill should also be placed in thin lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both.

Truck traffic should be avoided on the native soil subgrade or the trench backfill within the roadways especially under wet conditions.

6.9.2 Flexible Pavement Structures for the Parking Areas and Access Roadway

It is suggested that paved areas be constructed using the following minimum pavement structure in consideration for light duty levels (i.e., primarily parking of passenger vehicles):

- 60 millimetres of asphaltic concrete (Superpave 12.5 (Traffic Level B) Hot Mix Asphalt (HMA) meeting the requirements of OPSS 1151); over
- 150 millimetres of OPSS Granular A base; over
- 300 millimetres of OPSS Granular B Type II subbase.

It is suggested that paved areas be constructed using the following minimum pavement structure in consideration for regular and heavy-duty levels (i.e., primary driving lanes, commercial vehicle access routes, fire routes, etc.):

- 100 millimetres of asphaltic concrete (40 millimetres of Superpave 12.5 (Traffic Level B) over 60 millimetres of Superpave 19.0 (Traffic Level B) Hot Mix Asphalt (HMA) meeting the requirements of OPSS 1151); over
- 150 millimetres of OPSS Granular A base; over
- 450 millimetres of OPSS Granular B Type II subbase.

All HMA materials should incorporate PG 58-34 asphaltic cement meeting the requirements of OPSS 1101 and be constructed to the requirements of OPSS 310.

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 (frost tapers).

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.

6.9.3 Compaction Requirements

The granular base and subbase materials should be compacted in maximum 300 millimetre thick lifts to at least 99 percent of the standard Proctor maximum dry density value.

6.9.4 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long-term performance of the pavement at this site. The subgrade surfaces should be shaped to drain to the catch basins to promote drainage of the pavement granular materials. The catch basins should be provided with minimum 3-metre long perforated stub drains which extend in at least two (2) directions from each catch basin at pavement subgrade level.

6.9.5 Pavement Transitions

Where the new pavement will abut existing pavements the following is suggested to improve the performance of the joint between the new and the existing pavements:

- Neatly saw cut the existing asphaltic concrete;
- Remove the asphaltic concrete and slope the bottom of the excavation within the existing granular base and subbase at 1 horizontal to 1 vertical, or flatter, to avoid undermining the existing asphaltic concrete;
- To avoid cracking of the asphaltic concrete due to an abrupt change in the thickness of the roadway granular materials where new pavement areas join with the existing pavements, the granular depths should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the existing pavement structure; and,
- Remove (mill off) 40 to 50 millimetres of the existing asphaltic concrete to a distance of 300 millimetres at the joint and tack coat the asphaltic concrete at the joint in accordance with the requirements in OPSS 310.

6.10 Proposed Underground Fuel Storage Tanks

6.10.1 Excavation and Groundwater Pumping

It is understood that the service station will contain two (2) underground fuel storage tanks located adjacent to the east site limit.

Based on the investigation results, the excavation for the proposed underground storage tanks will be carried out through topsoil, fill (existing pavement structure) and native deposits of silty clay and glacial till and possibly into bedrock. Our comments on overburden excavation, bedrock excavation and groundwater pumping provided in the preceding sections apply equally to the fuel storage tanks.

6.10.2 Bedding

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

6.10.3 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, within the design frost penetration depth, between those areas of hard surfacing underlain by non-frost susceptible granular 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 5 horizontal to 1 vertical, or flatter.

For design purposes, the earth pressure parameters provided in Table 6.3 could be used to calculate the lateral earth pressure on the underground fuel storage tank.

Table 6.3 – Backfill Earth Pressure Parameters

Parameter	OPSS Granular A, Granular B Type II
Material Unit Weight, γ (kN/m^3)	22
Estimated Friction Angle (degrees)	38
"Active" Earth Pressure Coefficient, K_a, assuming horizontal backfill behind the structure	0.24

Parameter	OPSS Granular A, Granular B Type II
“Passive” Earth Pressure Coefficient, K_p, assuming horizontal backfill behind the structure	4.20
“At Rest” Earth Pressure Coefficient, K_o, assuming horizontal backfill behind the structure	0.38

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.

6.10.4 Buoyant Uplift of Tanks

The groundwater levels will likely be higher than the founding level of the tanks (the groundwater level will fluctuate 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 to be at elevation 101.0 m for buoyancy computations.

6.11 Rock Anchors

Rock anchors may be required for the proposed structures (i.e. pump island canopy, USTs) potentially to resist various forces (i.e., overturning, uplift and buoyancy). Grouted or mechanical rock anchors may be used for these purposes.

The design, construction, and testing of anchors should be carried out in accordance with OPSS 942. The design of the grouted rock anchors should consider the following failure modes:

- Failure within the rock mass or rock cone pull-out;
- Failure of the rock / grout bond;
- Failure of the grout / tendon bond; and,
- Failure of the steel tendon or top anchorage.

Of the failure modes identified above – failure of the tendon / grout bond, and failure of the tendon or top anchorage should be checked by a structural engineer.

As discussed, and noted on the borehole logs in Appendix B, the bedrock consists of strong to very strong sandstone rock. Between the depth of the top of the bedrock to about 3 metres below it, it appears that the overall quality of the bedrock is good to excellent (RQD above 80%). However, within borehole 25-03, at a depth of about 3 metres below the top of the bedrock surface, the quality of the bedrock is very poor (RQD of about 0%). As such, it is recommended that anchor resistance should be designed to mobilize within the upper 3 metres of the bedrock surface.

Anchor resistance, (Q_r) for a single anchor against failure within the rock mass can be determined from the equation for the volume of a cone, and in this case assuming a 90 degree cone apex angle, with the apex located at the mid-point of the fixed length section (a 90 degree angle is suggested based on the characteristics of the bedrock core recovered within the upper 3 metres of the bedrock; where a deeper anchor may be required, additional assessment may be required and the provided cone apex angle revised). The equation for anchor resistance for failure within the rock mass is provided below, neglecting shear resistance generated along the cone surface:

$$Q_r = \O*0.33*\pi*\gamma' D^3 \text{Tan}^2\theta$$

Where:

γ' = Buoyant unit weight of rock may be taken as about 16 kilonewtons per cubic metre (conservative value)

$\O$ = Resistance factor to be applied

D = cone height (anchor midpoint)

θ = Half the value of the apex angle.

Where loads are off vertical the capacity of the anchor should be modified according to the angle of application.

Group effects should be considered in assessing anchor capacity where overlapping occurs between adjacent cones. For this case, the volume of a truncated trapezoidal failure zone should be considered. However, for preliminary design purposes we suggest anchors should not be spaced closer than about 1.5 metres to reduce the potential for drillholes to intersect and avoid overstressed areas of bedrock.

For failure of the grout/rock bond the unfactored ULS bond strength at concrete to rock interface pull out use a value of 1,000 kilopascals (assuming a resistance factor of 0.3 to 0.4 is applied). This value assumes that the fixed anchor length is in rock of good to excellent quality with strong or better strength. To achieve the bond strength the surface of the rock bores should be rough and all debris and rock flour should be cleared from the bore or the anchor capacity shall be reduced as a result. The required bonded length should be determined according to the factored tensile resistance to be carried.

Long bonded anchor lengths should be avoided i.e., maximum of 8 metres. SLS movement in the anchor can be determined from the elastic elongation of the unbonded portion of the tendon under design load.

Rock anchors to be tested at time of construction by proof load testing to 1.5 times the anchor service load on at least 10 percent of the anchors (or according to Ontario Provincial Standards, whichever is the more stringent requirement). In this instance a resistance factor of 0.4 could be applied. If more than one type of rock anchor is to be installed it is recommended that one type of each anchor is tested to failure.

Corrosion protection of the anchor system should be provided which is adequate for the design life of the system. For permanent elements the rock anchors 'double corrosion protection' systems should be used.

The use of a specialist rock anchor contractor is recommended for installation of the anchors. The installation and testing of rock anchors shall be observed by a suitably qualified and experienced geotechnical practitioner.

6.12 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the samples of the native overburden soil was 19 and 69 micrograms per gram. According to the Canadian Standards Association “Concrete Materials and Methods of Concrete Construction” (CSA A23.1-14 Table 3), the degree of sulphate exposure stemming from the overburden soils can be considered as low. Therefore, any concrete in contact with the native soils at this site should be batched with appropriate cementing materials. Other factors (structurally reinforced or non-structurally reinforced, freeze-thaw environment, chloride exposure, etc.) should also be considered in selecting the cementing materials as well as the associated air entrainment and concrete mix proportions for any concrete.

Based on the pH and resistivity of the soil samples, the materials can be classified as non-aggressive towards unprotected steel. The manufacturer of any buried steel elements that will be in contact with the native soils should be consulted to ensure that the durability of the intended product is appropriate. It is noted that the corrosivity of the soil could vary throughout the year due to the application of de-icing chemicals. As well, the samples of soils that were analyzed may not be representative of all the native overburden soils across the Site.

7.0 ADDITIONAL CONSIDERATIONS

7.1 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, blasting, etc.) will cause ground vibration on and off the site. The vibrations will attenuate with distance from the source but may be felt at nearby structures.

Consideration could be given to carrying out preconstruction surveys 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.

7.2 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.

7.3 Winter Construction

Some of the native soils that exist at this site are considered to be frost susceptible and prone to ice lensing. In the event that construction is required during freezing temperatures, the native soils within excavations should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

The excavations should be opened for as short a time and limited in area as is reasonably practicable.

7.4 Abandonment of Monitoring Wells

The monitoring wells installed as part of this investigation should be decommissioned by a licensed well technician in accordance with Ontario Regulation 903, as amended by Ontario Regulation 128/03. The well abandonment could be carried out in advance of or during construction.

7.5 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.

8.0 CLOSURE

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.

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COPYRIGHT: The contents of this report are subject to copyright owned by GEMTEC, save to the extent that copyright has been legally assigned by us to another party or is used by GEMTEC under license. To the extent that GEMTEC owns the copyright in this report, it may not be copied without our prior written agreement for any purpose other than the purpose indicated in this report. The methodology (if any) contained in this report is provided to the Client in confidence and must not be disclosed or copied to third parties without the prior written agreement of GEMTEC. Disclosure of that information may constitute an actionable breach of confidence or may otherwise prejudice our commercial interests.

COMPLETE REPORT: This report is of a summary nature and is not intended to stand alone without reference to the instructions given to GEMTEC by the Client, communications between GEMTEC and the Client and to any other reports prepared by GEMTEC for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. GEMTEC can not be responsible for use of portions of the report without reference to the entire report.

BASIS OF REPORT: This Report has been prepared for the specific site, development, design objectives and purposes that were described to GEMTEC by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document, subject to the limitations provided herein, are only valid to the extent that this report expressly addresses the proposed development, design objectives and purposes. Any change of site conditions, purpose or development plans may alter the validity of the report and GEMTEC cannot be responsible for use of this report, or portions thereof, unless GEMTEC is requested to review any changes and, if necessary, revise the report.

TIME DEPENDENCE: If the proposed project is not undertaken by the Client within 18 months following the issuance of this report, or within the timeframe understood by GEMTEC to be contemplated by the Client, the guidance and recommendations within the report should not be considered valid unless reviewed and amended or validated by GEMTEC in writing.

USE OF THIS REPORT: The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without GEMTEC's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, GEMTEC may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

NO LEGAL REPRESENTATIONS: GEMTEC makes no representations whatsoever concerning the legal significance of its findings, or as to other legal matters touched on in this report, including but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and change. Such interpretations and regulatory changes should be reviewed with legal counsel.

DECREASE IN PROPERTY VALUE: GEMTEC shall not be responsible for any decrease, real or perceived, of the property or site's value or failure to complete a transaction, as a consequence of the information contained in this report.

RELIANCE ON PROVIDED INFORMATION: The evaluation and conclusions contained in this report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations. information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in this report as a result of misstatements, omissions, misrepresentations. or fraudulent acts of the Client or other persons providing information relied on by us. We are entitled to rely on such representations, information and instructions and are not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

INVESTIGATION LIMITATIONS: Site investigation programs are a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions but even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions.

The data derived from the site investigation program and subsequent laboratory testing are interpreted by trained personnel and extrapolated across the site to form an inferred geological representation and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Conditions between and beyond the borehole/test hole locations may differ from those encountered at the borehole/test hole locations and the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies. Accordingly, GEMTEC does not warrant or guarantee the exactness of the subsurface descriptions.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination-or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

In addition, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

SAMPLE DISPOSAL: GEMTEC will dispose of all uncontaminated soil and/or rock samples 60 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

FOLLOW-UP AND CONSTRUCTION SERVICES: All details of the design were not known at the time of submission of GEMTEC's report. GEMTEC should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of GEMTEC's report.

During construction, GEMTEC should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of GEMTEC's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in GEMTEC's report. Adequate field review, observation and testing during construction are necessary for GEMTEC to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, GEMTEC's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

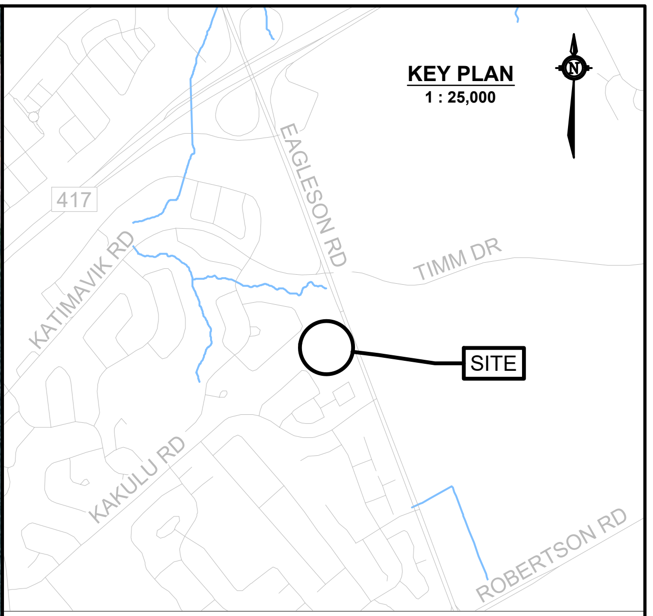
CHANGED CONDITIONS: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that GEMTEC be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that GEMTEC be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

DRAINAGE: Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. GEMTEC takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

APPENDIX A

Figure 1

Report to: AECOM
Project: 100041.032 (July 28, 2025)



LEGEND

BH #	BOREHOLE ID
XX.XX	GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM
⯐	BOREHOLE LOCATION (current investigation by GEMTEC)

DATA SOURCES AND REFERENCES

1. Coordinate system: CSRS.UTM-18N
2. Distances, elevations, and coordinates are shown in metres unless denoted otherwise
3. This drawing is a schematic representation and should not be taken as a substitute for a legal survey.
4. Image @2025 Google Maps, CNES / Airbus, First Base Solutions, Maxar Technologies
5. Contains information licensed under the Open Government Licence – Ontario
6. Geographic dataset source: Ontario GeoHub

SCALE
1:1,000
0 20 40 60m

DRAWING
SITE PLAN

CLIENT
AECOM CANADA LTD.

PROJECT
GEOTECHNICAL INVESTIGATION
SHELL SERVICE STATION
106 EAGLSEON ROAD
OTTAWA, ONTARIO

DRAWN BY SL	CHECKED BY MR
PROJECT NO. 100041.032	REVISION NO. 0
DATE JULY 2025	FIGURE NO. **FIGURE 1**

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

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Tel: (613) 836-1422
www.gemtec.ca
ottawa@gemtec.ca

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APPENDIX B

Record of Borehole Sheets
List of Abbreviations and Symbols

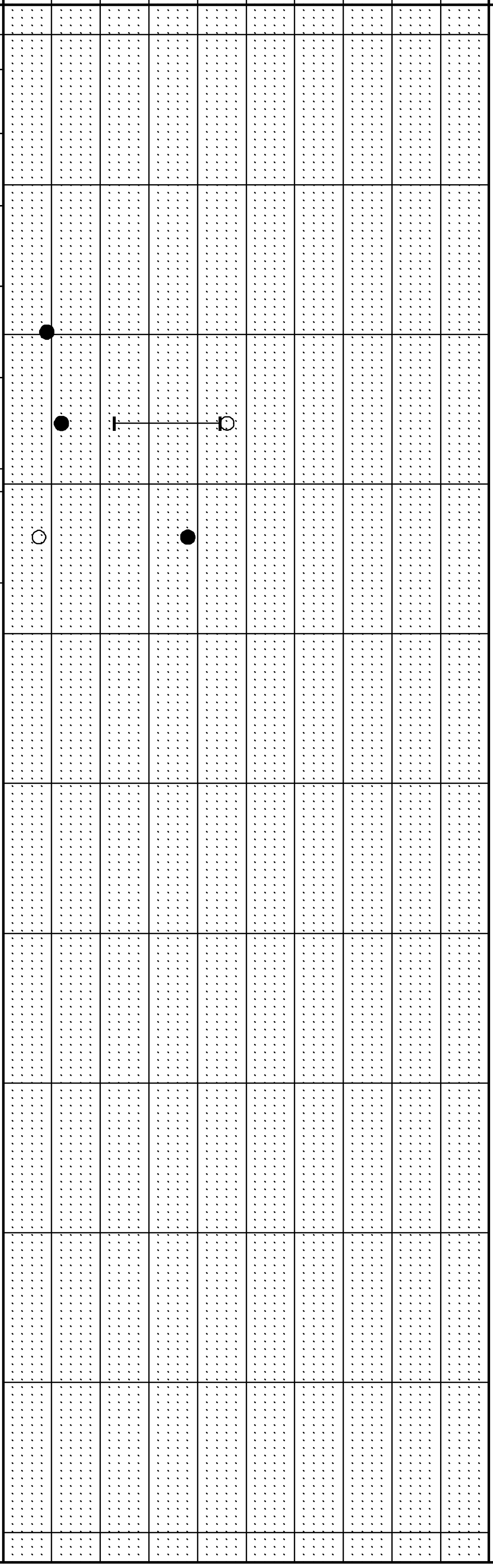
RECORD OF BOREHOLE 25-01

CLIENT: AECOM
PROJECT: Subsurface Investigation, Proposed Service Station, 106 Eagleson Rd., Ottawa, ON
JOB#: 100041.032
LOCATION: See Figure A1 - Borehole Location Plan

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jul 3 2025

● PENETRATION RESISTANCE (N), BLOWS/0.3m
▲ DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED
WATER CONTENT, % W_P ├─ W ○ ─┤ W_L

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface	101.89						
	Hydrovac	TOPSOIL - (SM) SILTY SAND; trace gravel, trace organics; brown; non-cohesive.	101.66 0.23	1	GS				
		FILL - (GP-SP) SAND and GRAVEL; with cobbles, trace organics; brown; non-cohesive.	101.23 0.66	2	GS				
1		(Increased Cobble/Boulder content)	100.75 1.14	3	GS				
		FILL - (SM) SILTY SAND; some clay, trace gravel; brown; non-cohesive.	100.21 1.68	4	GS				
2	Auger Hollow Stem Auger (210mm OD)	(no recovery - possible fill)	99.60 2.29	5	SS	0	9		Borehole backfilled with bentonite
		(CI) CLAY; some sand; grey brown (WEATHERED CRUST); cohesive, w>PL, stiff to very stiff	99.05 2.84	6	SS	457	12		
3		(SM) SILT and SAND; some clay, trace gravel; grey, (GLACIAL TILL); non-cohesive, moist, dense	98.13 3.76	7	SS	406	38	MH	
4		Auger refusal End of Borehole							No groundwater inflow observed within the open borehole at the time of drilling
5									
6									
7									
8									
9									
10									



GEO - BOREHOLE LOG 100041.032_GNT_2025-07-09_REVA PLAN B.GPJ GEMTEC 2018.GDT 7/25/25

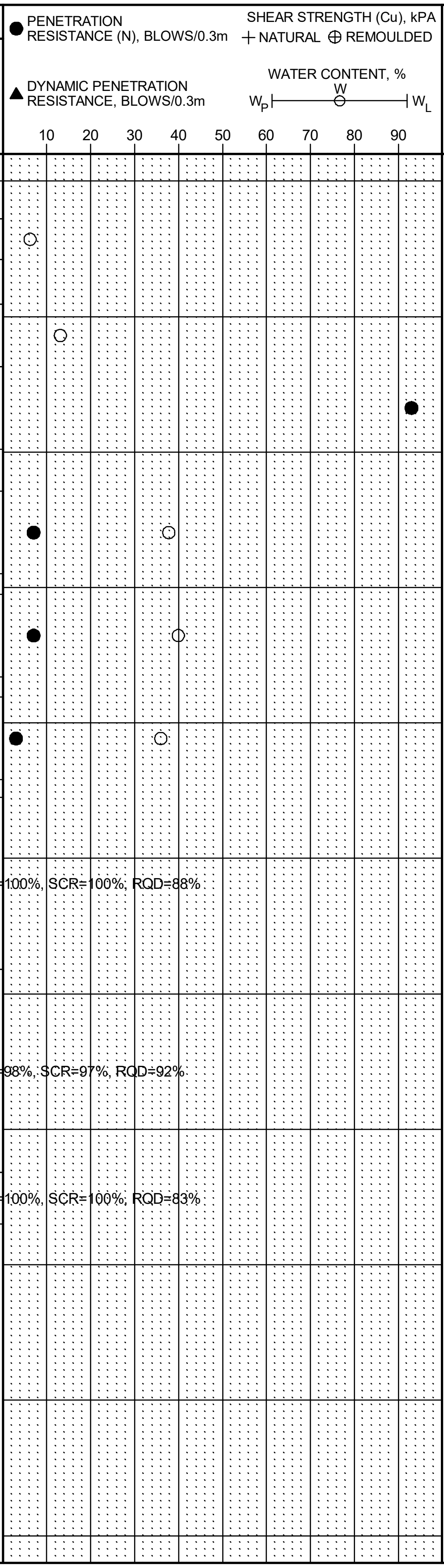
LOGGED: CD
CHECKED: MR

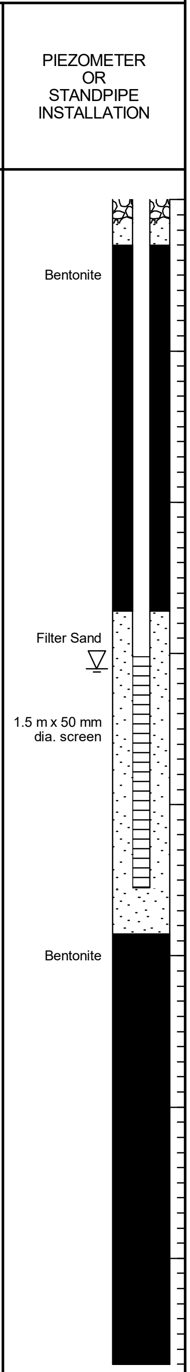
RECORD OF BOREHOLE 25-02

CLIENT: AECOM
PROJECT: Subsurface Investigation, Proposed Service Station, 106 Eagleson Rd., Ottawa, ON
JOB#: 100041.032
LOCATION: See Figure A1 - Borehole Location Plan

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jul 3 2025

BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
	Ground Surface	102.45					
Hydrovac	TOPSOIL - (SM) SILTY SAND; trace gravel, trace organics; brown; non-cohesive,dry	102.17 0.28	1	GS			
Hydrovac	FILL - (GP) GRAVEL; trace sand, trace organics; grey brown; non-cohesive, moist to wet	101.87 0.58	2	GS			
Hydrovac	(Increased Cobble/Boulder content)	101.54 0.91	3	GS			
Hydrovac	FILL - (GP-SM) SAND and GRAVEL; some silt, trace clay; brown; non-cohesive.	101.08 1.37	4	GS			M
Auger Hollow Stem Auger (210mm OD)	(no recovery - possible fill)	100.16 2.29	5	SS	0	93	
Auger Hollow Stem Auger (210mm OD)	(CI) CLAY; some sand, trace gravel; grey brown (WEATHERED CRUST); cohesive, w>PL, stiff to very stiff		6	SS	483	7	
			7	SS	457	7	MH
		98.16 4.29	8	SS	610	3	
Auger Hollow Stem Auger (210mm OD)	(SM) SILTY SAND; some gravel, some clay; grey, (GLACIAL TILL); non-cohesive, moist	97.90 4.55					
Diamond Rotary Core HQ (89mm OD)	Fresh, grey white black SANDSTONE, thinly laminated to very thinly bedded, strong to very strong rock		1	RC		TCR=100%, SCR=100%, RQD=88%	UCS
			2	RC		TCR=98%, SCR=97%, RQD=92%	
		94.75 7.70	3	RC		TCR=100%, SCR=100%, RQD=83%	
	End of borehole						





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
25/07/14	3.10 ▽	99.4

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LOGGED: CD
CHECKED: MR

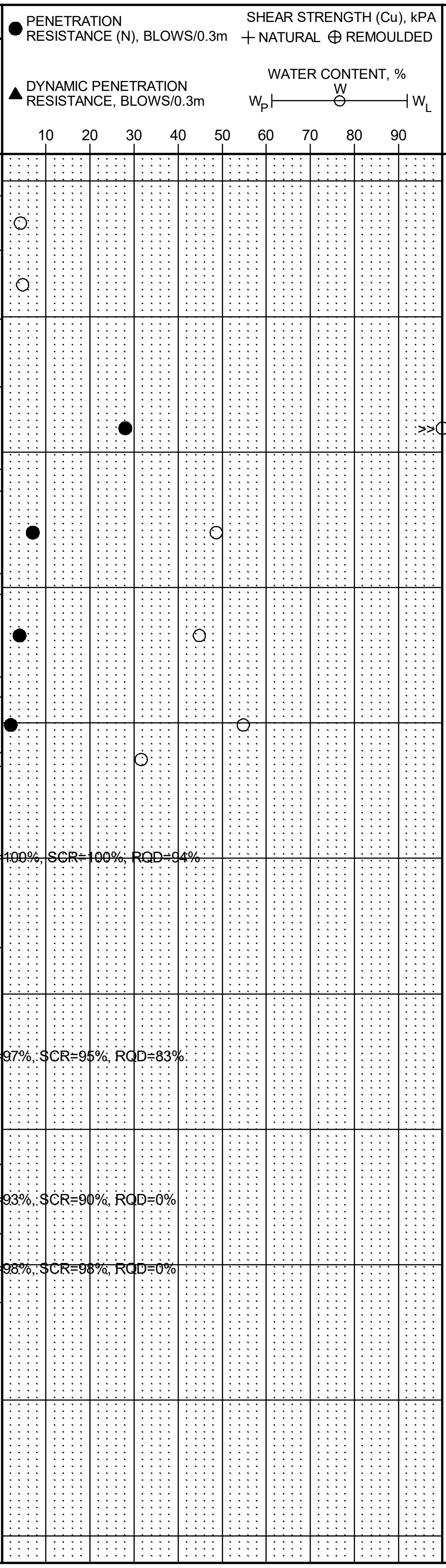
RECORD OF BOREHOLE 25-03

CLIENT: AECOM
PROJECT: Subsurface Investigation, Proposed Service Station, 106 Eagleson Rd., Ottawa, ON
JOB#: 100041.032
LOCATION: See Figure A1 - Borehole Location Plan

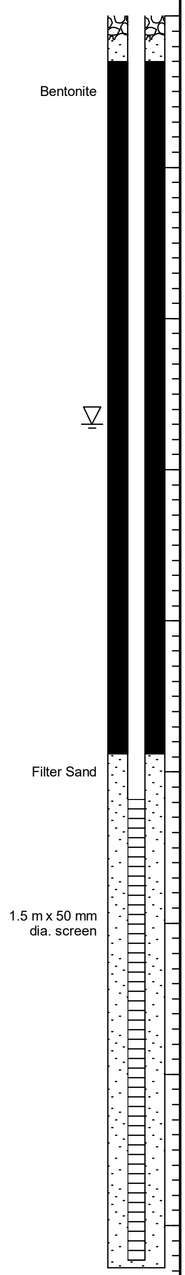
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jul 4 2025

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
	Ground Surface	102.13
Hydrovac	ASPHALTIC CONCRETE	102.02 0.11
	BASE/SUBBASE - (GP-SP) GRAVEL and SAND, some silt; with cobbles, grey brown to brown; non-cohesive	101.11 1.02
	(Increased Cobble/Boulder content)	100.61 1.52
Auger Hollow Stem Auger (210mm OD)	(SM) SILTY SAND;, trace clay, trace organics; dark brown, (TOPSOIL); non-cohesive, moist	100.00 2.13
	(CI) CLAY; trace sand; grey brown (WEATHERED CRUST); cohesive, w<PL, stiff to very stiff	97.91 4.22
	(CL) SILTY CLAY; with sand, some gravel; grey brown, (GLACIAL TILL); cohesive, moist, firm	97.81 4.32
Diamond Rotary Core HQ (89mm OD)	Fresh, grey white black SANDSTONE, thinly laminated to very thinly bedded, strong to very strong rock	93.85 8.28
	End of borehole	

SAMPLE NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	GS			M
2	GS			
3	GS			
4	SS	127	28	
5	SS	457	7	
6	SS	610	4	
7a	SS	610	2	
7b	SS			
1	RC		TCR=100%, SCR=100%, RQD=94%	
2	RC		TCR=97%, SCR=95%, RQD=83%	UCS
3	RC		TCR=93%, SCR=90%, RQD=0%	
4	RC		TCR=98%, SCR=98%, RQD=0%	



DEPTH SCALE METRES: 0 – 10



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
25/07/14	2.70 ∇	99.4

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Consulting Engineers and Scientists

LOGGED: CD
CHECKED: MR

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) 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) 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) 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

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

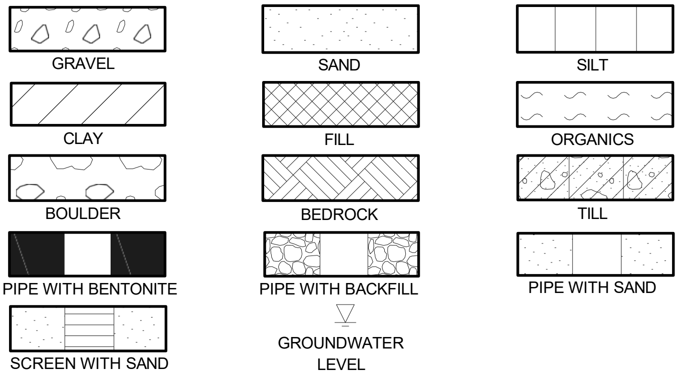
PENETRATION RESISTANCE

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.

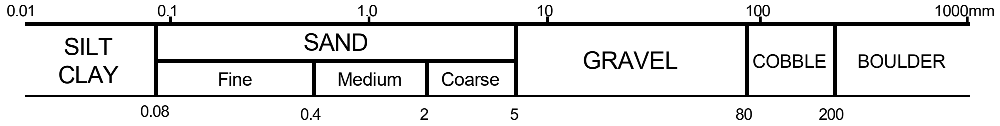
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

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

0 5 12 30

TRACE	SOME	ADJECTIVE	noun > 30% and main fraction
trace clay, etc	some gravel, etc.	silty, etc.	sand and gravel, etc.

APPENDIX C

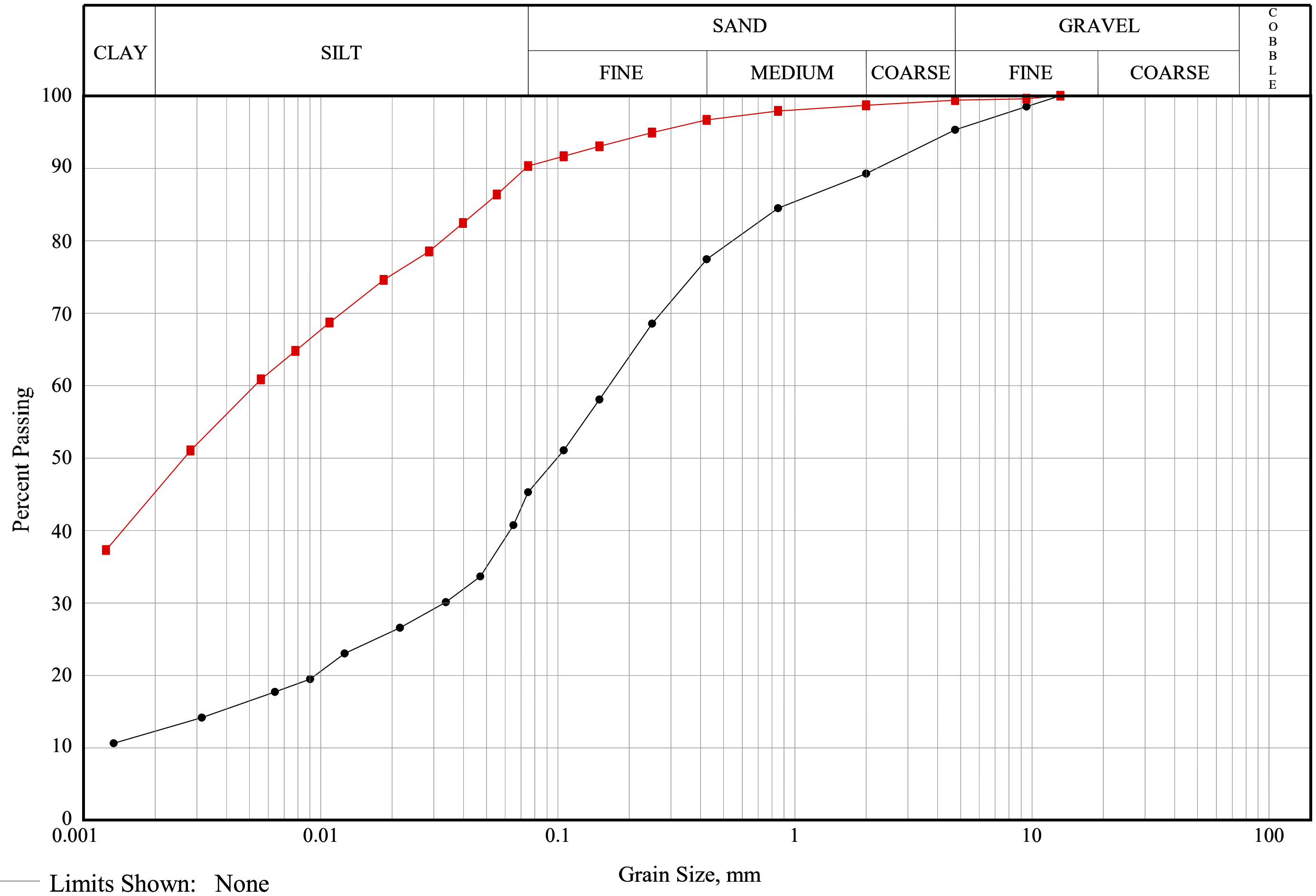
Laboratory Classification Analyses Results

Report to: AECOM
Project: 100041.032 (July 28, 2025)

GEMTEC
Consulting Engineers and Scientists

Client:	AECOM Canada Ltd.
Project:	Shell Service Station - 106 Eaglseon Rd, Ottawa
Project #:	100041032

Soils Grading Chart (LS-702/ ASTM D-422)



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	Silt and Sand, some clay, trace gravel	25-01	7	3.04-3.65	4.7	50.0	33.0	12.3
—■—	Silty Clay, some sand, trace gravel	25-02	7	3.04-3.65	0.6	9.1	45.0	45.3

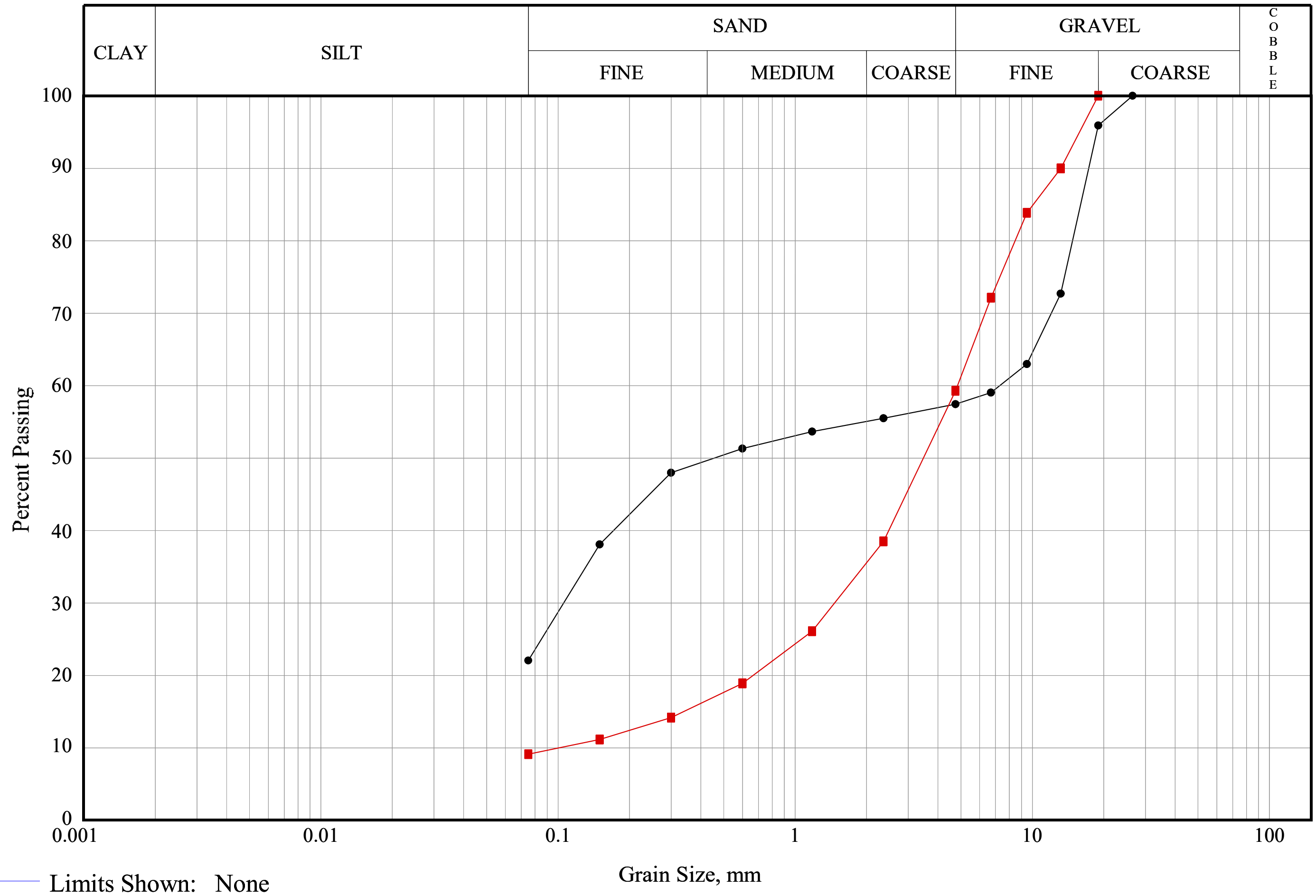
Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—		N/A	---	0.004	0.03	0.10	0.16	0.93	33.0
—■—		N/A	---	---	---	0.00	0.01	0.05	45.0

Note: More information available upon request

GEMTEC
Consulting Engineers
and Scientists

Client: AECOM Canada Ltd.
Project: Shell Service Station - 106 Eaglseon Rd, Ottawa
Project #: 100041032

Soils Grading Chart



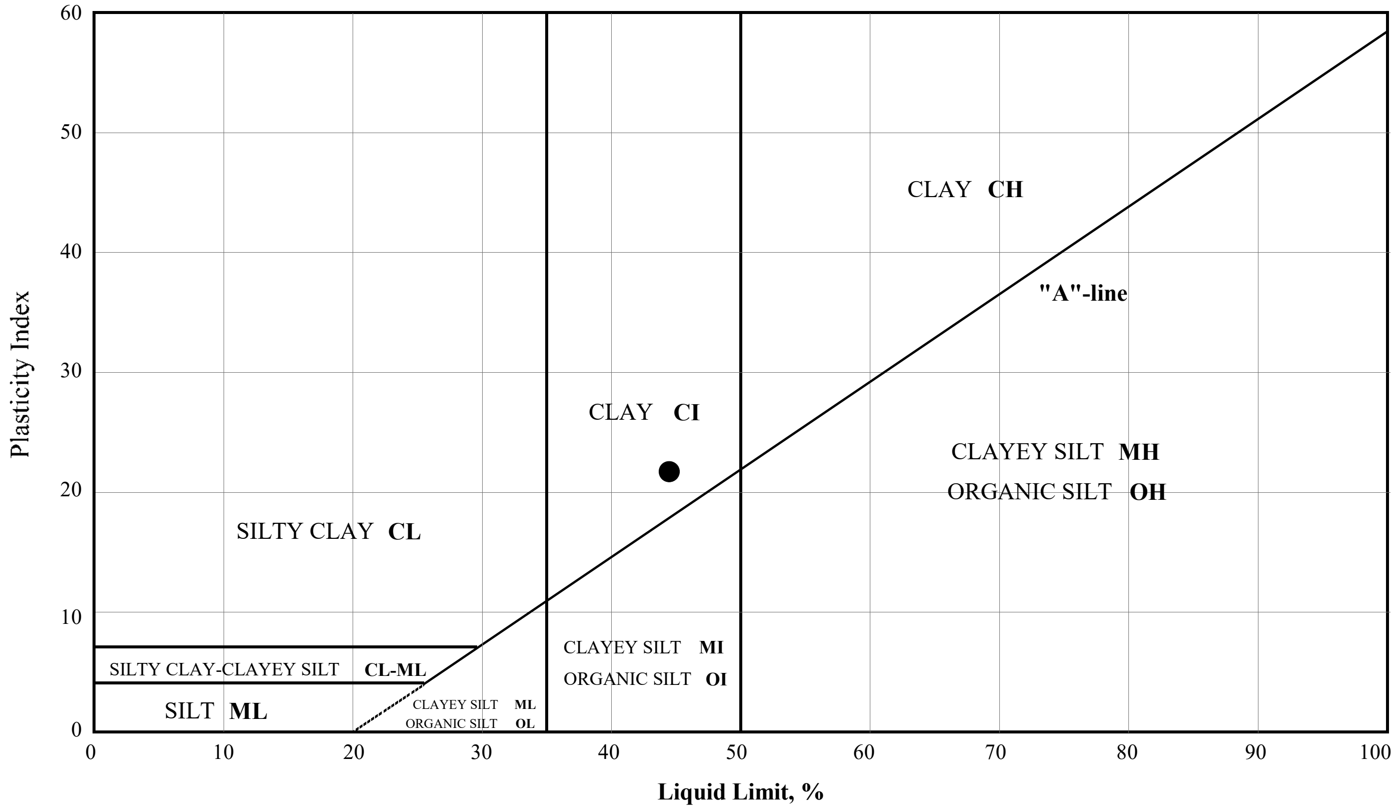
Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	Silty Sand and Gravel	25-02	4	0.914-1.372	42.6	35.4	22.1	
—■—	Gravel and Sand, some silt	25-03	1	0.114-0.508	40.7	50.2	9.1	

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—		N/A	---	---	0.11	0.46	7.30	16.01	---
—■—		N/A	0.101	0.339	1.47	3.48	4.84	10.11	---

Note: More information available upon request

GEMTEC Consulting Engineers and Scientists Ottawa, ON	Client: AECOM Canada Ltd. Project: Shell Service Station - 106 Eaglseon Rd, Ottawa Project #: 100041032	Plasticity Chart (LS-7034/ASTM D4318)



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	25-01	6	2.28-2.89	44.5	22.8	22	N/A	46.1

Note: More information available upon request

GEMTEC
CONSULTING ENGINEERS
AND SCIENTISTS

Client: AECOM Canada Ltd.
Project: Shell Service Station - 106 Eaglseon Rd, Ottawa
Project #: 100041032

Rock Core Compressive Strength

Date/Time Sampled: 25/07/14 3:35:00 PM

Date/Time Tested: 25/07/14 3:36:42 PM

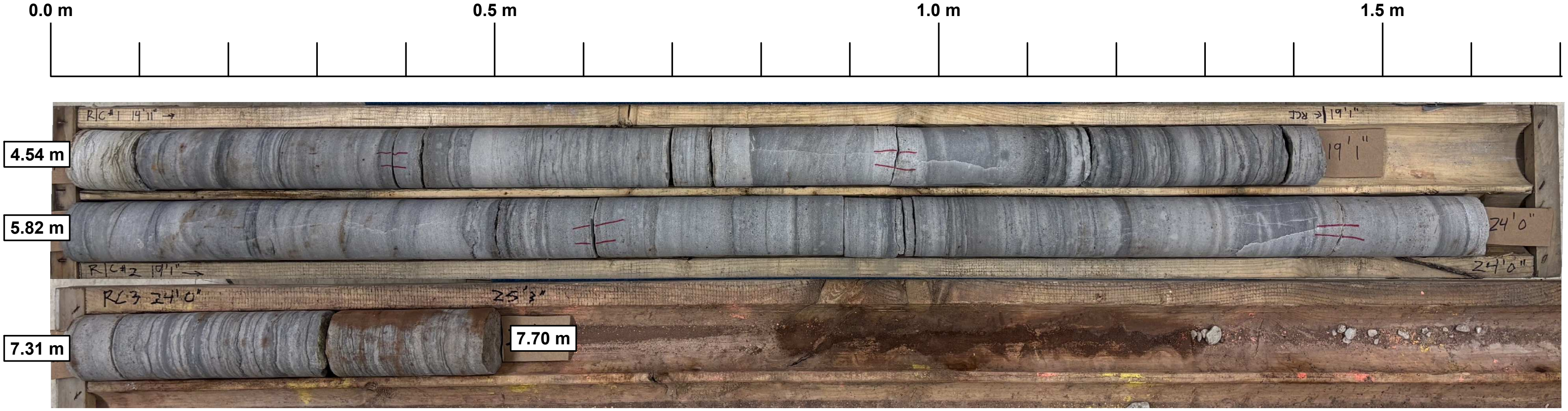
BH	Sample No	Depth	Description	Diameter, mm	Area, mm²	Length After Capping, mm	L/D	Load, kN	Comp. Str., MPa
25-02	RC1	4.64-4.85		63.2	3132	128	2.03	240.810	76.9
25-03	RC2	5.71-5.94		63.0	3117	128	2.04	326.480	104.7

APPENDIX D

Photographs of Bedrock Core Samples

Report to: AECOM
Project: 100041.032 (July 28, 2025)

BOREHOLE: BH25-02
BORING DATE: JULY 3, 2025
DEPTH: 4.54 TO 7.70 METRES BELOW GROUND SURFACE



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Project
AECOM CANADA LTD.
GEOTECHNICAL INVESTIGATION
SHELL SERVICE STATION
106 EAGLSEON ROAD
OTTAWA, ONTARIO

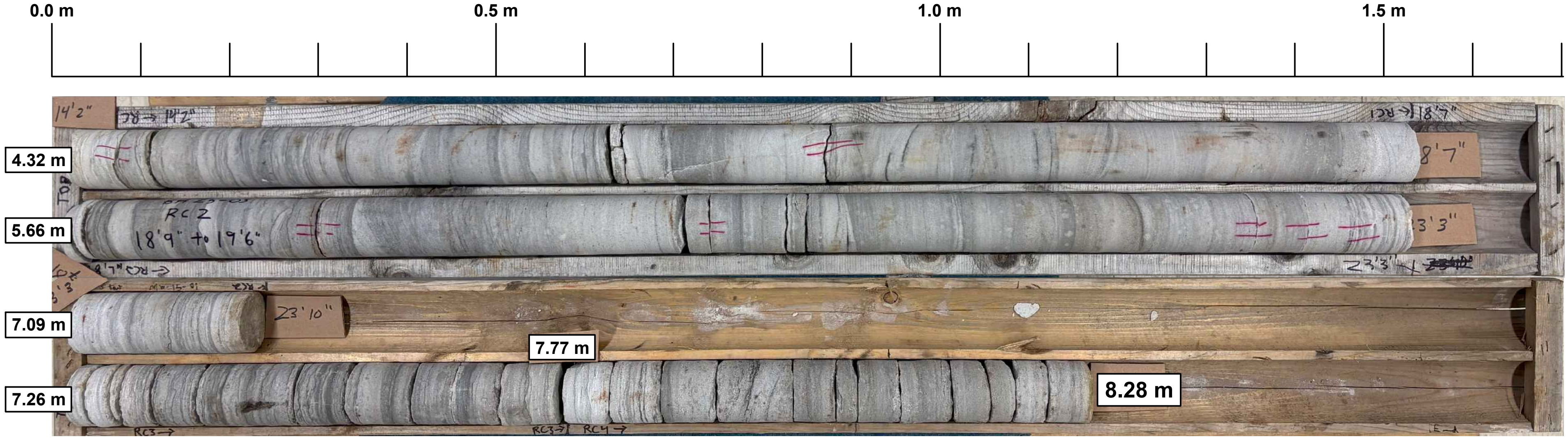
Drawing
ROCKCORE PHOTOGRAPH
BOREHOLE BH25-02

FIGURE B1

Project No.	Drwn By	Chkd By	Rev No.	Date
100041.032	SL	BC	0	JULY 2025

N:\PROJECTS\100000\100041.032\06_CIVIL DRAFTING\RC R0\100041.032_RC_R0_2025_07.DWG

BOREHOLE: BH25-03
BORING DATE: JULY 4, 2025
DEPTH: 4.32 TO 8.28 METRES BELOW GROUND SURFACE



N:\PROJECTS\100000\100041.032\06_CIVIL DRAFTING\RC R0\100041.032_RC_R0_2025_07.DWG

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Project	Drawing				
AECOM CANADA LTD. GEOTECHNICAL INVESTIGATION SHELL SERVICE STATION 106 EAGLSEON ROAD OTTAWA, ONTARIO	ROCKCORE PHOTOGRAPH BOREHOLE BH25-03				**FIGURE B2**
	Project No.	Drwn By	Chkd By	Rev No.	Date
	100041.032	SL	BC	0	JULY 2025

APPENDIX E

Hydraulic Conductivity Testing

Report to: AECOM
Project: 100041.032 (July 28, 2025)

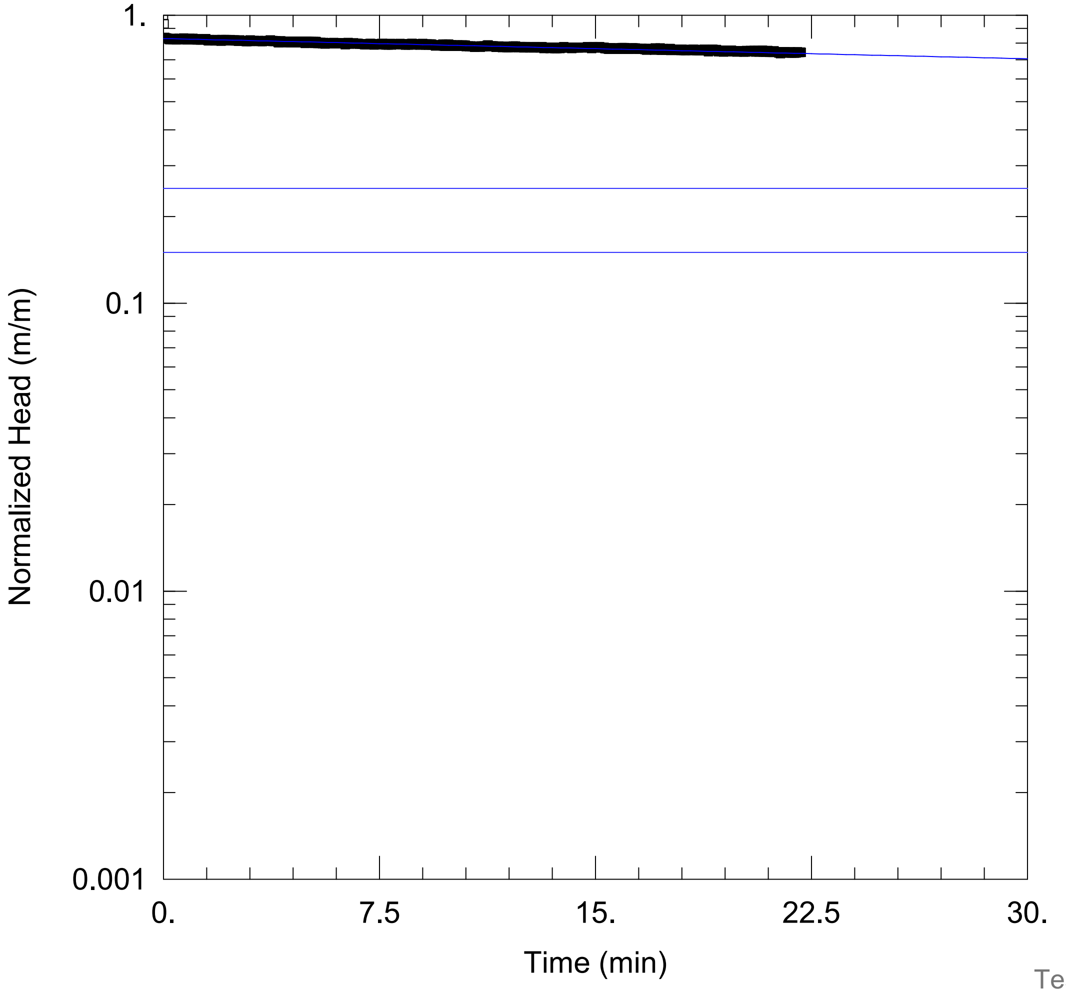
BH25-02: Falling Head Test

Prepared By:
GEMTEC

Prepared For:
AECOM Canada Ltd.

Project:
100041.032

Location:
106 Eagleson Road



Test Date: 2025-07-1

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev

K = 4.246E-7 m/sec   y0 = 0.3731 m

AQUIFER DATA

Saturated Thickness: 1.6
Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (BH25-02)

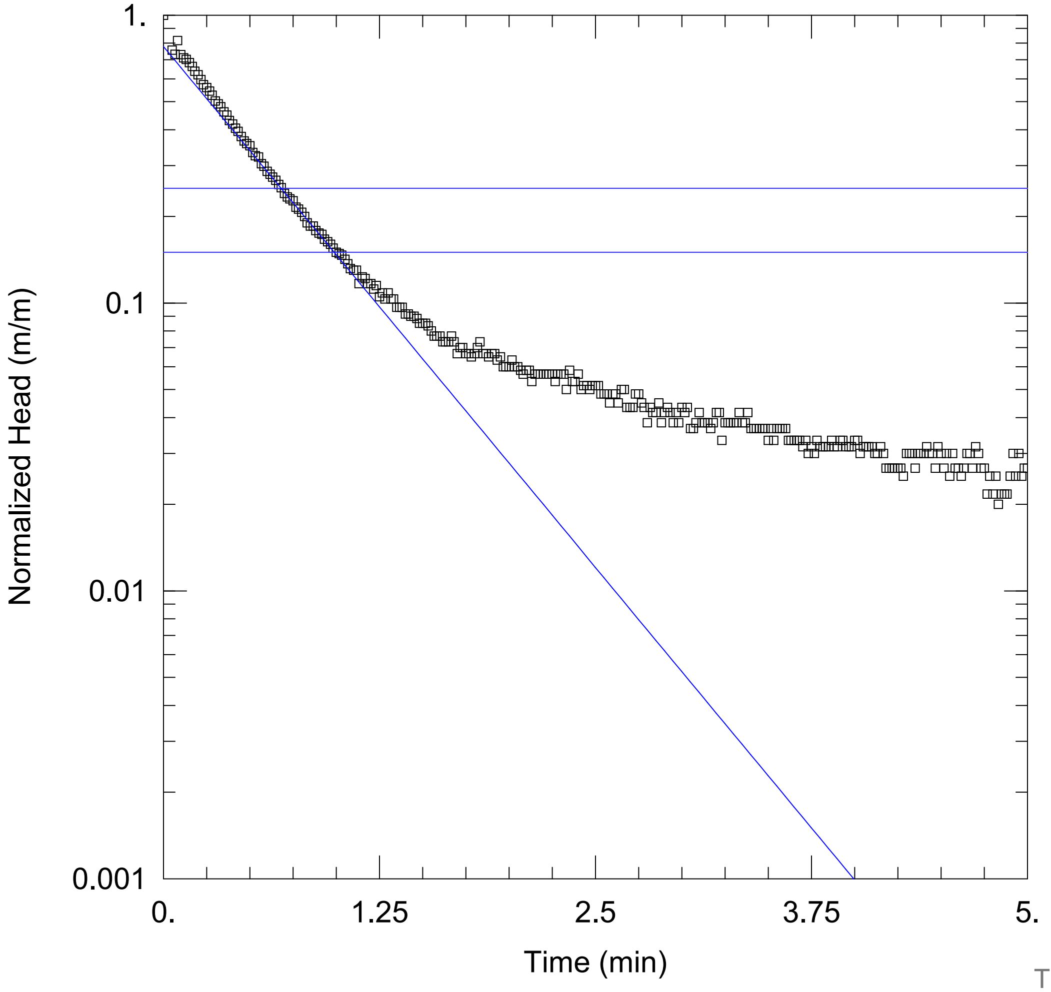
Initial Displacement: 0.45 m
Static Water Column Height: 1.6 m
Total Well Penetration Depth: 1.52 m
Screen Length: 1.52 m
Casing Radius: 0.0254 m
Well Radius: 0.105 m
Gravel Pack Porosity: 0.3

GEMTEC
Consulting Engineers and Scientists

Analysed by: JKA
Reviewed by: AP

BH25-03: Falling Head Test 1

Prepared By:	Prepared For:
GEMTEC	AECOM Canada Ltd.
Project:	Location:
100041.032	106 Eagleson Road



Test Date: 2025-07-14

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev

K = 8.92E-5 m/sec
y0 = 0.4665 m

AQUIFER DATA

Saturated Thickness: 5.42
Anisotropy Ratio Kz/Kr: 0.1

WELL DATA (BH25-03)

Initial Displacement: 0.6 m
Static Water Column Height: 5.42 m
Total Well Penetration Depth: 5.67 m
Screen Length: 3.05 m
Casing Radius: 0.0254 m
Well Radius: 0.105 m
Gravel Pack Porosity: 0.3

GEMTEC
Consulting Engineers and Scientists

Analysed by: DC
Reviewed by: AP

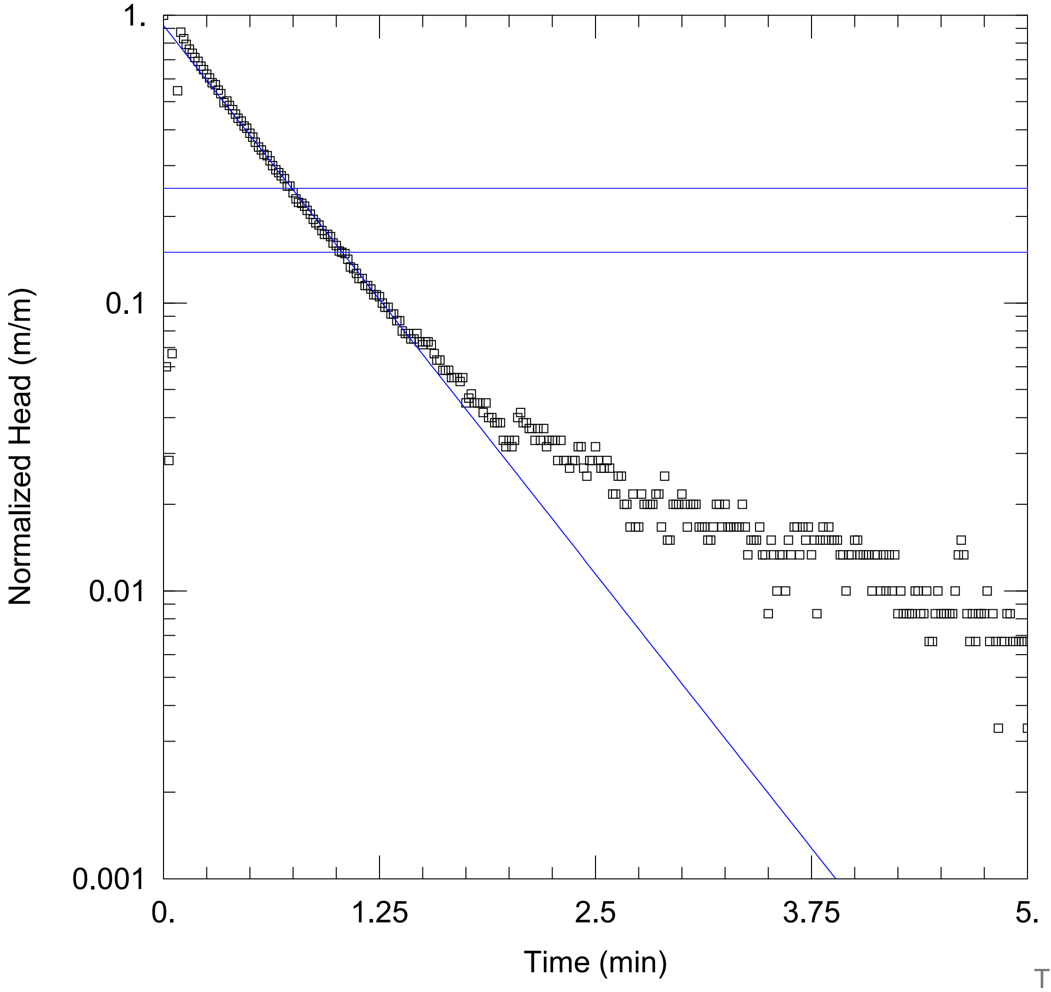
BH25-03: Rising Head Test 1

Prepared By:
GEMTEC

Prepared For:
AECOM Canada Ltd.

Project:
100041.032

Location:
106 Eagleson Road



Test Date: 2025-07-1

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev

K = 9.391E-5 m/sec
y0 = 0.5519 m

AQUIFER DATA

Saturated Thickness: 5.42
Anisotropy Ratio Kz/Kr: 0.1

WELL DATA (BH25-03)

Initial Displacement: 0.6 m
Static Water Column Height: 5.42 m
Total Well Penetration Depth: 5.67 m
Screen Length: 3.05 m
Casing Radius: 0.0254 m
Well Radius: 0.105 m
Gravel Pack Porosity: 0.3

GEMTEC
Consulting Engineers and Scientists

Analysed by: JKA
Reviewed by: AP

APPENDIX F

Chemical Analysis of Soil Samples Relating to Corrosion

Report to: AECOM
Project: 100041.032 (July 28, 2025)

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Certificate of Analysis

GEMTEC Consulting Engineers and Scientists Limited
32 Steacie Drive
Kanata, ON K2K 2A9
Attn: Matt Rainville

Report Date: 15-Jul-2025
Order Date: 9-Jul-2025

Client PO:
Project: 100041.032
Custody:

Order #: 2528449

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

Paracel ID	Client ID
2528449-01	BH25-01 SA5 5'6"-7'6"
2528449-02	BH25-02 SA4 3'-4'6"

Approved By: A. Tirca

Adriana Tirca, B.Eng (Chem)

Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	14-Jul-25	14-Jul-25
Conductivity	MOE E3138 - probe @25 °C, water ext	14-Jul-25	15-Jul-25
pH, soil	MOE E3137 - probe @25 °C, CaCl2 ext	15-Jul-25	15-Jul-25
Resistivity	EPA 120.1 - probe, water extraction	14-Jul-25	15-Jul-25
Solids, %	CWS Tier 1 - Gravimetric	11-Jul-25	14-Jul-25

Certificate of Analysis

Report Date: 15-Jul-2025

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

Order Date: 9-Jul-2025

Client PO:

Project Description: 100041.032

	Client ID:	BH25-01 SA5 5'6"-7'6"	BH25-02 SA4 3'-4'6"	-	-		
	Sample Date:	03-Jul-25 09:00	03-Jul-25 09:00	-	-	-	-
	Sample ID:	2528449-01	2528449-02	-	-		
	Matrix:	Soil	Soil	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	72.7	81.9	-	-	-	-
General Inorganics							
Conductivity	5 uS/cm	384	375	-	-	-	-
pH	0.05 pH Units	7.28	7.54	-	-	-	-
Resistivity	0.1 Ohm.m	26.1	26.7	-	-	-	-
Anions							
Chloride	10 ug/g	130	72	-	-	-	-
Sulphate	10 ug/g	69	19	-	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions								
Chloride	ND	10	ug/g					
Sulphate	ND	10	ug/g					
General Inorganics								
Conductivity	ND	5	uS/cm					
Resistivity	ND	0.1	Ohm.m					

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	70.2	10	ug/g	72.4			3.1	35	
Sulphate	18.2	10	ug/g	18.8			3.6	35	
General Inorganics									
Conductivity	630	5	uS/cm	627			0.5	5	
pH	7.48	0.05	pH Units	7.47			0.1	2.3	
Resistivity	15.9	0.1	Ohm.m	15.9			0.5	20	
Physical Characteristics									
% Solids	78.9	0.1	% by Wt.	79.3			0.5	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	165	10	ug/g	72.4	93.0	82-118			
Sulphate	114	10	ug/g	18.8	95.5	80-120			

Qualifier Notes:

Sample Data Revisions:

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

NC: Not Calculated

Soil results are reported on a dry weight basis unlesss otherwise noted.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Any use of these results implies your agreement that our total liabilty in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstances be liable to you in connection with this work.

experience • knowledge • integrity

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

expérience • connaissance • intégrité