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PG6260-LET.01

Strathburn Almonte Regional Inc.
1737 Woodward Drive
Ottawa, ON
K2C 0P9

Attention: **Mr. Evan Garfinkel**

Subject: **Geotechnical Investigation**
Proposed Off-Site Services
County Road No. 29, Strathburn Street and Malcolm Street
Almonte Ontario

Dear Sir,

Further to your request, Paterson Group (Paterson) completed a geotechnical investigation to assess the existing subsurface conditions throughout the areas of County Road No. 29, Strathburn Street and Malcolm Street in Almonte, Ontario. The investigation was undertaken in support of the future site servicing works for the proposed residential development that will be located at the northeastern corner of Strathburn Street and County Road No. 29.

The objectives of the assessment were to:

- ❐ Determine the existing pavement structure and subgrade conditions by means of boreholes undertaken throughout the roadway shoulder.

- ❐ Provide service installation and pavement reconstruction recommendations from a geotechnical perspective, based on the existing subsurface conditions associated with the subject site.

This report presents a summary of our findings and provides geotechnical recommendations pertaining to the existing subsurface conditions.

1.0 Field Observations

1.1 Field Program & Laboratory Testing

The field program for the investigation was conducted on December 1 and 2, 2022, and consisted of advancing 8 boreholes to a maximum depth of 5.1 m. The fieldwork was reviewed in the field by Paterson personnel under the direction of a senior engineer from the geotechnical division. The borehole procedure consisted of drilling to the required depths at the selected locations and sampling the overburden.

The boreholes were placed in a manner to provide general coverage of the subject roadways, taking into consideration existing site features and underground services. The locations of the test holes are shown on Drawing PG6260-4 - Test Hole Location Plan attached to the present report.

All samples were submitted for moisture content testing following completion of the field program. The results of the moisture content testing are presented on the Soil Profile and Test Data Sheets.

Two (2) soil samples (one from each service alignment) were submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 4.4.

1.2 Site Conditions

The subject site consists of existing paved roads and street with gravel shoulders. Multiple guardrails are located along County Road No. 29. Two water-crossings along County Road No. 29 and one water-crossing along Strathburn Street were observed below the road surface, which were facilitated via culverts. The ground surface across the subject site generally slopes from the southwest downward to the northeast and northeast from approximate geodetic elevations of 125 m to 112 m. The subject site is depicted on Drawing PG6260-4 - Test Hole Location Plan, attached to the present report.

1.3 Roadway Surface Conditions

Based on our observations, the existing asphalt pavement road and street surfaces throughout the subject site were generally observed to be in good to fair condition.

The surface conditions along Country Road 29 were noted to be in good condition, with mild longitudinal and transverse cracking. No signs of erosion were noted along the gravel shoulder of the roadway.

The alignment of the guardrails was observed to deviate slightly towards the watercourse on both sides of the road shoulder where County Road No. 29 crosses over the watercourse below.

The surface conditions along Strathburn Street and Malcolm Street were noted to be in fair condition, with moderate longitudinal and transverse cracking. Minor signs of erosion were noted along the gravel shoulder of Strathburn Street and Malcolm Street.

1.4 Subsurface Conditions

Pavement Structure and Overburden

Generally, the subsurface profile along County Road 29 consists of fill, ranging in thickness from approximately 0.8 to 4 m, underlain by a hard to very stiff, brown silty clay or glacial till. The fill mainly consists of crushed stone and gravel with sand, gravelly sand with silt, and silty sand with gravel. The boreholes adjacent to the crossing of the watercourses (boreholes BH-R02-22 and BH-R04-22) encountered fill to a greater depth and the fill consisted of silty clay, some to trace gravel.

The subsurface profile along Strathburn Street generally consists of a 0.4 to 1.5 m thickness of fill, composed of crushed stone with gravel and sand, over hard to very stiff, brown silty clay.

The subsurface profile along Malcolm Street generally consists of crushed stone with sand and gravel fill underlain by a deposit of hard, brown silty clay. An approximately 180 mm thick layer of silty sand was observed below the silty clay layer and was further underlain by glacial till. The glacial till layer was observed to consist of very dense silty sand to sandy silt with gravel, cobbles and boulders, some clay.

Practical refusal to augering was encountered at boreholes BH-R01-22, BH-R05-22 and BH-R08-22 at depths of 3.6 m, 3.9 m and 1.6 m respectively.

Bedrock

Based on geological mapping, the bedrock at the subject site consists of dolomite of the Oxford Formation and interbedded sandstone and dolomite of the March Formation with overburden drift thicknesses ranging between 0 and 3 m.

Groundwater

Based on the field observations and the recovered soil samples, the long-term groundwater table is expected to be located at greater depth than the test holes advanced throughout the subject site. However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

2.0 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed municipal service installation and subsequent road reconstruction. It is expected that the proposed municipal services will be founded on the undisturbed existing fill, hard to very stiff silty clay, or compact to dense glacial till. Due to the presence of the silty clay deposit, the sections of proposed roadway reconstruction will be subjected to grade raise restrictions.

It is also anticipated that two culvert crossing will be required along County Road No. 29 and one culvert crossing will be required along Strathburn Street.

Bedrock removal may be required dependent on the municipal servicing depths. The above and other considerations are discussed in the following paragraphs.

3.0 Geotechnical Recommendations

3.1 Site Grading and Preparation

Stripping Depth

It is expected all existing overburden will be removed up to the depth of the service subgrade depth. Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any services, paved areas and other settlement sensitive structures prior to placing fill to raise the grade.

It is anticipated that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed services and pavement structure. However, it is recommended that the existing fill layer be proof-compacted several times under dry conditions and above freezing temperatures. Any poor performing areas noted during the proof-compaction operation should be removed and replaced with an approved fill.

Bedrock Removal

In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Line drilling and controlled blasting could also be used where a large volume of bedrock needs to be removed. However, prior to blasting, the potential blast damage to the existing structures must be considered.

A pre-blast or pre-construction survey of the existing buildings and underground structures should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

As a general guideline, maximum peak particle velocities of 25 to 50 mm/s (measured at the structure) should not be exceeded during the blasting program to reduce the risks of damages to the existing structures. Blasting close to freshly placed concrete should also be closely controlled.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

Construction operations are the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations.

As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed services.

Fill Placement

Fill used to raise the subgrade for the pavement structure should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type I or II. These materials should be tested and approved prior to delivery to the site. The existing silty sand, silty sand and gravel, moist (not wet) and free of cobbles, boulders, and organic matter, may also be used. However, placement of site-generated soil fill material should be reviewed prior to use and at the time of placement by Paterson personnel.

The fill should be placed in lifts no greater than 300 mm in thickness and compacted using suitable compaction equipment for the lift thickness. Fill placed below the pavement subgrade level, beneath the paved areas, should be compacted to at least 95% of its standard Proctor maximum dry density (SPMDD). Site-generated soil fill is recommended to be placed using a suitably sized vibratory sheepsfoot roller and reviewed at the time placement by Paterson personnel.

3.2 Foundation Design

Bearing Resistance Values

Catch basins and maintenance chambers may be founded on engineered fill placed on silty clay, glacial till, or bedrock and can be designed using the allowable bearing presented in Table 1 below. Engineered fill under catch basins and maintenance chambers should consist of OPSS Granular A material placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of its SPMDD.

Table 1 – Bearing Resistance Values

Bearing Surface	Serviceability Limit States (SLS) kPa	Ultimate Limit State (ULS) kPa
Silty Clay	150	225
Glacial Till	200	300
Bedrock	-	750

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Catch basis, maintenance chambers and associated structures placed on engineered fill overlying an undisturbed soil bearing surface and designed using the bearing resistance values herein will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under proposed services is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to clay, sand, and engineered fill bearing media when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through the in-situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. Weathered bedrock will require a lateral support zone of 1H:1V (or flatter).

Settlement and Permissible Grade Raise

Since the proposed works will be undertaken to install new services throughout existing residential areas, it is not expected the existing road surface grade will be raised as part of the proposed works. However, based on the results of our investigation, a permissible grade raise recommendation of **3 m** above the current road surface elevation may be considered throughout the subject site.

3.3 Excavation Side Slopes

The side slopes of excavations in the overburden and weathered bedrock should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Unsupported Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. Excavations below the groundwater level should be cut back at a maximum slope of 2H:1V. Site servicing plans should be reviewed by Paterson prior to tendering to advise on precautions for excavations that would be anticipated to extend below the anticipated groundwater table.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1.0 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing.

Temporary Shoring

Where space restrictions exist, temporary shoring may be required. The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor.

It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 2 – Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At Rest Earth Pressure Coefficient (K_0)	0.5
Unit Weight (y), kN/m^3	21
Submerged Unit Weight (y), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Excavation Base Stability

The base of supported excavations can fail by three general modes:

- ☐ Shear failure within the ground caused by inadequate resistance to loads imposed by grade differences inside and outside of the excavation,
- ☐ Piping from water seepage through granular soils, and
- ☐ Heave of layered soils due to water pressures confined by intervening low permeability soils.

Shear failure of excavation bases are typically rare in granular soils if adequate lateral support is provided. Inadequate dewatering can cause instability in excavations made through granular or layered soils. The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems.

The factor of safety with respect to base heave, FS_b is:

$$\mathrm{FS}_b = \mathrm{N}_b s_u / \sigma_z$$

where,

N_b - Stability factor dependent upon geometry of the excavation and give in Figure 1
S_u - Undrained shear strength of the soil below the base level
σ_z - Total overburden and surcharge pressure at the bottom of the excavation

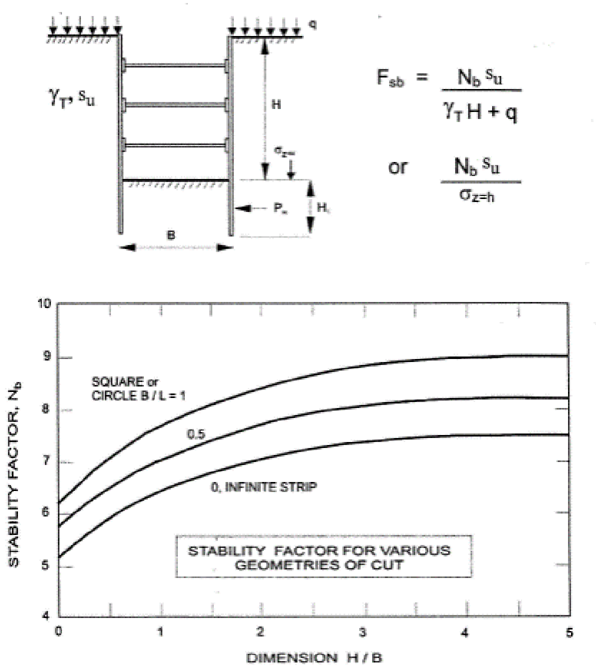


Figure 1 – Stability Factor for Various Geometries of Cut

In the case of soft to firm clays, a factor of safety of 2 is recommended for the base stability.

Separation Between Existing Services

Given the possible existing service along County Road No. 29, north of Wylie Street, it is recommended that the proposed excavation be carried out in a manner as to locate the sidewall of the excavation as far as possible from the existing services. A minimum clearance of 1.5 m is recommended between the centerline of the existing services and sidewalls of the proposed excavation. It is recommended that the clearance be increased as much as possible while providing the required clearance for the proposed service installation. The backfill material above the proposed services and within the lateral support zone of existing services should be placed in maximum 225 mm thick lifts compacted to a minimum of 99% of the material’s standard Proctor maximum dry density (SPMDD).

3.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with OPSS standards and specifications.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located on a bedrock bearing surface, the thickness of the bedding material should be increased to a minimum of 300 mm. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PVC pipes). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 99% of the material's standard Proctor maximum dry density (SPMDD).

In areas where the service subgrade transitions from soil to bedrock, it is recommended that the founding medium be inspected in the field to determine how steeply the bedrock surface, where encountered, drops off. A transition treatment should be provided where the bedrock slopes at more than 3H:1V. At these locations, the bedrock should be excavated, and extra bedding placed to provide a 3H:1V transition from the bedrock subgrade toward the soil subgrade. This treatment will reduce the propensity for bending stresses to occur in the service.

Engineered fill under service pipes, where required, should consist of OPSS Granular A (crushed stone) or Granular B Type II placed in maximum 300 mm thick layers and compacted to a minimum of 98% of the material's SPMDD.

It should generally be possible to re-use the pavement granulars and fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. All fill material placed above the cover layer and to raise the road subgrade should be placed in maximum 300 mm thick loose lifts and compacted using a suitably sized vibratory smooth-drum (crushed stone fill) or sheepsfoot (soil fill) roller. The subgrade should be compacted to a minimum of 98% of the materials SPMDD. The in-situ SPMDD of a soil fill will be difficult to measure adequately using a nuclear field density gauge. As such, field personnel experienced in visually reviewing the placement of soil fill should review the placement of these materials accordingly at the time of construction.

Well fractured bedrock should be acceptable as backfill above the cover material provided that the rock fill is placed above the pipe cover layer and in maximum 300 mm thick loose lifts and that all stones are 300 mm or smaller in their longest dimension.

The trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

Approach Transitions

Approach transitions should be provided when the trench backfill material is located within the frost zone (1.8 m below the final grade) and backfill material is not compatible with soil exposed on the excavation side walls (i.e. do not have similar frost heaving behaviour).

The excavation side walls are recommended to be profiled at a minimum of 3H:1V from a depth of 1.8 m to the underside of the pavement granules, if the excavation is transverse to the traffic direction. Slopes should be excavated to 1.5H:1V if the trench is longitudinal to the traffic lanes.

3.5 Pavement Structure Design

To accommodate the proposed roadway reconstruction the recommendations indicated below should be followed:

- A 300 mm wide section of the existing asphalt roadway should be saw cut from the edge of the service trench.
- It is recommended to mill a 300 mm wide and 40 mm deep section of the existing asphalt, adjacent to the service trench.
- Construct the subbase and base within the service trench, and then place a new asphalt surface across the service trench to the milled section.
- The proposed pavement structure subbase materials should be tapered no greater than 3H:1V to meet the existing subbase materials.
- The new pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material’s SPMDD.
- If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular A or Granular B Type II material.
- Clean existing granular road subbase materials can be reused upon assessment by Paterson at the time of excavation (construction) as to its suitability.
- The minimum pavement structure for the subject portion of County Road No. 29 is presented in Table 3. An Ontario Traffic Category D was used for design of the pavement structure. Performance Graded (PG) 64-34 asphalt cement should be used for this roadway.
- The minimum pavement structure for the subject portion of Strathburn Street and Malcolm Street is presented in Table 4. An Ontario Traffic Category B was used for design of the pavement structure. Performance Graded (PG) 58-34 asphalt cement should be used for this project.
- ESAL values are based on Paterson’s interpretation of the available data within the document titled “MUNICIPALITY OF MISSISSIPPI MILLS Comprehensive Transportation Master Plan” prepared by Dillon Consulting in March 2016.
- Asphalt should be compacted to a minimum average density of 93% and no more than 98%.
- A tack coat should be provided between the asphalt lifts to create an adequate bond.

Table 3 - Recommended Pavement Structure – County Road No. 29

Thickness (mm)	Material Description
40	**Wear Course** – Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** – Superpave 19.0 Asphaltic Concrete
50	**Lower Binder Course** – Superpave 19.0 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
600	**SUBBASE** – OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ silty clay, glacial till, bedrock or engineered fill placed over in situ soil.	

Table 4 - Recommended Pavement Structure – Strathburn Street and Malcolm Street

Thickness (mm)	Material Description
40	**Wear Course** – Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** – Superpave 19.0 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
400	**SUBBASE** – OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ silty clay, glacial till, bedrock or engineered fill placed over in situ soil.	

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Consideration should be given to installing subdrains at each catch basin during the pavement construction. These drains should be at least 3 m long and extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines. The subdrains will help drain the pavement structure, especially in early Spring when the subgrade is saturated and weaker and, therefore, more susceptible to permanent deformation.

Precaution must also be taken for when the subgrade consists of bedrock, to ensure that the upper 300 mm of the bedrock surface is shattered to permit drainage. Also, in the soil/bedrock transitions the lowest transition point should be drainage satisfactorily.

Transitions should be provided when the subgrade changes from being not frost susceptible to frost susceptible, in particular but not limited to bedrock to soil or soil to bedrock. The transitions should be per Ontario Provincial Standard Drawings (OPSD) 205.010, 205.020, 205.030, 205.040, and 205.050 using a transition treatment depth of 1.8 m.

4.0 Construction Considerations

4.1 Culvert and Road Crossing

Based on available site plans provided by Novatech Engineers, Planners & Landscape Architects, it is understood that the proposed services will cross two culverts along County Road No. 29 and one culvert along Strathburn Street.

The following subsections discuss design and construction precautions in relation to the possible installation of box or pipe culverts underlain by the proposed services.

Bearing Resistance Values

It is anticipated that box or pipe culverts will be founded on the surround material consisting of compacted OPSS Granular A for the proposed underlying municipal services which in turn will be placed on a native silty clay, glacial till or bedrock bearing surface.

Culvert structures placed on engineered fill overlying an undisturbed soil bearing surface and designed using the bearing resistance values presented in Table 1 Section 5.3 will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Construction Water Control - Cofferdam

Should culverts be installed, or services installed below the watercourse surface, it is recommended that cofferdams be installed upstream and downstream of watercourses to prevent streamflow into the proposed excavation for the installation of the culverts and services in order to promote worker safety during the construction program.

It is important to emphasize that streamflow seepage and groundwater infiltration must be diverted away from the excavation and downstream from the area of the proposed work. If the watercourse will be diverted from the work area, consideration should be given to installing temporary culverts, channels, or pumping systems able to sustain the temporary flows of the watercourse. Systems should be put in place prior to full interruption of the creek flow.

It is recommended that a watertight cofferdam such as sheet piling or sandbags (temporary cofferdam system) such as Portadam be considered if the stream will temporarily cut-off for the duration of the works. Due to the proposed depth of excavation, it is recommended that a sheet pile system designed under full hydrostatic pressure and using the design parameters in Section 3.4 above be considered.

Cofferdams should be designed by a specialized contractor and their engineer with experience with cofferdams and submerged temporary shoring systems and as described herein.

Backfill and Frost Treatment

Reference should be made to OPSD-803.01 regarding standard frost treatments for backfill and cover of concrete culverts and OPSD-803.030 and OPSD-803.031 regarding the frost treatment for pipe culverts. The frost taper should consist of OPSS Granular B Type II crushed stone, placed a maximum 300 mm thick loose lifts and compacted to a minimum of 99% of the material’s Standard Proctor Maximum Dry Density. Further, it is recommended that Paterson complete compaction testing on the granular material.

Paterson should review the depth of services crossing below proposed culvert structures at the design phase to assess the suitability of potential frost cover and the requirement for the use of rigid insulation for protect services and soil subgrade form frost action.

Groundwater

The culvert, services and their construction may be below the long-term groundwater level. As such, groundwater infiltration may be present during the construction program through the excavation side walls within the fill layers observed in boreholes BH-R02-22, BH-R04-22 and BH-R07-22. Low to moderate groundwater infiltration rates should be expected and should be controlled using open sumps at the bottom of the excavation. It is recommended to divert all water infiltration away from the working area at the time of construction to prevent disturbance to the bearing surfaces.

Dewatering

Prior to carrying out the work throughout the area of the pipe crossings, water influx for works taking place throughout the existing creek alignment should be controlled so that the servicing operations can be conducted in dry conditions Based on the existing test hole coverage, the pipe crossings will be carried out within a deposit of stiff to very stiff, brown silty clay. This clay deposit is considered to be of very low permeability, such that it is anticipated that pumping from open excavations will be sufficient to control the minimal groundwater influx throughout the work area.

However, due to the overlying permeable layer of fill, the contractor should be prepared for potentially initially moderate influx due to excavations crossing below the permeable fill layer. The contractor should be prepared to direct surface water away from subgrade areas during the construction process. The influx from precipitation should also be considered by the contractor during the excavation.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

Culvert Waterproofing

Should concrete culverts be considered, it is expected that a portion of the precast concrete box culvert will be submerged within the watercourse. As such, it is recommended that the exterior footing and foundation walls of concrete box culverts be waterproofed to prevent long-term deterioration of the concrete in the form of streamflow erosion. The waterproofing membrane should consist of the Colphene Torch’N Stick or equivalent other approved by Paterson.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations.
The area between the excavation side walls and concrete box culvert foundation walls should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular A or Granular B Type II. The granular material should be placed in maximum 300 mm loose lifts and compacted to 98% of the material’s SPMDD.

4.2 Bearing Medium and Bedding for Thrust Blocks

The details of standard thrust blocks should be as per the most recent applicable standards and specifications. The thrust block details require specified minimum soil bearing capacities. However, from a geotechnical perspective, the bearing capacity, especially in cohesionless soils, is dependent upon the direction of application of the load.

Thrust blocks resisting lateral loads should be sized based on allowable passive earth pressure at the pipe depth. Those resisting (downward) vertical loads should be sized based on the allowable vertical soil bearing capacity. The lateral and vertical bearing capacities of the bearing media expected at the proposed pump station location are presented in Table 5 below. These values should be confirmed by the geotechnical consultant during the design phase of the project, once the locations of the thrust blocks are determined.

Table 5 - Summary of Allowable Bearing Pressures for Thrust Block Sizing

Bearing Surface	**Allowable Bearing Pressure, kPa**	
	Lateral	**Vertical**
Hard to very stiff, undisturbed brown silty clay	100	150
Surface-sounded sandstone bedrock	500	500
Engineered fill	100	150

The allowable lateral bearing capacities presented are based on a minimum soil cover of 2 m. The allowable vertical bearing pressures presented are provided for the thrust block concrete which is installed on an undisturbed soil bearing surface. An undisturbed soil bearing surface is free of all topsoil and deleterious materials, such as loose, frozen or disturbed soil and in dry condition prior to the placement of concrete. The total ultimate friction force exerted on the backfilled pipe in response to longitudinal forces is a function of the trench depth and diameter of the pipe. The following formula is provided.

$$f_{ult} = \pi D H \gamma \tan \delta$$

where:

f_{ult}	Ultimate friction force (kN)	
D	Outer diameter of pipe (m)	
H	Height of backfill from the centre of pipe (m)	
γ	Unit weight of cover material: assume 20 kN/m^3 for engineered fill above groundwater level and 12 kN/m^3 below groundwater level	
$\tan \delta$	Friction factor:	0.35 for iron/steel pipe 0.40 for concrete pipe 0.35 for plastic pipe

The bearing medium under the thrust blocks is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the engineered fill and/or in-situ bearing medium soils when a plane extending down and out from the bottom edge of the thrust block at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

4.3 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter.

The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

4.4 Corrosion Potential and Sulphate

The results of the analytical testing of two (2) soil samples show that the sulphate content is less than 0.1%. This result is indicative that Type 10 (GU) Portland cement (general use cement) would be appropriate. The results of the chloride content and pH indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site while the resistivity tests yielded results indicative of a non-aggressive to slightly corrosive environment.

4.5 Field Inspections During Construction

It is recommended that Paterson personnel complete periodic inspections during construction. The inspections would include, at minimum, review of the subgrade material, compaction testing, sampling and testing of the granular fill and asphaltic layers and drainage inspections, where required.

5.0 Recommendations

It is recommended that Paterson review all preliminary and detailed site servicing and grading plans, from a geotechnical perspective. Additional recommendations and consideration may be provided by Paterson at those stages for further incorporation into those designs and prior to tendering the proposed site works.

It is a requirement for the design data provided herein to be applicable that a suitable material testing and observation program, including the following aspects, be performed by Paterson.

- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials used.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.
- Review of excavated material from an environmental perspective for off-site disposal recommendations.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon demand, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

Nicolas Seguin

Nicolas Seguin, EIT

LICENSED PROFESSIONAL ENGINEER
Jan 25, 2023
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Scott S. Dennis, P.Eng.

Attachments

☐ Soil Profile and Test Data Sheets
☐ Symbols and Terms
☐ Analytical Testing Results
☐ Figure 1 – Key Plan
☐ Drawing PG6260-1 - Test Hole Location Plan

Report Distribution

☐ Strathburn Almonte Regional Inc. (e-mail copy)
☐ Paterson Group (1 copy)

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 1, 2022

FILE NO. **PG6260**

HOLE NO. **BH-R01-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	121.73
FILL: Gravelly sand with silt — 0.18							
FILL: Brown silty sand with gravel and crushed stone		AU	1				
		SS	2	50	27	1	120.73
— 1.45							
FILL: Dark brown silty sand, some gravel and topsoil, trace organics		SS	3	75	8	2	119.73
— 2.21							
GLACIAL TILL: Compact to dense, brown silty sand, some gravel, cobbles, boulders and rock fragments, trace clay		SS	4	25	26		
		SS	5	31	50+	3	118.73
— 3.66							

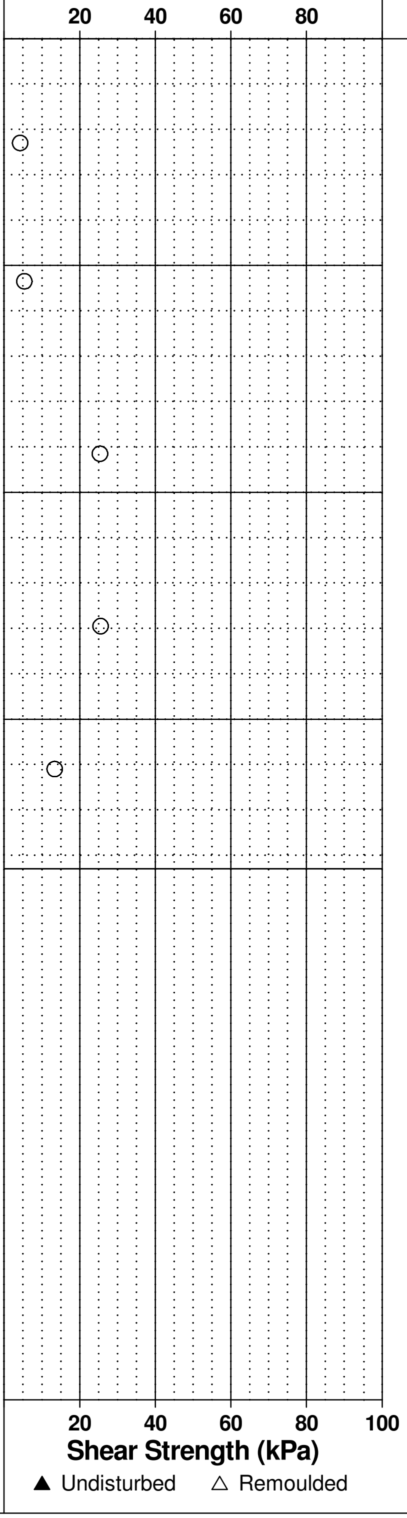
End of Borehole

Practical refusal to augering at 3.66m depth.

(BH dry upon completion)

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**



Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

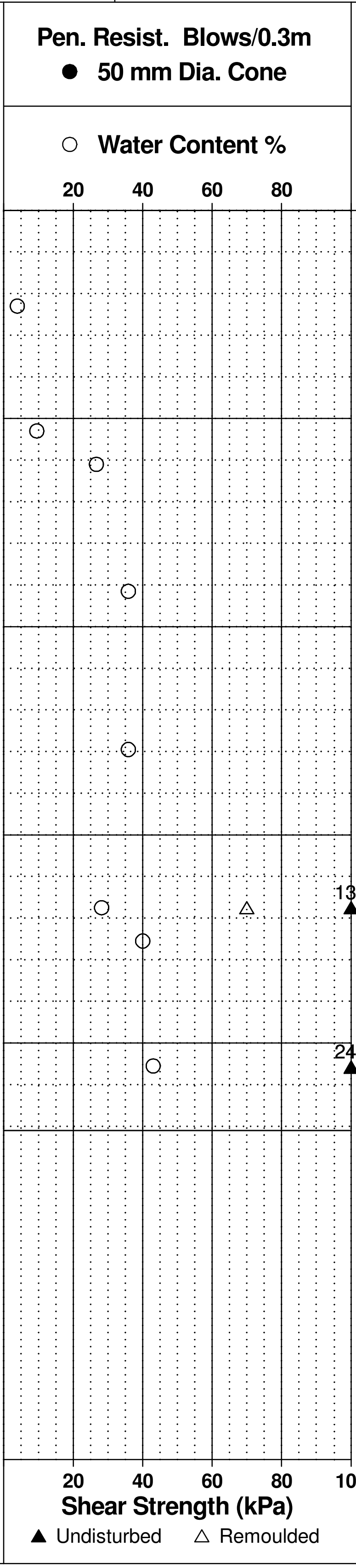
DATE December 1, 2022

FILE NO. PG6260

HOLE NO. BH-R02-22

SOIL DESCRIPTION	Strata bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Gravelly sand with silt	0.20				
FILL: Brown silty sand with gravel	0.84	AU	1		
FILL: Brown silty clay, some to trace gravel	3.07	SS	2	42	11
		SS	3	54	7
		SS	4	58	5
Very stiff to hard, brown **SILTY CLAY**	4.42	SS	5	71	P
		SS	6	100	P
End of Borehole (BH dry upon completion)					

DEPTH (m)	ELEV. (m)
0	120.50
1	119.50
2	118.50
3	117.50
4	116.50



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

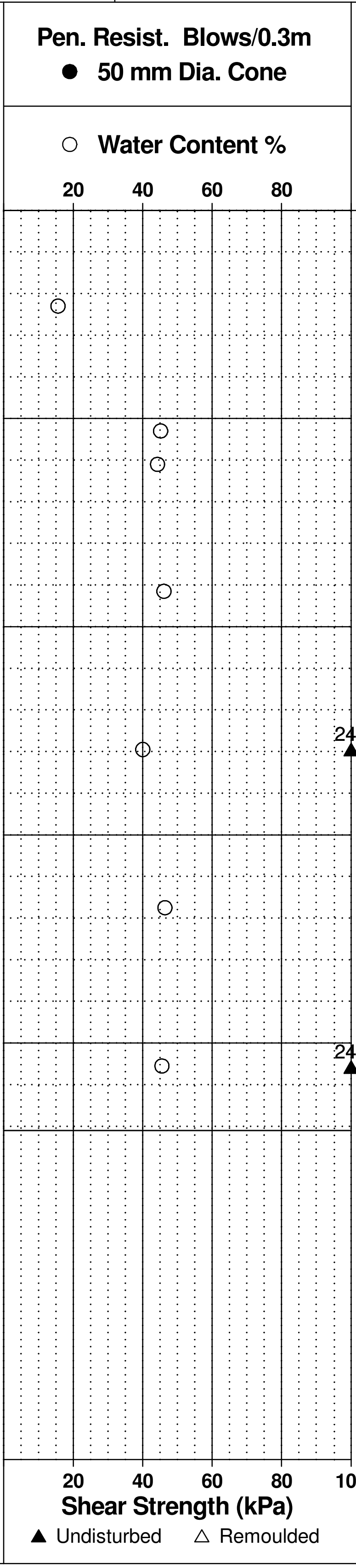
DATE December 1, 2022

FILE NO. PG6260

HOLE NO. BH-R03-22

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
FILL: Crushed stone and gravel with sand 0.18						0	120.68
FILL: Brown silty sand with gravel, some clay 0.60		AU	1				
Very stiff, brown **SILTY CLAY,** trace organics 0.97							
Hard, brown **SILTY CLAY** - intermittent sand seams from 1.5 to 2.9m depth		SS	2	75	15	1	119.68
		SS	3	100	9		
						2	118.68
		SS	4	100	P		
						3	117.68
		SS	5	100	9		
						4	116.68
4.42		SS	6	100	P		
End of Borehole (BH dry upon completion)							

Piezometer Construction



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE December 1, 2022

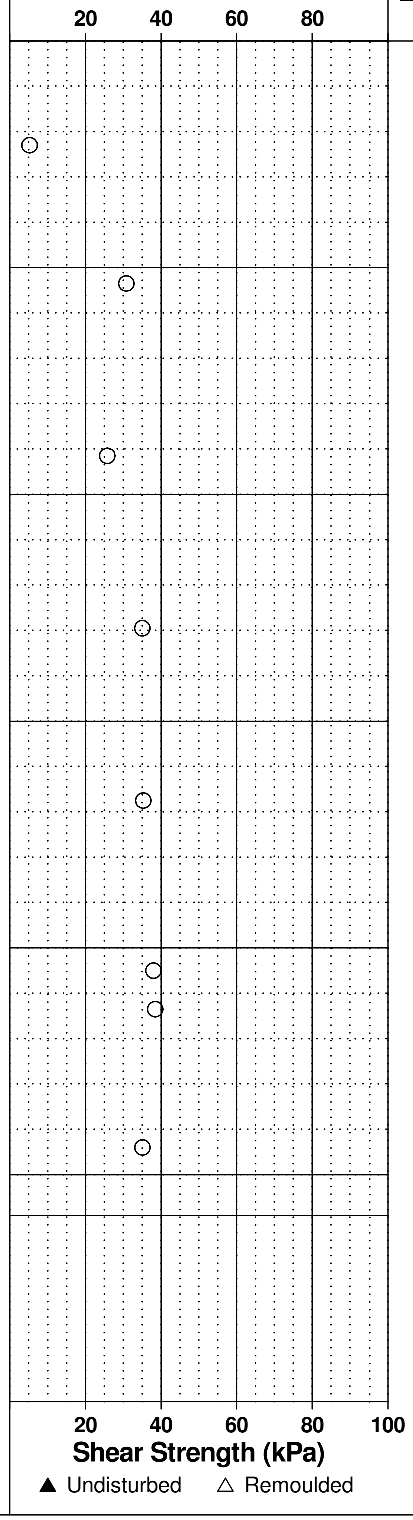
FILE NO. PG6260

HOLE NO. BH-R04-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	122.48
FILL: Crushed stone and gravel with sand 0.15							
FILL: Brown silty sand with gravel 0.69		AU	1				
FILL: Brown silty clay, some gravel, trace debris		SS	2	17	6	1	121.48
		SS	3	12	4	2	120.48
		SS	4	67	6		
3.96		SS	5	62	5	3	119.48
Hard, brown **SILTY CLAY,** trace seashells		SS	6	96	10	4	118.48
5.18		SS	7	88	13	5	117.48
End of Borehole (BH dry upon completion)							

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

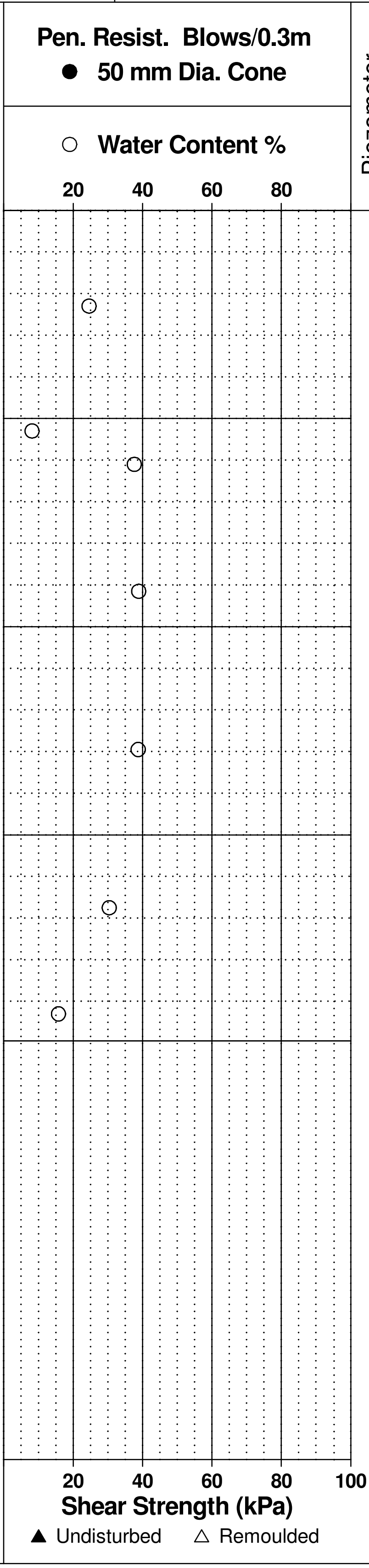
BORINGS BY CME-55 Low Clearance Drill

DATE December 1, 2022

FILE NO. PG6260

HOLE NO. BH-R05-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	124.88
FILL: Crushed stone and gravel with sand 0.15							
FILL: Brown silty clay with gravel, some sand, trace topsoil 0.84		AU	1				
Hard, brown **SILTY CLAY**		SS	2	29	12	1	123.88
		SS	3	96	10	2	122.88
- trace gravel from 2.9 to 3.7m depth		SS	4	100	11	3	121.88
		SS	5	100	13		
3.99		SS	6	100	50+		
End of Borehole Practical refusal to augering at 3.99m depth. (BH dry upon completion)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

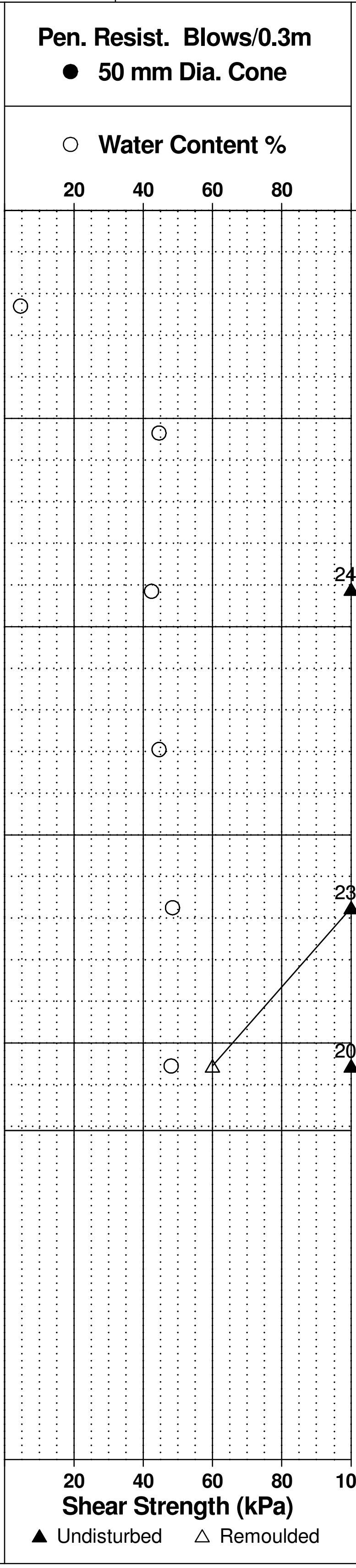
BORINGS BY CME-55 Low Clearance Drill

DATE December 2, 2022

FILE NO. **PG6260**

HOLE NO. **BH-R06-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	114.83	
FILL: Crushed stone with sand and gravel 0.33		AU	1					
		SS	2	67	9	1	113.83	
		SS	3	83	P			
						2	112.83	
Hard, brown **SILTY CLAY**								
		SS	4	88	10			
						3	111.83	
		SS	5	88	P			
						4	110.83	
4.42		SS	6	92	P			
End of Borehole (BH dry upon completion)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

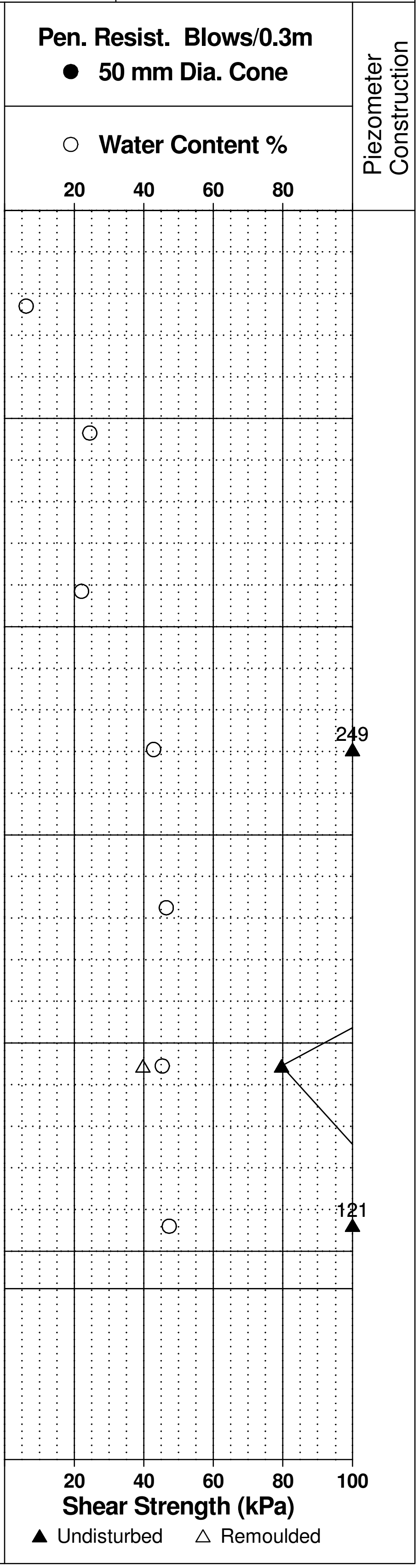
DATE December 2, 2022

FILE NO. PG6260

HOLE NO. BH-R07-22

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.05				
FILL: Crushed stone with gravel and sand 0.69	AU	1		
FILL: Brown silty clay, trace sand and gravel 1.45	SS	2	21	21
Hard to very stiff, brown **SILTY CLAY**	SS	3	58	10
	SS	4		P
	SS	5	83	7
	SS	6	42	P
5.18	SS	7	83	P
End of Borehole (BH dry upon completion)				

DEPTH (m)	ELEV. (m)
0	112.10
1	111.10
2	110.10
3	109.10
4	108.10
5	107.10



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

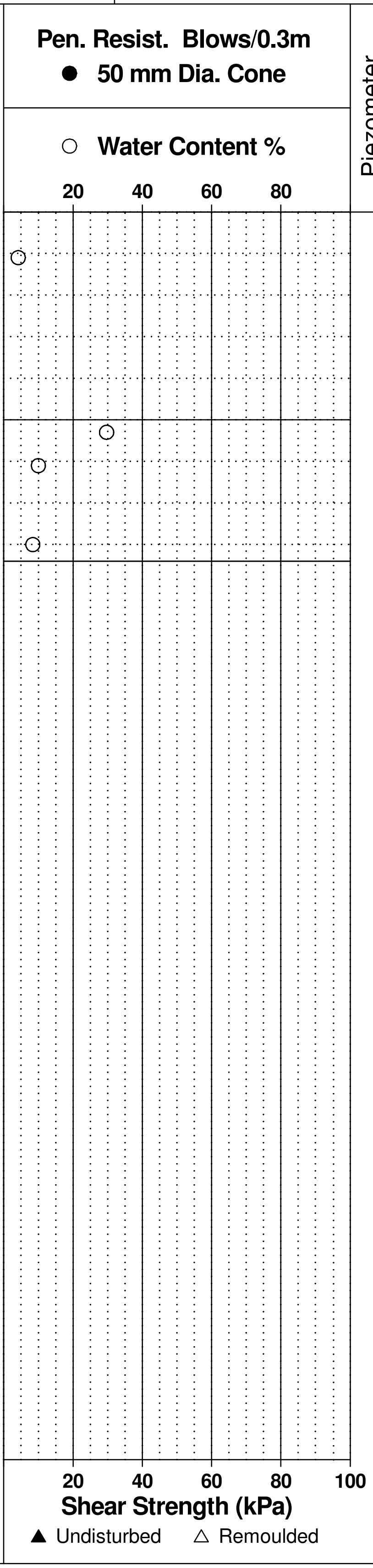
BORINGS BY CME-55 Low Clearance Drill

DATE December 2, 2022

FILE NO. **PG6260**

HOLE NO. **BH-R08-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	112.68
FILL: Crushed stone with sand and gravel 0.43		AU	1				
Hard, brown **SILTY CLAY** 1.27		SS	2	62	11	1	111.68
Compact, brown **SILTY SAND** with gravel 1.45							
GLACIAL TILL: Very dense, light brown silty sand to sandy silt with gravel, cobbles and boulders, some clay 1.68		SS	3	100	50+		
End fo Borehole Practical refusal to augering at 1.68m depth. (BH dry upon completion)							



Piezometer Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

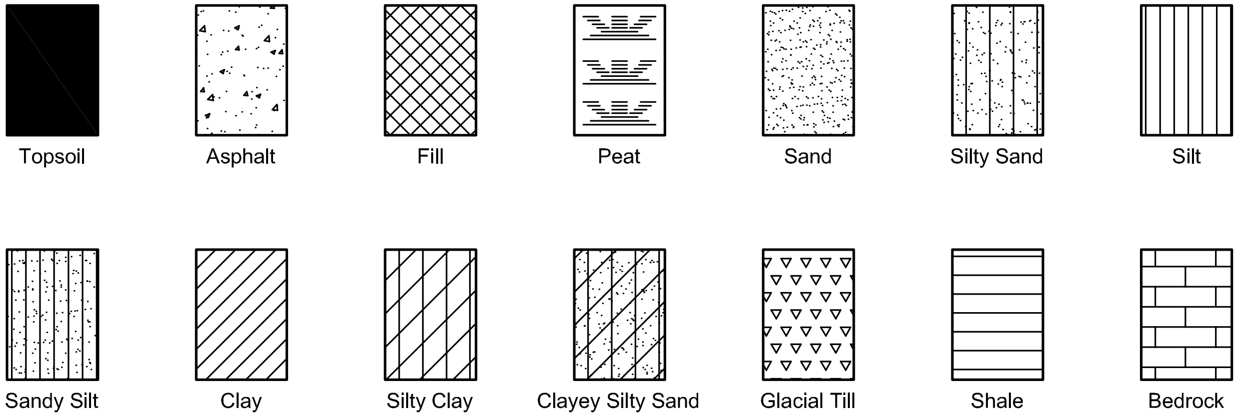
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

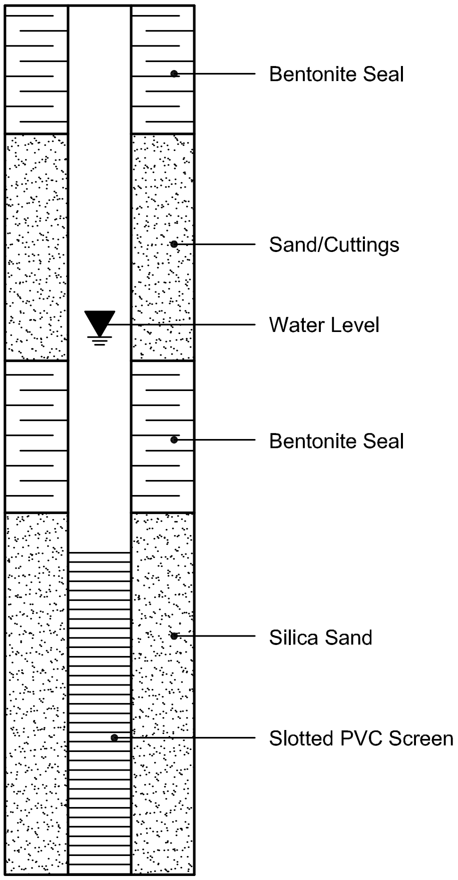
SYMBOLS AND TERMS (continued)

STRATA PLOT

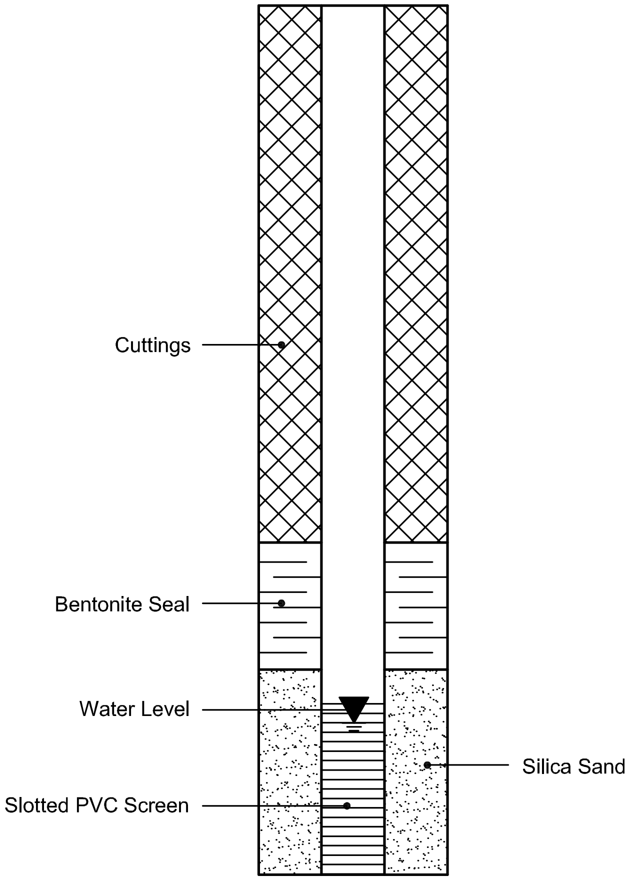


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 09-Dec-2022

Client: Paterson Group Consulting Engineers

Order Date: 5-Dec-2022

Client PO: 56375

Project Description: PG6260

	Client ID:	BH-R03-22-SS4	BH-R07-22-SS4	-	-		
	Sample Date:	01-Dec-22 09:00	02-Dec-22 09:00	-	-	-	-
	Sample ID:	2250105-01	2250105-02	-	-		
	Matrix:	Soil	Soil	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	70.9	70.0	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.54	7.59	-	-	-	-
Resistivity	0.1 Ohm.m	14.5	103	-	-	-	-
Anions							
Chloride	5 ug/g	1290	356	-	-	-	-
Sulphate	5 ug/g	82	75	-	-	-	-

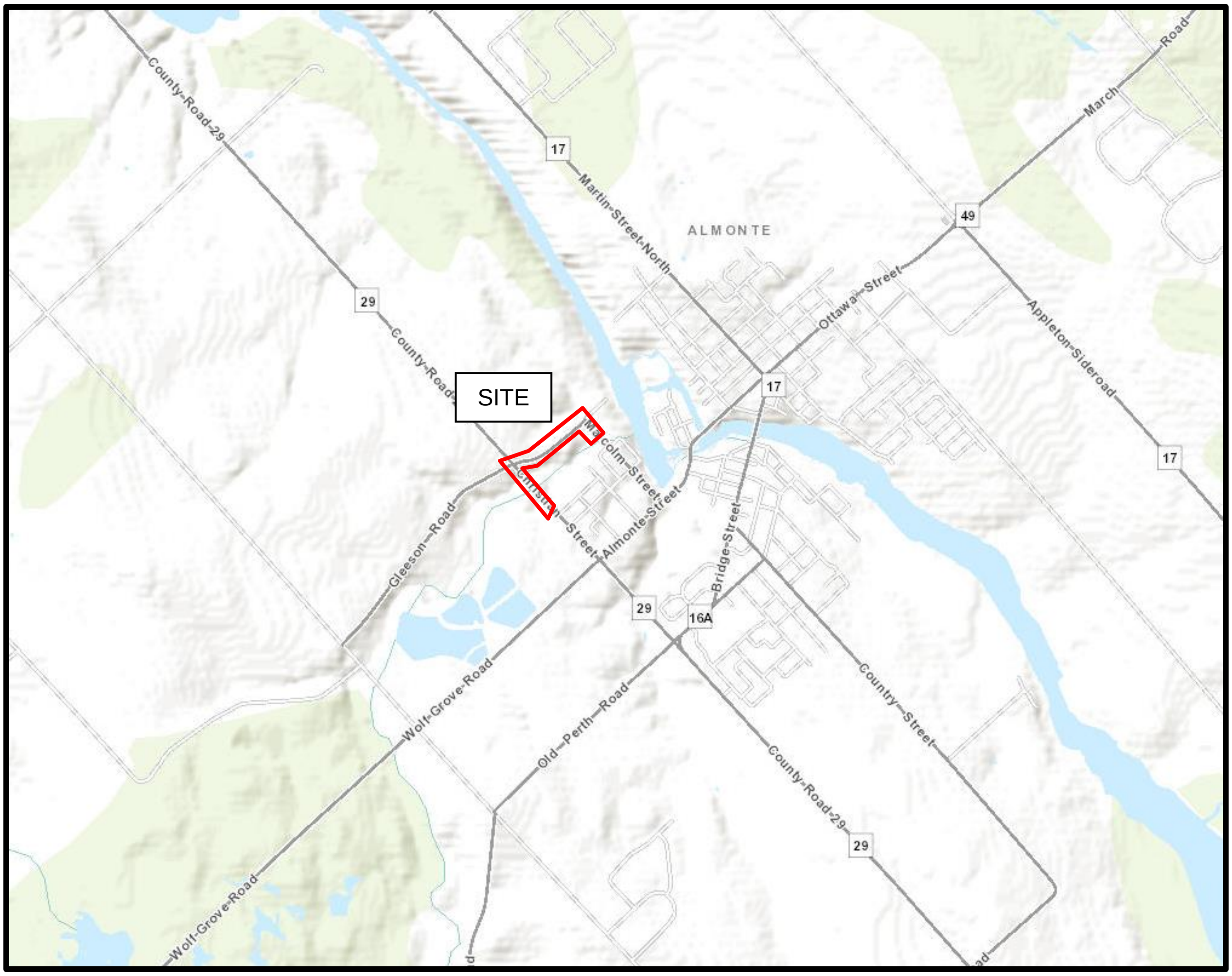
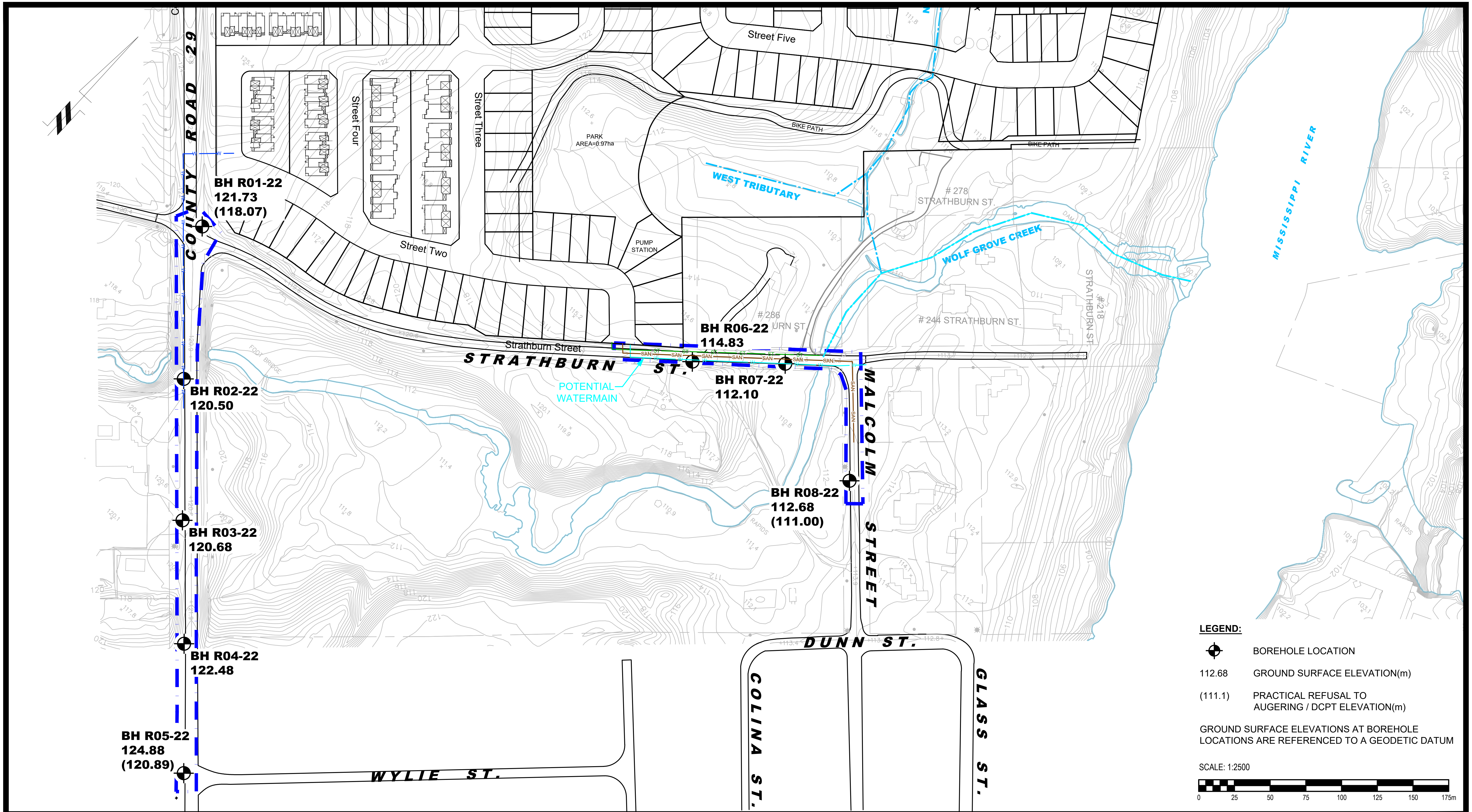


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION
112.68	GROUND SURFACE ELEVATION(m)
(111.1)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION(m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:2500

0 25 50 75 100 125 150 175m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	PROPOSED SERVICES UPDATED TO DRAWING	24/01/2022	NS
1	UPDATED CONCEPTUAL PLAN TO DRAWING	12/01/2022	NS

STRATHBURN ALMONTE REGIONAL INC.

GEOTECHNICAL INVESTIGATION
PROPOSED OFF-SITE SERVICES
COUNTY ROAD NO. 29 AND STRATHBURN STREET

ALMONTE, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2500

Date: 01/2023

Drawn by: RCG

Report No.: PG6260-LET.01

Checked by: NS

Dwg. No.: PG6260-4

Approved by: DP

Revision No.: 2

p:\autocad drawings\geotechnical\pg62xx\pg6260\pg6260-1 test hole location plan (rev.02) (off-site).dwg