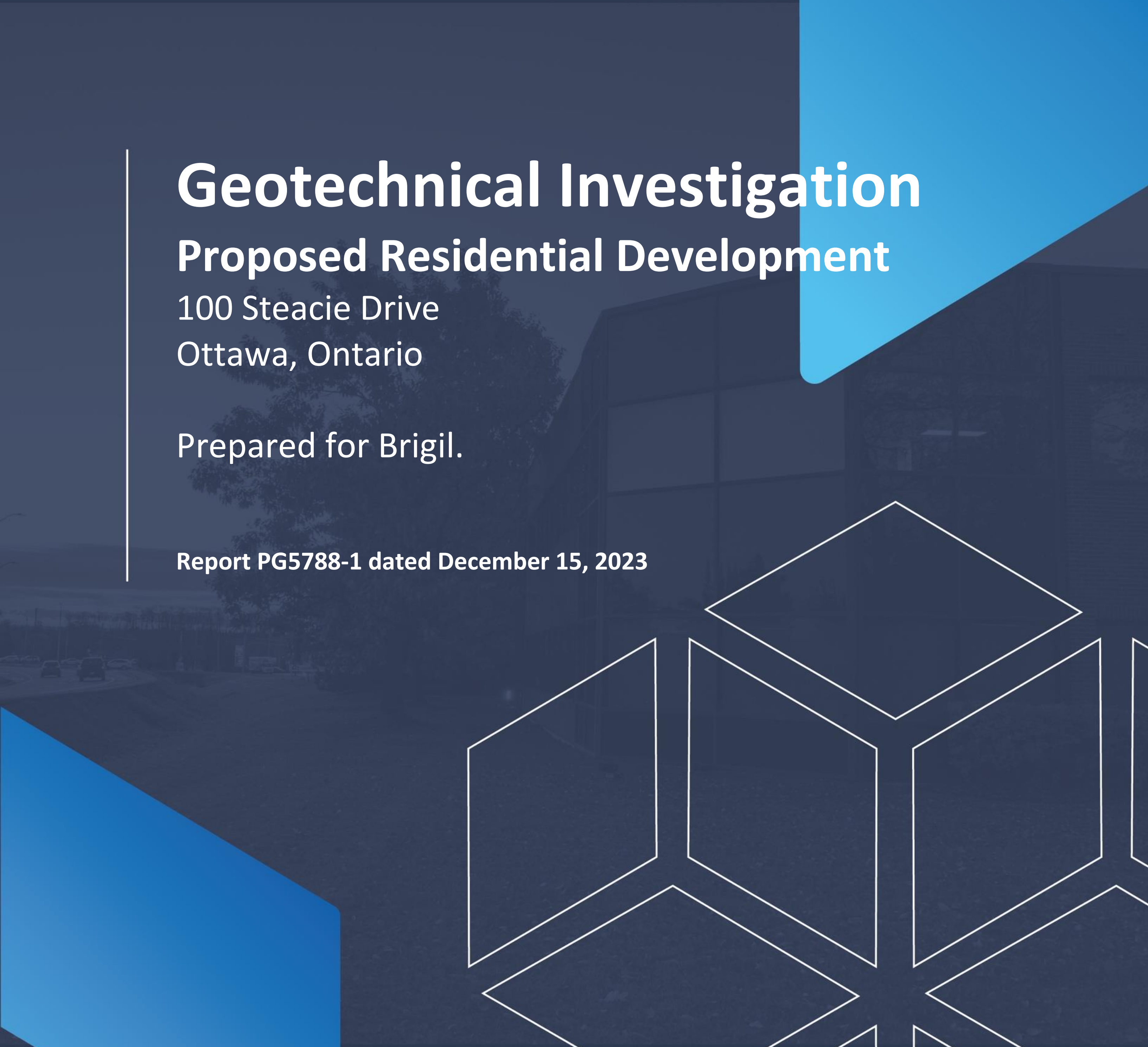

Geotechnical Investigation

Proposed Residential Development

100 Steacie Drive
Ottawa, Ontario

Prepared for Brigil.

Report PG5788-1 dated December 15, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Brigil to conduct a geotechnical investigation for the proposed residential development (subject site) to be located at 100 Steacie Drive (Kanata) in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of test pits and existing soils information.
- Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will include two interconnected midrise residential buildings over a single level of parking structure. The site will also feature an outdoor amenity space or park in front of the buildings. Associated access lanes and hardscaped areas are also anticipated as part of the development.

The development is anticipated to be municipally serviced by water, storm, and sanitary services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on November 27, 2023. At that time, four (4) test pits were excavated to a maximum depth of 4.5 m below existing grade using an excavator. A previous investigation was conducted by others on site and consisted of sixteen (16) boreholes advanced to a maximum depth of 5.8 m below existing grade. The test hole locations from the current investigation were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG5788-1 - Test Hole Location Plan included in Appendix 2.

All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The test pitting procedure consisted of excavating to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the test pits from the current investigation were recovered from the side walls of the open excavation and all soil samples were initially classified on site. All samples were transported to our laboratory for further examination and classification. The depths at which the grab samples were recovered from the test pits are shown as ‘G’ on the Soil Profile and Test Data sheets in Appendix 1.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils using a vane apparatus.

The subsurface conditions observed in the test pits were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

The open hole groundwater infiltration levels were observed at the time of excavation at each test pit location. Our observations are presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The ground surface elevations were referenced to a geodetic datum and were surveyed using a high precision, handheld GPS unit. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG5788-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of one (1) shrinkage test and two (2) Atterberg limit tests were completed on selected soil samples. The results are presented in Subsection 4.2 and Atterberg Limit Results and Shrinkage Test Results, presented in Appendix 1.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless directed otherwise.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures by Paterson. The sample was 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 Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of an undeveloped site, characterized by dense vegetation and boulders and/or rock outcrops. It exhibits a gradual slope from north to south. Bedrock outcrop was observed in the center of the subjected site, within the footprint of the two buildings. To the north, the site is bordered by a railway, while an adjacent commercial building is situated to the east. To the west and south, the site is neighbored by a park and a residential development. Additionally, a high voltage overhead powerline traverses the southern portion of the site, and a multi-use pathway meanders between Steacie Drive, the park, and the residential development.

It should also be noted that an existing sanitary sewer crosses the site from south to north. Based on available information the sewer invert is located between 6.5 m to 8.0 m below ground surface.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of a layer of 0.1 to 0.3 m thick topsoil, underlain by glacial till and/or hard brown silty clay, and occasionally dark brown fill composed of silty clay with sand gravel and cobles. The glacial till generally consists of a stiff to hard, brown silty clay mixed with silty sand with some cobbles, gravel and boulders and was observed to extend to the bedrock surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test pits location.

Atterberg Limits Testing

Atterberg limits testing, as well as associated moisture content testing, was completed on select silty clay samples where encountered. The results of the Atterberg limits test are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1. The results of the moisture content test are presented on the Soil Profile and Test Data Sheet in Appendix 1. The tested silty clay samples classify as inorganic silt of high plasticity (CL) in accordance with the Unified Soil Classification System.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
TP 2-23 G6	2.3 to 2.4	46	21	25	46	CL
TP 3-23 G6	2.3 to 2.4	48	22	26	48	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content.
CL: Low-Plasticity Clay

Shrinkage Test

Linear shrinkage testing was completed on a sample recovered from 2.3 m depth from test pit TP 2-23 and yielded a shrinkage limit of 17.07 and a shrinkage ratio of 1.86.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists primarily of quartzite, with an anticipated overburden thickness ranging across site from 1 to 10 m depth.

4.3 Groundwater

The groundwater infiltration was measured within the side walls of the test pits at the time of excavation on November 27, 2023. The measured open hole groundwater infiltration readings are presented in Table 2 below and in the Soil Profile and Test Data sheets in Appendix 1.

Table 2 - Summary of Groundwater Infiltration Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
TP 1-23	91.44	Dry	-	November 27, 2023
TP 2-23	87.13	2.5	84.63	
TP 3-23	86.99	4.0	82.99	
TP 4-23	87.50	1.4	86.10	

Note: Ground surface elevations at test hole locations are referenced to a geodetic datum.

It should be noted that the groundwater infiltration levels could be influenced by surface water infiltrating the upper soil profile. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on the groundwater infiltration readings and the soil sample observations, the long-term groundwater table can be expected at approximately 4 to 5 m below ground surface. Groundwater levels are subject to seasonal fluctuations and therefore may vary at the time of construction. The recorded groundwater infiltration levels are noted on the applicable Soil profile and Test Data sheets presented in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed multi-story residential building is anticipated to be founded over conventional shallow footings placed on an undisturbed, stiff silty clay bearing surface or on the bedrock surface.

Due to the presence of the bedrock outcrop observed in the middle of the subjected site, bedrock removal is anticipated to be required to complete the underground parking level and/or site servicing work. Line drilling and controlled blasting where large quantities of bedrock need to be removed may be required. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

Due to the presence of the silty clay layer, the subject site will have a permissible grade raise restriction. The permissible grade raise recommendations are discussed in Subsection 5.3.

The above and other considerations are discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant amounts of organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures.

Bedrock Removal

Considering the bedrock composition found in that region, it is expected that line-drilling in conjunction with hoe-ramming and controlled blasting will be required to remove the bedrock. 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.

Prior to considering blasting operations, the effects on the existing services, buildings and other structures should be addressed. A pre-blast or construction survey located in proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

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

Lean Concrete In-Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (15 to 20 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance provided in subsection 5.3.

Vibration Considerations

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

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, 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). These guidelines are for current construction standards.

Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill placed for grading beneath the building footprints should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II or blast rock fill approved by Paterson. The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in a maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the buildings should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD). Overbreak in bedrock below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material’s SPMDD.
Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a geo-composite drainage membrane such as Miradrain G100N or Delta Drain 6000 connected to a perimeter drainage system. a composite drainage membrane.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 150 mm. Where the fill is open graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

Protection of Subgrade

Since the subgrade building foundations is mostly expected to consist of firm silty clay and compact glacial till, it is recommended that a minimum 50 to 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic or workers and equipment.

5.3 Foundation Design

Bearing Resistance Values

Based on the subsurface profile encountered in the test holes, it is expected that the proposed buildings will be founded on conventional spread footings placed on undisturbed, stiff to firm grey silty clay, compact glacial till or clean surface sounded bedrock (southeastern portion of the eastern building).

Using continuously applied loads, footings for the proposed buildings can be designed using the bearing resistance values presented in Table 3.

Table 3 - Bearing Resistance Values

Bearing Surface	**Bearing Resistance Value at SLS (kPa)**	**Factored Bearing Resistance Value at ULS (kPa)**
Surface-Sounded Bedrock	N/A	1,500
Compact to Dense Glacial Till	200	300
Hard to Very Stiff Brown Silty Clay	150	225
Stiff to Firm Grey Silty Clay	100	150

Note: Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over a brown silty clay bearing surface can be designed using the above noted bearing resistance values. Strip footings, up to 2 m wide, and pad footings, up to 6 m wide, placed over a grey silty clay bearing surface can be designed using the above noted bearing resistance values.

A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS. Bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or undisturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures, or open joints which can be detected from surface sounding with a rock hammer. Overbreak in bedrock located directly below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

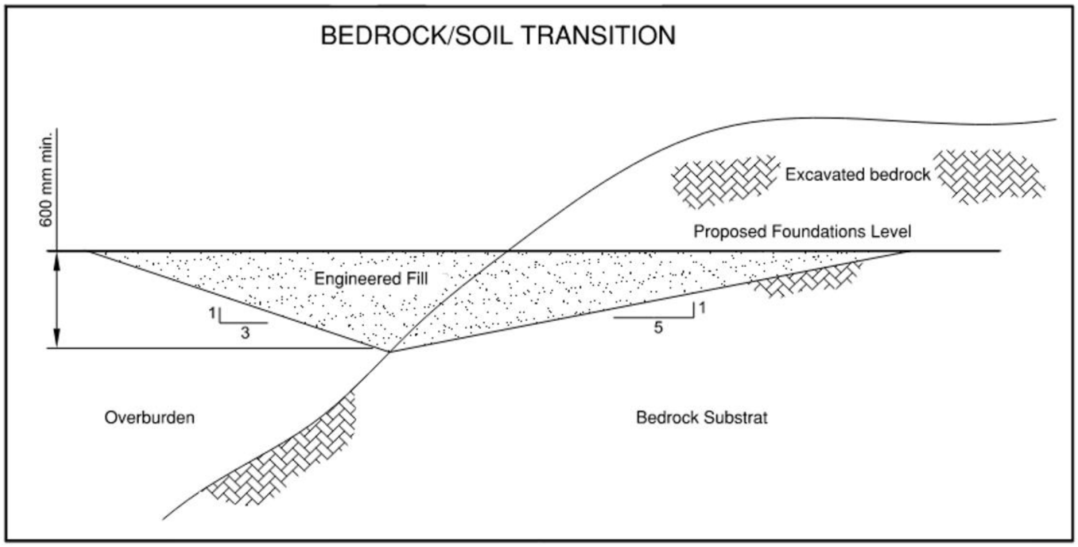
Frictional Resistance

An unfactored coefficient of friction of 0.7 is considered applicable for the design of concrete footings supported on clean, surface sounded bedrock at this site.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to provide Bedrock/soil transition to reduce the risks of excessive differential settlements. This transition involves profiling the rock with a slope of 1.0 vertical to 5.0 horizontal, while the soils will be profiled with a slope of 1.0 vertical to 3.0 horizontal, reaching a depth of 600 mm at their point of contact relative to the projected foundation level.

The excavation should be filled with clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD). The figure below illustrates a cross-section of a Bedrock/soil transition.



In case it is not possible to follow the detailed profile mentioned above, additional reinforcing bars should be integrated where the transition between the soil and the bedrock occurs. This will reinforce the section of the foundation footing affected by this transition.

Settlement

The total and differential settlement will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 to 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance value provided herein will be subjected to negligible potential postconstruction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil.

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

A heavily fractured, weathered bedrock and/or overburden bearing medium will require a lateral support zone of 1H:1V (or flatter).

Permissible Grade Raise Restrictions

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site, a permissible grade raise restriction of **1.5 m** is recommended in the immediate area of settlement sensitive structures and where silty clay is encountered at underside of footing elevations. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. The soil underlying the subject site are not susceptible to liquefaction. A higher site class, such as Class A or B, may be achievable for foundations placed within 3 m of the bedrock surface. However, a site-specific shear wave velocity test is required to be completed to confirm the seismic site classification.

5.5 Slab-on-Grade and Basement Slab Construction

With the removal of all topsoil and deleterious materials within the footprint of the proposed buildings, an approved soil subgrade or bedrock surface, approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

Where silty sand or glacial till is encountered below the slab, provisions should be made to proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and replaced with appropriate backfill material.

The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm of clear crushed stone. For slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone compacted to a minimum of 98% of the materials SPMDD.

An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. Alternatively, excavated bedrock could be used as select subgrade material around the proposed building footings if well-graded blast-rock with a maximum particle size of 150 mm in its longest dimension and sampled/reviewed and approved by Paterson at the time of crushing and prior to use throughout the subject site.

All backfill material within the footprint of the proposed building should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD. An engineered fill such as an OPSS Granular A, Granular B Type II or blast rock compacted to 98% of its SPMDD could be placed around the proposed footings. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. This is discussed further in Section 6.1 of this report.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face.

Where blind side pours are proposed on a vertical rock face, a supplemental layer of a minimum of 25 mm of compressible insulation material should be used in combination with the shale protection described in sub section 5.3.

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component ($\triangle P_{AE}$).

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $Ko \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45\text{-}a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above. The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Design

Rigid Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below.

Table 4 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
Specified by Others	**Exposure Class C2 – 32 MPa Concrete** (5 to 8 % Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE Fill or OPSS Granular B Type I or II or material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Flexible Pavement Structure

The flexible pavement structure presented in Table 5 and Table 6 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 5 - Recommended Pavement Structure – Driveways Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 6 - Recommended Pavement Structure – Access Lanes

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, select subgrade material or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. 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 B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on 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 load carrying capacity.

Due to the low permeability of the clay soils subgrade materials that may be encountered, consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. For slab-on-grade structures, the system is considered optional throughout landscaped areas. It is recommended that the drainage system consist of the following:

❑ Where foundation walls will be double-sided poured, a composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) is recommended to be installed directly onto the exterior foundation wall in combination with a damp proofing membrane between the top of the footing and finished grade.

❑ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by Paterson.

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall. Elevator shafts located below the underslab drainage system should be waterproofed and provided with a PVC waterstop at the shaft wall and footing interface.

Review of architectural design drawings should be completed by Paterson for the above-noted items once the building design has been finalized and prior to tender. It is recommended that Paterson reviews all details associated with the foundation drainage system prior to tender.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the buildings foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover alone, or a combination of soil cover in conjunction with foundation insulation should be provided in this regard.

The underground parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 600 mm m of soil cover, in conjunction with foundation insulation and as reviewed and advised by Paterson, should be provided.

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

Where the bedrock is considered frost susceptible (i.e., weathered bedrock or bedrock with significant fissures filled with soil), foundation insulation will need to be provided. Alternatively, frost susceptible bedrock will need to be removed and replaced with lean concrete (minimum 15 MPa 28-day strength). It is recommended Paterson field personnel review the frost susceptibility of bedrock surface located within 1.8 m of finished grade.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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. The subsurface soil is considered to be mainly a Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods.

The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below.

These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base.

It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 7.

Table 7 – Geotechnical Parameters Lateral Properties

Material Description	**Unit Weight (kN/m^3)**		**Friction Angle (°) φ'**	**Friction Factor, $\tan\delta$**	**Earth Pressure Coefficients**		
	Drained γ_{dr}	**Effective γ'**			**Active K_A**	**At-Rest K_O**	**Passive K_P**
Glacial till	20	10.2	35	0.4	0.27	0.43	3.7
Silty Clay	18	10	30	0.3	0.33	0.5	3
Engineered Fill (Granular A)	22	13.5	40	0.5	0.22	0.36	4.6
Engineered Fill (Granular B Type II)	22.5	14	42	0.5	0.2	0.33	5.04
Bedrock	23.5	15.2	55	0.6	0.18	0.1	10

Notes:

I. The earth pressure coefficients provided are for horizontal profile.

II. For soil above the groundwater level the “drained” unit weight should be used and below groundwater level the “effective” unit weight should be used.

III. Existing fill should be free of significant amounts of deleterious material such as those containing organic materials, wood chips and peat. The fill should be approved by Paterson prior to placement

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 is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock 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.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular. However, when the bedding is located within bedrock subgrade, a minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement. Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

Where hard surface areas are considered above the trench backfill, 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 minimize 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.

Existing Sanitary Service

It is understood that an existing sanitary sewer crosses the site from south to north. Based on available information the sewer invert is located between 6.5 m to 8.0 m below ground surface. Based on our review of available subsurface conditions in the area, the local overburden soils consist mainly of silty clay and some glacial till. Excavations through soils of this type are considered acceptable to be cut back at a slope of 1H:1V above the groundwater table. Further, based on the geotechnical investigation, it is anticipated that the bedrock surface in this area is between 5 m - 9 m below ground surface. Excavations through bedrock can be completed with nearly vertical sides.

Given the depth of excavation required for sewer repair, maintenance or replacement, it is anticipated that groundwater infiltration into the excavations will be relatively low and should be controllable using open sumps.

Where the required overburden excavation is greater than 3 m, it is recommended that any maintenance work be completed with a trench box. This will allow for greater protection and will minimize the required excavation width at the surface.

Therefore, in accordance with the recommendations above, an easement of 10 m width is considered adequate to safely allow access to the existing sewer pipe for future repair, maintenance or replacement.

It is expected that service crossing under the existing railway to the north of the site may be required. The borehole results indicate that the crossings will be completed in very stiff to stiff silty clay. Based on the size of the required excavation, horizontal auger boring trenchless excavation should be considered. However, the contractor should be fully responsible for the selection of the trenchless technology which best fits the contract requirements, the equipment availability, staff capabilities and experience. Pipeline crossing must meet TC E-10 requirements at railway crossings.

Horizontal Auger Boring method requires the excavation of entry and receiving pits to accommodate the jacking equipment. A steel casing is advanced by jacking with simultaneous removal of spoils using helical augers within the casing. Successive lengths of casing are welded together prior to each advance. The lead casing is generally equipped with a shield or thickened leading end to create a minor amount of overbreak to reduce shear stress.

The main advantage of this system is that, with suitable soil conditions and good workmanship, minimal settlement generally occurs due to the simultaneous installation of the casing. However, the auger head should be kept 0.5 metres behind the end of the casing at all times to minimize over excavation and loss of ground with resultant post construction settlements based on the presence of very stiff to hard silty clay with sand fill. The use of an injected bentonite lubricant will probably be required to minimize casing friction and jacking loads. Care will be required to maintain alignment and grade during the casing installations.

All trenchless work must be carried out by an experienced specialist contractor employing only qualified workers skilled in their trade under the direction of an experienced foreman. The contractor's work plan should include a method of sealing the ends of the bore/casing at the end of each work day or in case of an emergency. It should also include a procedure for compensation grouting should uncontrolled loss of ground or drilling fluid occur. It is recommended that the geotechnical aspects of the contractor's work plan for the proposed crossings be reviewed by Paterson prior to construction. The trenchless contractor is responsible to locate existing services and exposed them as required.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations through the overburden materials should be low to moderate and controllable using open sumps.

Higher infiltration rates may be encountered below the bedrock surface; however, infiltration is expected be controlled using open sumps. Provisions should be carried out for using higher capacity open sump systems for excavations undertaken below the bedrock surface.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

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 of 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The bedrock and overburden material present on site are considered frost susceptible.

Where excavations are completed in proximity to existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means. 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 foundation is protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

Under winter conditions, if snow and ice is present within the blast rock or other imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

6.7 Corrosion Potential and Sulphate

The results of analytical testing from an adjacent site show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a non-aggressive to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for the recovered silty clay samples at selected locations throughout the subject site. The soil samples were recovered from elevations below the anticipated design underside of footing elevation and 3.5 m depth below anticipated finished grade. The results of our testing are presented in Table 1 in Subsection 4.2 and in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture level and consistency, the silty clay across the subject site is considered to be a clay of low to medium potential for soil volume change.

The following tree planting setbacks are recommended for the low to medium sensitivity silty clay deposit and where trees are located near buildings founded on cohesive soils. It should be noted that footings bearing upon a compact glacial till or surface sounded bedrock will not be subject to tree planting setbacks restrictions.

- ☐ Large trees (mature height over 14 m) can be planted within these areas provided that a tree to foundation setback equal to the full mature height of the tree can be provided.

- ☐ Tree planting setback limits may be reduced to 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m), provided that the conditions noted below are met.

- ☐ A small tree must be provided with a minimum of 25 m^3 of available soils volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

- ☐ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

☐ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the Grading Plan.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. The three varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e., Manitoba Maples) and, as such, they should not be considered in the landscaping design.

7.0 Recommendations

For the foundation design data provided herein to be applicable that a material testing and observation services program is required to be completed.

The following aspects be performed by the geotechnical consultant:

☐ Review preliminary and detailed grading, servicing, and structural plan(s) from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.

☐ Review of architectural plans pertaining to foundation and underfloor drainage systems and waterproofing details for elevator shafts.

For the foundation design data provided herein to be applicable, a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

☐ Review the bedrock stabilization and excavation requirements at the time of construction.

☐ Review and inspection of the installation of the foundation and underfloor drainage systems and elevator waterproofing.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and fill materials.

☐ 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.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant. All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractor's construction methods, equipment capabilities and schedules.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Brigil or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
December 15, 2023
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Fabrice Venadiambu, CPI, E.I.T.　　Joey R. Villeneuve,M.A.Sc., P.Eng., ing.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS TESTING RESULTS

SHRINKAGE MILITS TESTING

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

PATERSON GROUP

SOIL PROFILE AND TEST DATA
GEOTECHNICAL INVESTIGATION
100 Steacie Drive, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 350688.565 **NORTHING:** 5021956.353 **ELEVATION:** 91.44 m

PROJECT: Proposed Residential Development

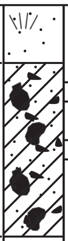
BORINGS BY: Excavator

REMARKS:

DATE: November 27, 2023

FILE NO. PG5788

HOLE NO. TP 1-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Piezometer Construction
Ground Surface EL 91.44 m										
TOPSOIL 0.3 m EL 91.14 m						0				No Data
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders 1.2 m EL 90.24 m		G1 G2				1				
End of Test Pit Practical refusal to excavation on bedrock surface at 1.20m depth.						2				
						3				
						4				
						5				
						6				

RSLog / Geotechnical Test Pit - Geodetic / paterson-group / admin / December 06, 2023 02:27 PM

SOIL PROFILE AND TEST DATA
GEOTECHNICAL INVESTIGATION
100 Steacie Drive, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 350760.323 **NORTHING:** 5022000.578 **ELEVATION:** 87.13 m

PROJECT: Proposed Residential Development

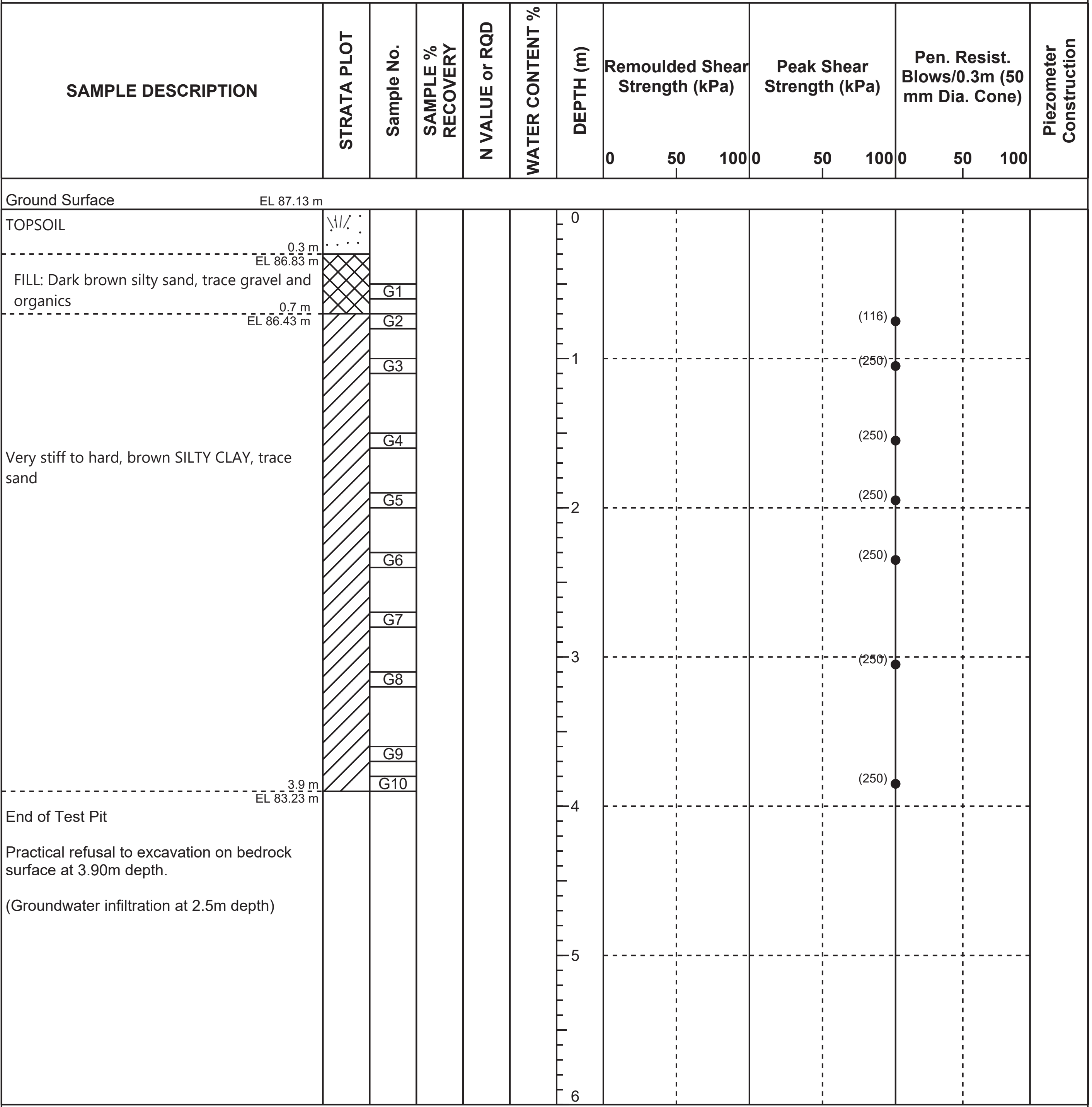
BORINGS BY: Excavator

REMARKS:

DATE: November 27, 2023

FILE NO. PG5788

HOLE NO. TP 2-23



SAMPLE DESCRIPTION

Ground Surface EL 87.13 m

TOPSOIL

0.3 m
EL 86.83 m

FILL: Dark brown silty sand, trace gravel and organics

0.7 m
EL 86.43 m

Very stiff to hard, brown SILTY CLAY, trace sand

3.9 m
EL 83.23 m

End of Test Pit

Practical refusal to excavation on bedrock surface at 3.90m depth.

(Groundwater infiltration at 2.5m depth)

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

100 Steacie Drive, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 350696.862 **NORTHING:** 5022027.551 **ELEVATION:** 86.99 m

PROJECT: Proposed Residential Development

BORINGS BY: Excavator

REMARKS:

DATE: November 27, 2023

FILE NO. PG5788

HOLE NO. TP 3-23

SAMPLE DESCRIPTION

Ground Surface EL 86.99 m

TOPSOIL 0.2 m EL 86.79 m

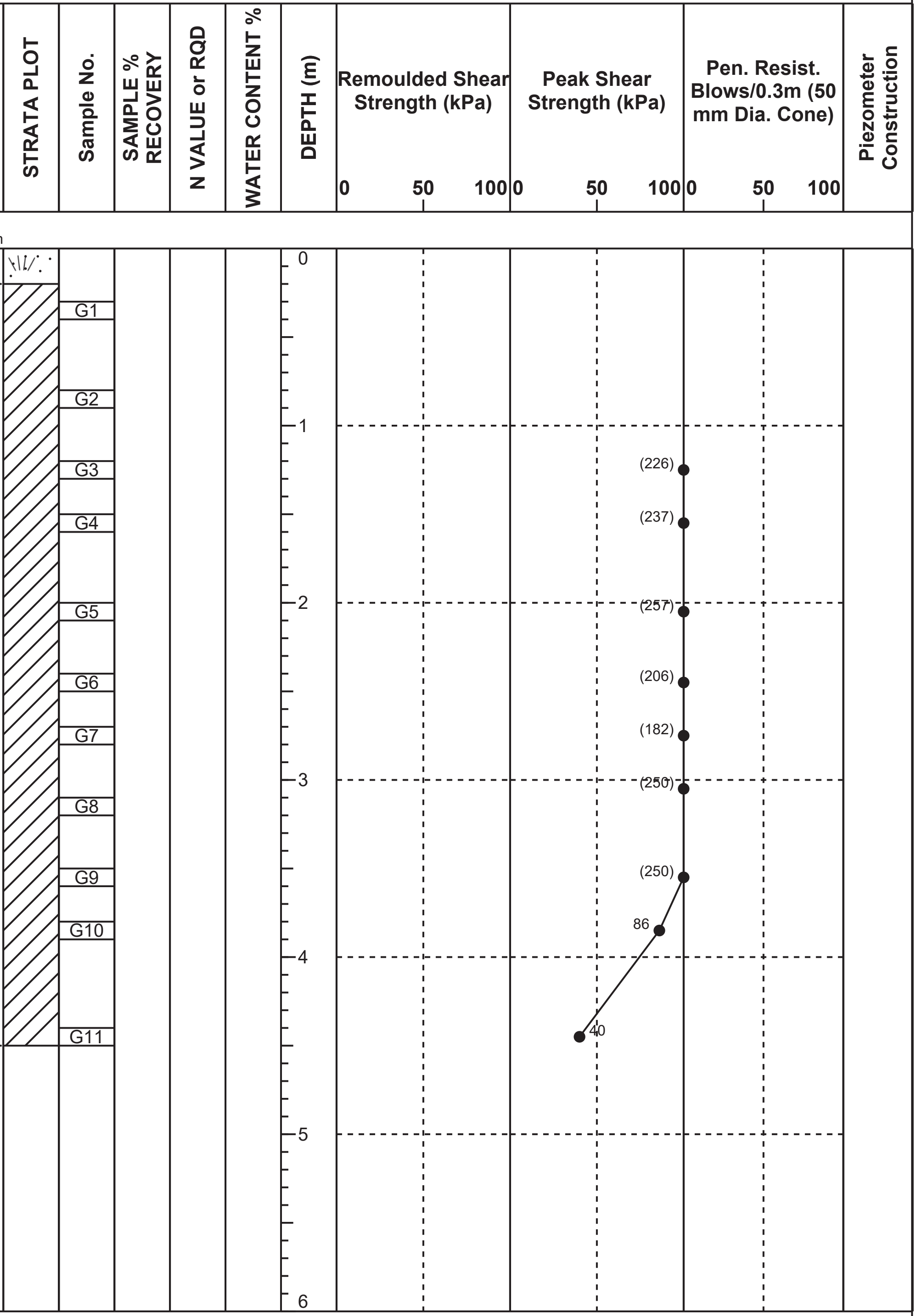
Hard to very stiff, brown SILTY CLAY

- stiff to firm and grey by 3.8m depth

4.5 m EL 82.49 m

End of Test Pit

(Groundwater infiltration at 4.0m depth)



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SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

100 Steacie Drive, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 350671.479 **NORTHING:** 5022015.775 **ELEVATION:** 87.5 m

PROJECT: Proposed Residential Development

BORINGS BY: Excavator

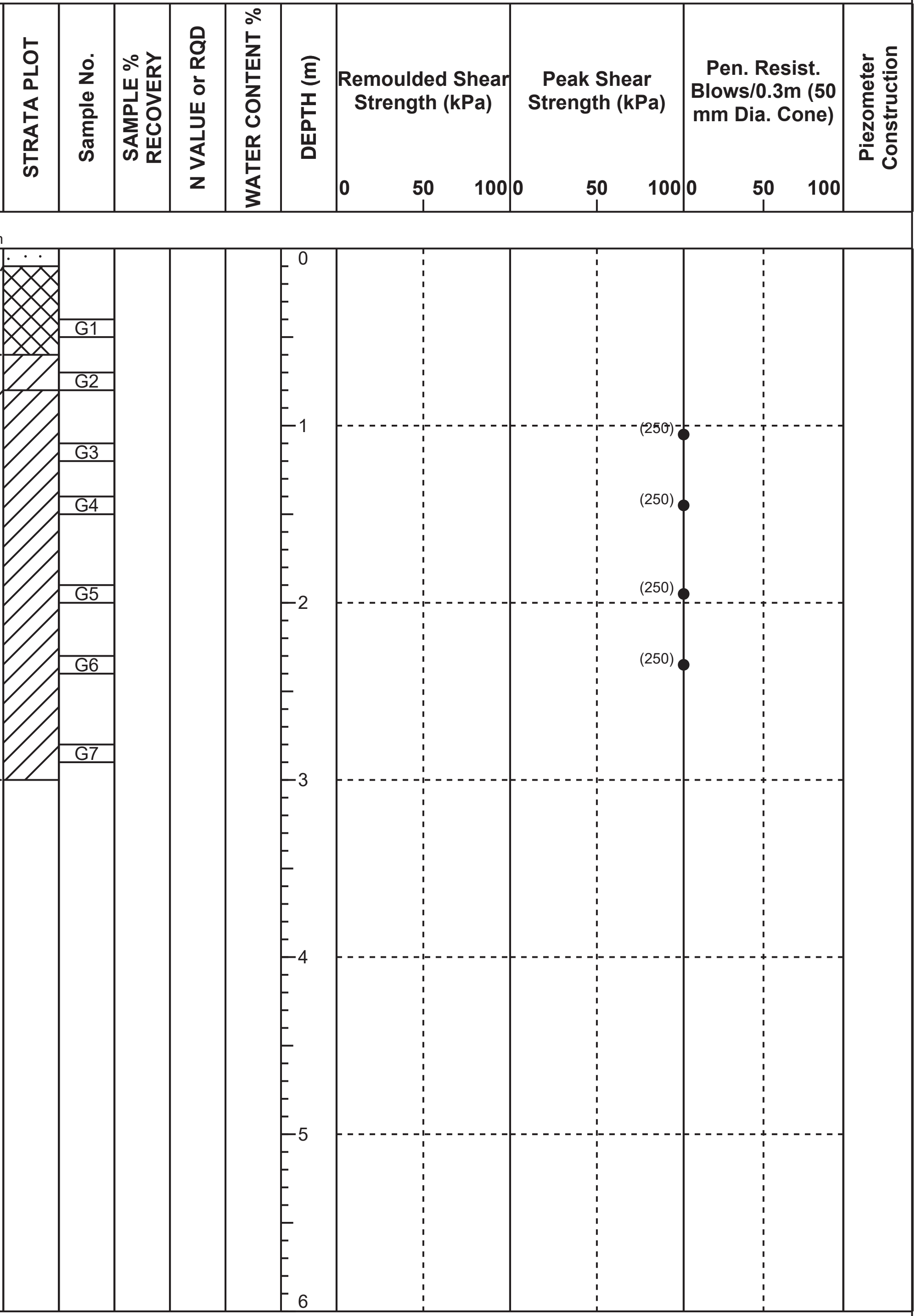
REMARKS:

DATE: November 27, 2023

FILE NO. PG5788

HOLE NO. TP 4-23

SAMPLE DESCRIPTION	Depth	Elevation
Ground Surface		EL 87.5 m
TOPSOIL	0.1 m	EL 87.4 m
FILL: Dark brown silty clay with sand, gravel and cobbles	0.6 m	EL 86.9 m
Brown SILTY CLAY with sand	0.8 m	EL 86.7 m
Hard, brown SILTY CLAY - grey with sand by 1.4m depth	3 m	EL 84.5 m
End of Test Pit (Groundwater infiltration at 1.4m depth)		



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 strength of cohesionless soils is the relative density, 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.

Relative Density	'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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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 is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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

PS - Piston sample

AU - Auger sample or bulk sample

WS - Wash sample

RC - Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture 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 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)

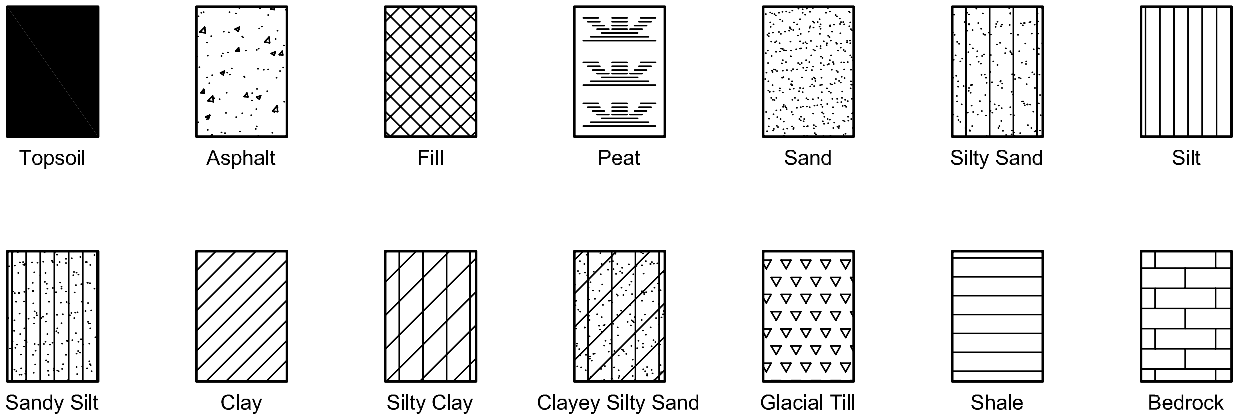
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

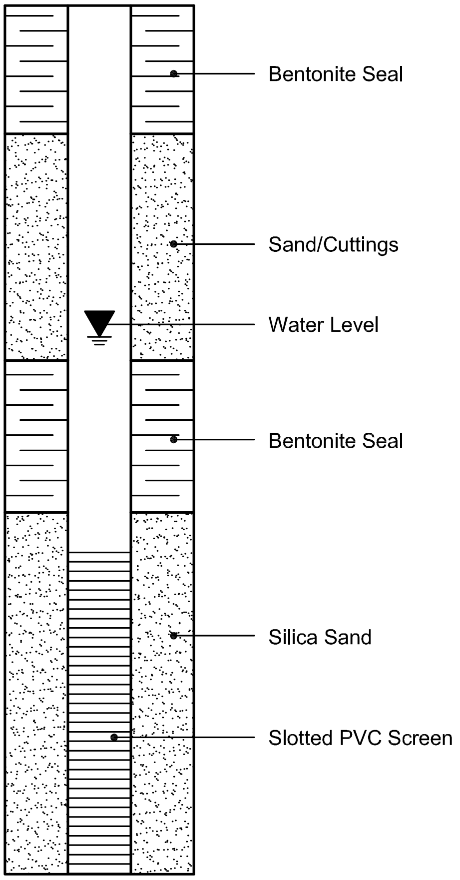
SYMBOLS AND TERMS (continued)

STRATA PLOT

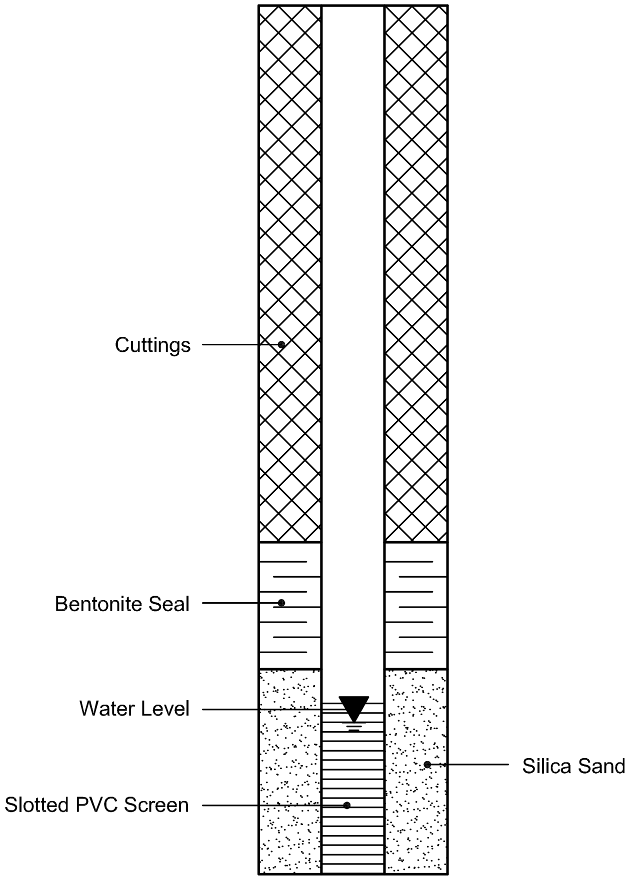


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PATERSON GROUP

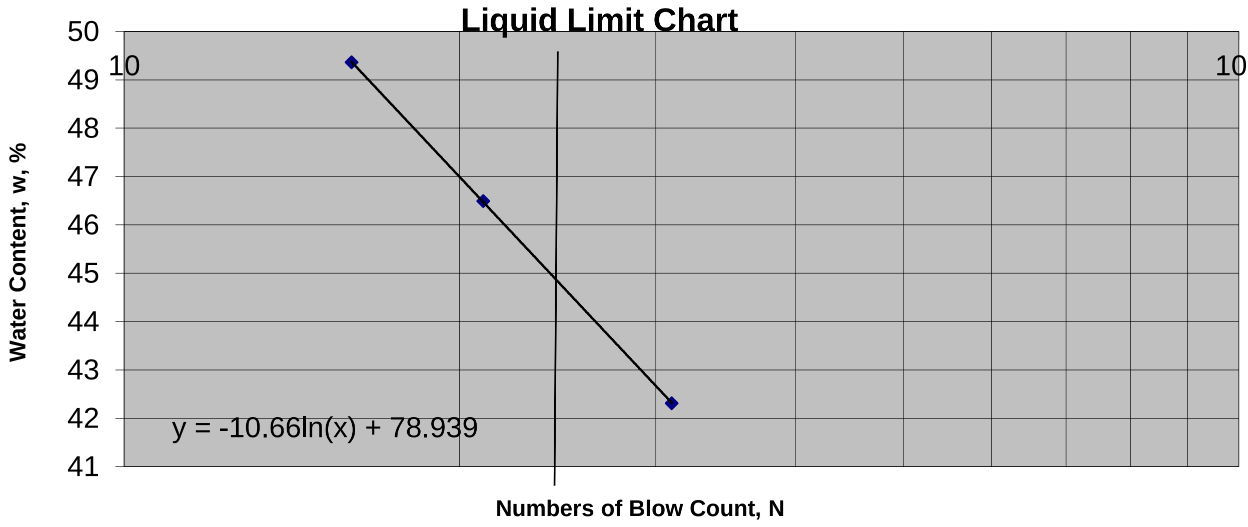
ATTERBERG LIMITS
LS-703/704

CLIENT:	Brigil	FILE NO.:	PG5788
PROJECT:	100 Steacie Drive	DATE SAMPLED:	27-Nov
LOCATION:	TP2-23 G6 @ 2.3m - 2.4m	DATE REPORTED:	11-Dec

CAN NO.	**4**	**11**	**16**				
WT. OF CAN	8.70	8.71	8.75				
WT. OF SOIL & CAN	20.29	19.55	19.11				
WT. OF DRY SOIL & CAN	16.46	16.11	16.03				
WT. OF MOISTURE	3.83	3.44	3.08				
WT. OF DRY SOIL & CAN	7.76	7.4	7.28				
WATER CONTENT, w, %	**49.36**	**46.49**	**42.31**				
NO. OF BLOWS, N	16	21	31				

CAN NO.	**9**	**12**
WT. OF CAN	19.37	16.75
WT. OF SOIL & CAN	27.94	26.75
WT. OF DRY SOIL & CAN	26.42	25.00
WT. OF MOISTURE	1.52	1.75
WT. OF DRY SOIL & CAN	7.05	8.25
WATER CONTENT, w, %	**21.56**	**21.21**

RESULTS	
LIQUID LIMIT	**46**
PLASTIC LIMIT	**21**
PLASTICITY INDEX	**25**



TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

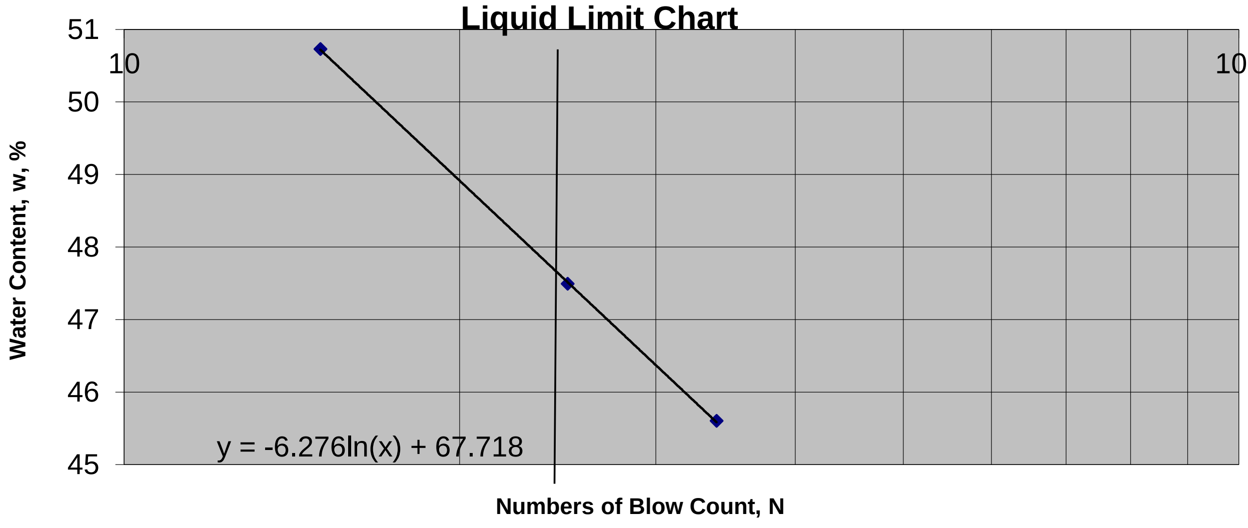
ATTERBERG LIMITS
LS-703/704

CLIENT:	Brigil	FILE NO.:	PG5788
PROJECT:	100 Steacie Drive	DATE SAMPLED:	27-Nov
LOCATION:	TP3-23 G6 @ 2.3m - 2.4m	DATE REPORTED:	11-Dec

CAN NO.	**13**	**70**	**67**				
WT. OF CAN	8.67	7.12	7.22				
WT. OF SOIL & CAN	17.94	18.30	20.28				
WT. OF DRY SOIL & CAN	14.82	14.70	16.19				
WT. OF MOISTURE	3.12	3.6	4.09				
WT. OF DRY SOIL & CAN	6.15	7.58	8.97				
WATER CONTENT, w, %	**50.73**	**47.49**	**45.6**				
NO. OF BLOWS, N	15	25	34				

CAN NO.	**14**	**4**
WT. OF CAN	19.94	19.94
WT. OF SOIL & CAN	29.51	29.30
WT. OF DRY SOIL & CAN	27.79	27.62
WT. OF MOISTURE	1.72	1.68
WT. OF DRY SOIL & CAN	7.85	7.68
WATER CONTENT, w, %	**21.91**	**21.88**

RESULTS	
LIQUID LIMIT	**48**
PLASTIC LIMIT	**22**
PLASTICITY INDEX	**26**



TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

Linear Shrinkage
ASTM D4943-02

CLIENT:	Brigil	DEPTH	2.3m - 2.4m	FILE NO.:	PG5788
PROJECT:	100 Steacie Drive	BH OR TP No:	TP2-23 G6	DATE SAMPLED	27-Nov-23
LAB No:	50512	TESTED BY:	C.P	DATE RECEIVED	30-Nov-23
SAMPLED BY:	A.E	DATE REPORTED:	11-Dec-23	DATE TESTED	4-Dec-23

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(8)	Calibration (Two Trials)	Tin NO.(x22)	
Tare	5.18	Tin	4.84	4.84
Soil Pat Wet + Tare	70	Tin + Grease	5.18	5.18
Soil Pat Wet	64.82	Glass	48.97	48.97
Soil Pat Dry + Tare	48.76	Tin + Glass + Water	91.35	91.35
Soil Pat Dry	43.58	Volume	37.20	37.2
Moisture	**48.74**	**Average Volume**	**37.20**	

Soil Pat + String	43.87
Soil Pat + Wax + String in Air	50.61
Soil Pat + Wax + String in Water	19.64
Volume Of Pat (Vdx)	30.97

RESULTS:

Shrinkage Limit	**17.07**
Shrinkage Ratio	**1.863**
Volumetric Shrinkage	**58.995**
Linear Shrinkage	**14.320**

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PARACEL
LABORATORIES LTD.

Order #: 2348236

Certificate of Analysis

Report Date: 04-Dec-2023

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 28-Nov-2023

Client PO: 58950

Project Description: PG5788

Client ID:		TP2 G5 1.9m to 2.0m	-	-	-		
Sample Date:		27-Nov-23 11:00	-	-	-	-	-
Sample ID:		2348236-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	77.9	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	6.84	-	-	-	-	-
Resistivity	0.1 Ohm.m	140	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	<10	-	-	-	-	-

RECORD OF BOREHOLE 5

PROJECT: 05-068
LOCATION: Refer to Site Plan, Figure 2
BORING DATE: April 11, 2005

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - O (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}) WATER CONTENT, PERCENT Wp ⊢—O^W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		89.49							
		TOPSOIL		89.34 0.15							
		Grey brown SILTY CLAY, weathered crust		89.03 0.46							
1		Loose grey brown SILTY SAND, some gravel			1	50 DO	10				
				87.51 1.98	2	50 DO	5				
2		Very stiff to stiff grey brown SILTY CLAY, weathered crust			3	50 DO	21				
3					4	50 DO	13				
4					5	50 DO	10	⊕ + ⊕ +			
5				84.23 5.26	6	50 DO	10				
		Grey brown silty sand, some gravel, cobbles and boulders (GLACIAL TILL)		84.08 5.41	7	50 DO	5				
6		Practical refusal to excavating on probable BEDROCK End of borehole									Groundwater conditions not observed.

DEPTH SCALE
1 to 30

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW
CHECKED: ACH

PROJECT: 05-068
LOCATION: Refer to Site Plan, Figure 2
BORING DATE: April 11, 2005

RECORD OF BOREHOLE 6

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-7} 10^{-6} 10^{-5} 10^{-4} / WATER CONTENT, PERCENT Wp ⊢ W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		90.32							
		TOPSOIL		90.17 0.15							
		Grey brown CLAYEY SILT, some gravel and cobbles		89.79 0.53							
1		Compact grey brown fine to coarse SAND, some silt and gravel		88.80 1.52	1	50 DO	12				
2		Very stiff to stiff grey brown SILTY CLAY, weathered crust			2	50 DO	15				
					3	50 DO	17				
3					4	50 DO	14				
4					5	50 DO	11				
5				85.14 5.18	6	50 DO	8				
		End of borehole									Groundwater conditions not observed.
6											

DEPTH SCALE
1 to 30

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW
CHECKED: ACH

PROJECT: 05-068

LOCATION: Refer to Site Plan, Figure 2

BORING DATE: April 7, 2005

RECORD OF BOREHOLE 7

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.6 kg; drop 0.76 m

BORING METHOD: Power Auger
200 mm Diameter Hollow Stem

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
0	Ground Surface	91.27
	TOPSOIL	91.24 0.03
	Very stiff to stiff grey brown SILTY CLAY, weathered crust	
1		
	Compact grey brown silty sand, some gravel, cobbles and boulders (GLACIAL TILL)	89.44 1.83
2		
3		
4	Practical refusal to excavating on probable BEDROCK End of borehole	87.28 3.99
5		

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	10
2	50 DO	19
3	50 DO	26
4	50 DO	22
5	50 DO	3

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7} 10^{-6} 10^{-5} 10^{-4}

WATER CONTENT, PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Groundwater conditions not observed.

DEPTH SCALE

1 to 25

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED: ACH

BOREHOLE_RECORD 05-068.GPJ MHECL.GDT 5/5/05

PROJECT: 05-068

LOCATION: Refer to Site Plan, Figure 2

BORING DATE: April 11, 2005

RECORD OF BOREHOLE 8

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-7} 10^{-6} 10^{-5} 10^{-4}; WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		91.36							
		Grey brown silty sand and clayey silt, some boulders, metal wire and brick (FILL MATERIAL)		90.85 0.51							
1		Very stiff to stiff grey brown SILTY CLAY, weathered crust			1	50 DO	13				
2					2	50 DO	15				
		Compact grey brown silty sand, some gravel, cobbles and boulders (GLACIAL TILL)		88.85 2.51	3	50 DO	15				
3					4	50 DO	25				
					5	50 DO	8				
4		Practical refusal to excavating on probable BEDROCK End of borehole		87.55 3.81							Groundwater conditions not observed.
5											

DEPTH SCALE

1 to 25

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED: ACH

BOREHOLE RECORD 05-068.GPJ MHECL.GDT 5/5/05

PROJECT: 05-068

RECORD OF BOREHOLE 9

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: Geodetic

BORING DATE: April 7, 2005

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}) WATER CONTENT, PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		91.45							
		TOPSOIL		0.05							
		Grey brown SILTY CLAY, weathered crust		91.20 0.25							
		Grey brown silty sand, some gravel, cobbles and boulders (GLACIAL TILL)		90.54 0.91	1	50 DO	5				
1		Practical refusal to excavating on probable BEDROCK End of borehole									Groundwater inflow near ground surface on April 7, 2005.
2											
3											
4											
5											

DEPTH SCALE

1 to 25

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED: AC

PROJECT: 05-068

LOCATION: Refer to Site Plan, Figure 2

BORING DATE: April 8, 2005

RECORD OF BOREHOLE 10

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}); WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface	90.14							
		TOPSOIL	90.04 0.10							
		Loose grey brown silty sand, some gravel, cobbles and boulders (GLACIAL TILL)								
1				1	50 DO	7				
		Fresh to faintly weathered amphibolite BEDROCK	89.07 1.07							
2					R.C.		TCR=100% SCR = 91% RQD= 73%			
		End of borehole/corehole	87.50 2.64							Groundwater conditions not observed.
3										
4										
5										

DEPTH SCALE

1 to 25

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED: ACH

PROJECT: 05-068

RECORD OF BOREHOLE 11

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: Geodetic

BORING DATE: April 8, 2005

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-7} 10^{-6} 10^{-5} 10^{-4} WATER CONTENT, PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		88.30							
		TOPSOIL		88.00 0.30							
		Loose to compact grey brown silty sand, some gravel, cobbles and boulders (GLACIAL TILL)									
1					1	50 DO	11				
2					2	50 DO	9				
					3	50 DO	14				
		Practical refusal to excavating on probable BEDROCK End of borehole		85.48 2.82							
3											Groundwater conditions not observed.
4											
5											

DEPTH SCALE

1 to 25

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED: ACH

PROJECT: 05-068

RECORD OF BOREHOLE 12

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

DATUM: Geodetic

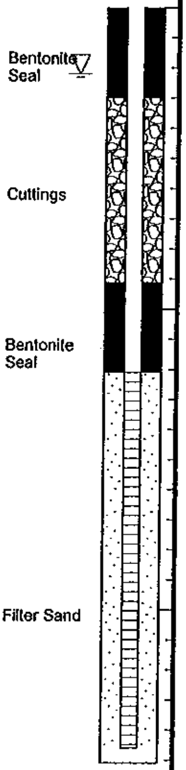
BORING DATE: April 8, 2005

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | SAMPLES | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 | SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80 | HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-7} 10^{-6} 10^{-5} 10^{-4} | WATER CONTENT, PERCENT Wp —○ W — Wl 20 40 60 80 | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m
Ground Surface	86.80			
TOPSOIL	86.67 0.13			
Very stiff to stiff grey brown SILTY CLAY, weathered crust				
		1	50 DO	12
		2	50 DO	18
		3	50 DO	7
Grey brown silty sand, some gravel, cobbles and boulders (GLACIAL TILL)	84.34 2.46			
Practical refusal to excavating on probable BEDROCK End of borehole	2.51			



Groundwater at elevation 86.58 metres below ground surface on April 14, 2005.

DEPTH SCALE 1 to 25

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED: ACH

BOREHOLE RECORD 05-068.GPJ MHECL.GDT 5/5/05

PROJECT: 05-068

RECORD OF BOREHOLE 12 A

SHEET 1 OF 1

LOCATION: 2 metres south of BH 12

DATUM: Geodetic

BORING DATE: April 11, 2005

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat. V - + rem. V - ⊕ Q - ● U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-7} 10^{-6} 10^{-5} 10^{-4} WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
0	Casing	Ground Surface		86.80							
		OVERBURDEN									
1											
2		Fresh to faintly weathered amphibolite BEDROCK	▨	84.67 2.13							
3						R.C.		TCR = 100% SCR = 80% RQD = 80%			
		End of corehole		83.09 3.71							Groundwater conditions not observed.
4											
5											

DEPTH SCALE
1 to 25

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW
CHECKED: ACH

PROJECT: 05-068

RECORD OF BOREHOLE 13

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 2

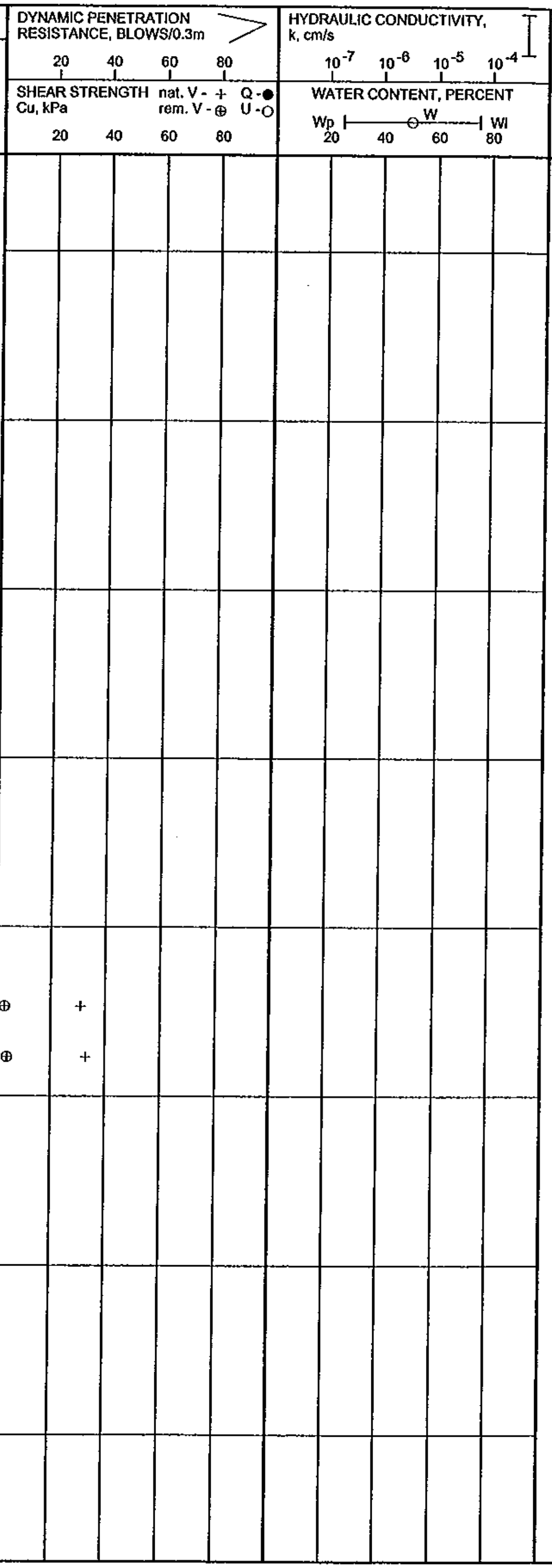
DATUM: Geodetic

BORING DATE: April 8, 2005

SPT HAMMER: 63.6 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
0	Ground Surface		86.41			
	TOPSOIL		86.11 0.30			
	Very stiff to stiff grey brown SILTY CLAY, weathered crust					
1				1	50 DO	7
2				2	50 DO	11
				3	50 DO	9
3				4	50 DO	5
	Firm grey SILTY CLAY		82.75 3.66			
4				5	50 DO	2
5				6	50 DO	PH
	End of borehole		80.62 5.79			
6						
7						
8						



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Groundwater conditions not observed.

BOREHOLE_RECORD 05-068.GPJ MHECL.GDT 5/5/05

DEPTH SCALE

1 to 40

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED: ACH

PROJECT: 05-068

LOCATION: Refer to Site Plan, Figure 2

BORING DATE: April 8, 2005

RECORD OF BOREHOLE 14

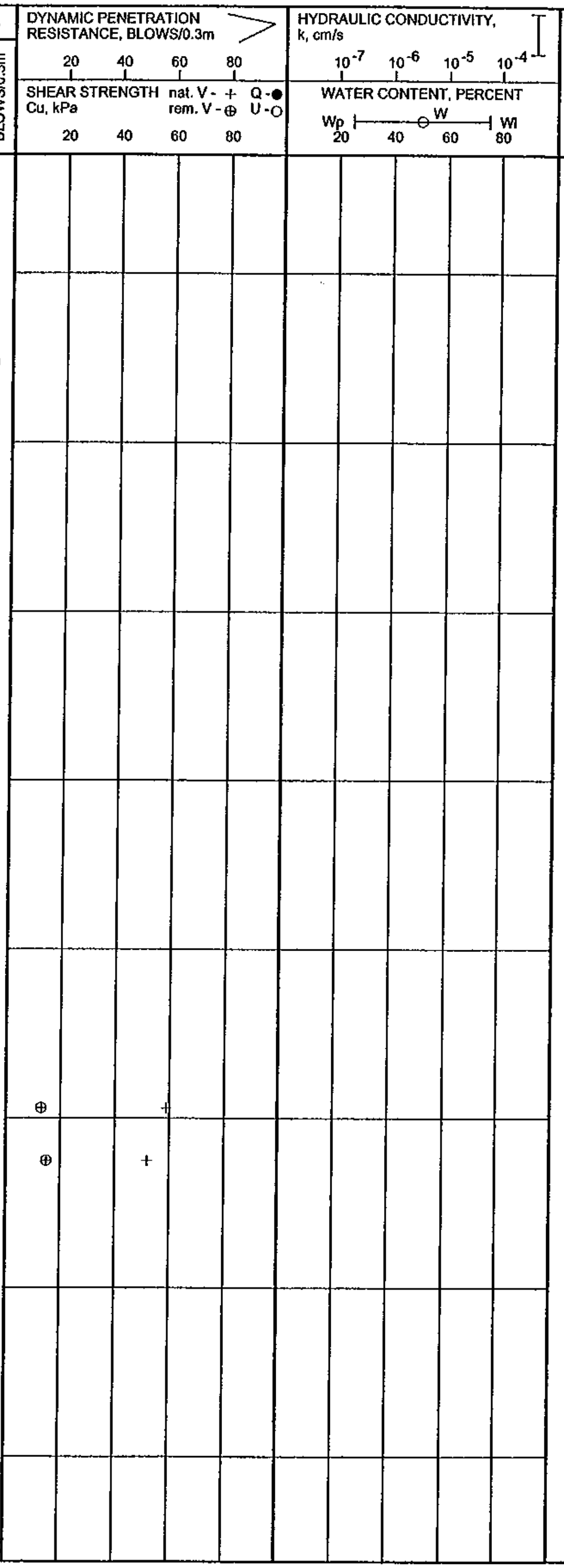
SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.6 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m
0	Ground Surface	86.54			
	Grey brown silty clay FILL MATERIAL	86.41 0.13			
	FORMER TOPSOIL	0.23			
	Very stiff to stiff grey brown SILTY CLAY, weathered crust				
1			1	50 DO	9
2			2	50 DO	6
			3	50 DO	3
3			4	50 DO	5
4	Stiff grey SILTY CLAY	82.73 3.81	5	50 DO	3
5			6	50 DO	2
	End of borehole	80.75 5.79			
6					
7					
8					



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Groundwater conditions not observed.

DEPTH SCALE

1 to 40

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED: ACH

PROJECT: 05-068
LOCATION: Refer to Site Plan, Figure 2
BORING DATE: April 8, 2005

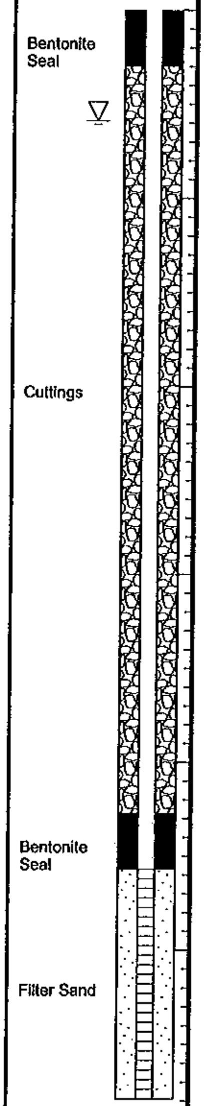
RECORD OF BOREHOLE 15

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.6 kg; drop 0.76 m

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7} 10^{-6} 10^{-5} 10^{-4}
WATER CONTENT, PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80
ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface	87.16			
		TOPSOIL	87.06 0.10			
		Very stiff to stiff grey brown SILTY CLAY, weathered crust				
1				1	50 DO	8
2				2	50 DO	13
				3	50 DO	11
3			84.11 3.05			
		Stiff to firm grey SILTY CLAY		4	50 DO	7
4				5	50 DO	6
5				6	50 DO	4
				7	50 DO	2
		End of borehole	81.37 5.79			
6						
7						
8						

PIEZOMETER OR STANDPIPE INSTALLATION



Groundwater at elevation 86.57 metres below ground surface on April 14, 2005.

BOREHOLE RECORD 05-068.GPJ MHECL.GDT 5/5/05

DEPTH SCALE
1 to 40

Morey Houle Chevrier Engineering Ltd.

LOGGED: BW
CHECKED:

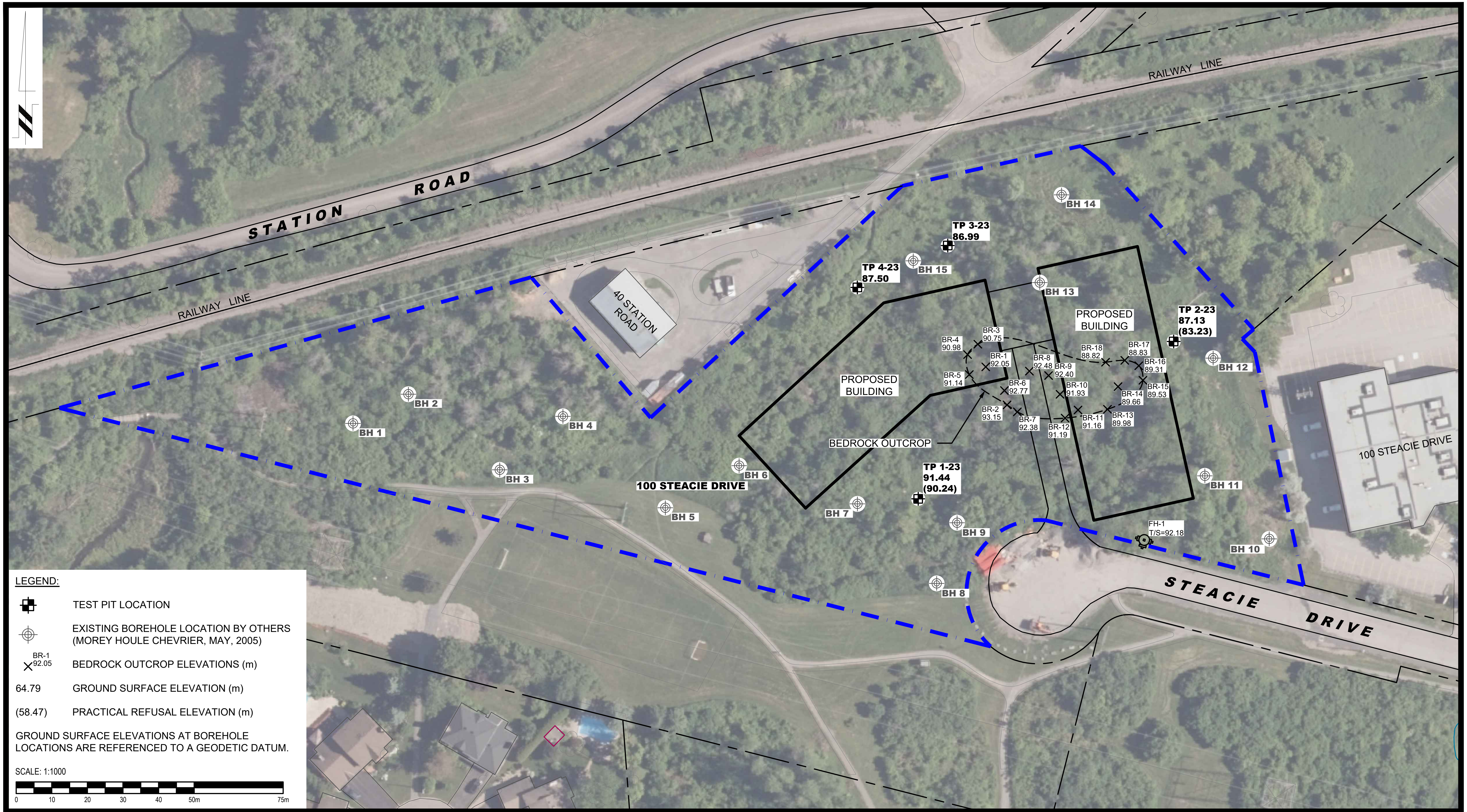
APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5788-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN



LEGEND:

⊞	TEST PIT LOCATION
⌖	EXISTING BOREHOLE LOCATION BY OTHERS (MOREY HOULE CHEVRIER, MAY, 2005)
BR-1 ✕ 92.05	BEDROCK OUTCROP ELEVATIONS (m)
64.79	GROUND SURFACE ELEVATION (m)
(58.47)	PRACTICAL REFUSAL ELEVATION (m)

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP

9 AURIGA DRIVE
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NO.	REVISIONS	DATE	INITIAL

BRIGIL
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
100 STEACIE DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:1000	Date:	11/2023
Drawn by:	ZS	Report No.:	PG5788-1
Checked by:	FV	Dwg. No.:	PG5788-1
Approved by:	JV	Revision No.:	

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