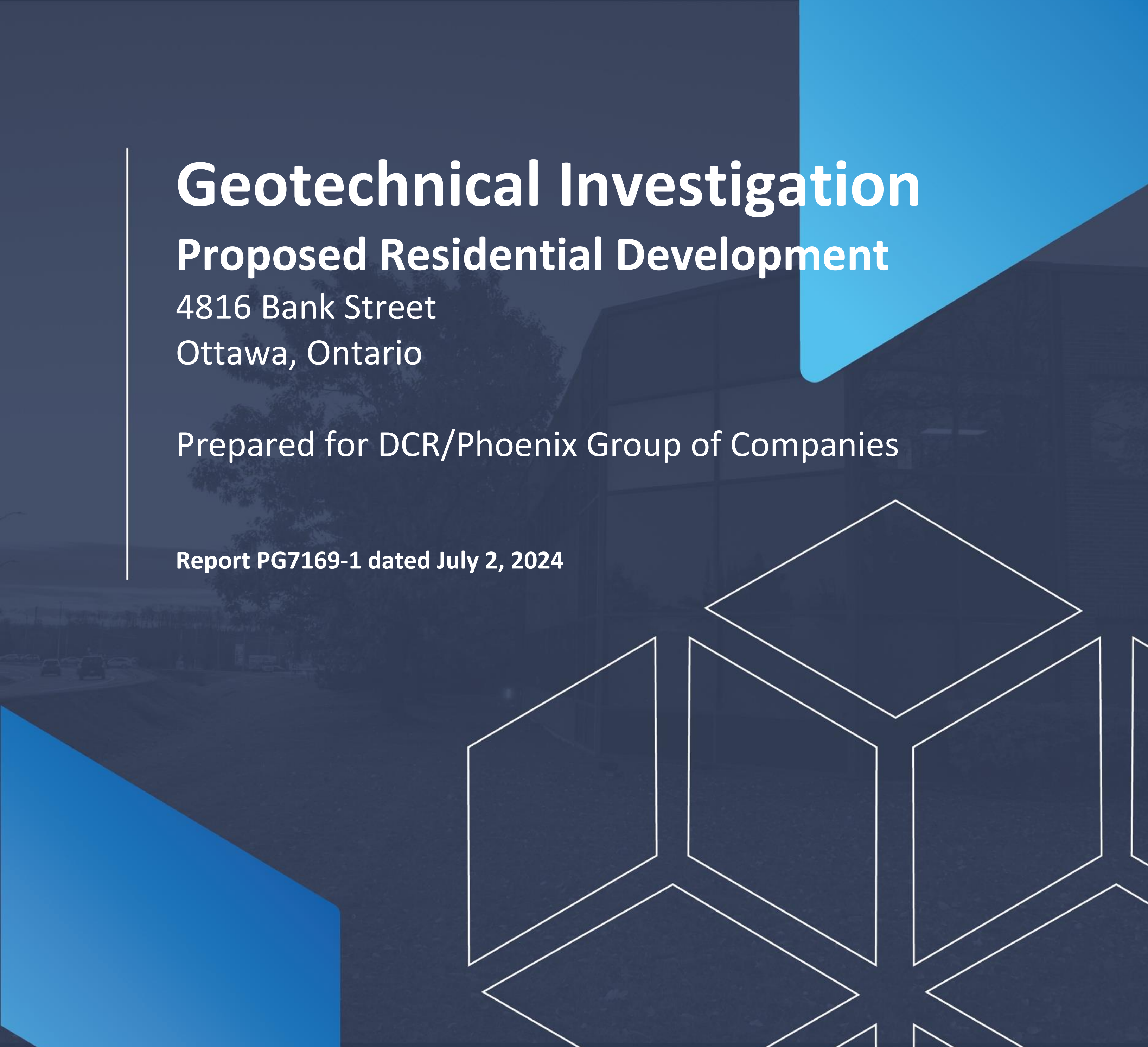

Geotechnical Investigation

Proposed Residential Development

4816 Bank Street
Ottawa, Ontario

Prepared for DCR/Phoenix Group of Companies

Report PG7169-1 dated July 2, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by DCR/Phoenix Group of Companies to conduct a geotechnical investigation for a future residential development to be located at 4816 Bank Street, Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

☐ Determine the subsoil and groundwater conditions at this site by means of test holes.

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

2.0 Proposed Development

Detailed plans were not available at the time of preparing this report. However, it is expected that the proposed development will consist of a series of single and townhouse-style residential dwellings with basements or slab-on-grade construction. Access lanes, at-grade parking areas, associated driveways, local roadways and landscaped areas are also proposed as part of the development.

It is also understood that the proposed development is expected to be municipally serviced.

It is further anticipated that the existing structures will be demolished to accommodate the construction of the proposed developments.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on June 10, 2024, and consisted of advancing a total of three (3) boreholes and seven (7) test pits to maximum depths of 3.3 m and 2.6 m below the existing ground surface, respectively. The test hole locations were distributed in a manner to provide general coverage of the proposed development, taking into consideration existing site features and underground services. The approximate locations of the test pits are shown on Drawing PG7169-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low-clearance, auger drill rig operated by a two-person crew. The test pits were excavated using a backhoe and backfilled with the excavated soil upon completion. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedure of augering/excavating to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In-Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Soil samples were recovered from the sidewalls of the test pits. All soil samples were visually inspected and classified on site. The soil samples were then placed in sealed plastic bags and transported to our laboratory for further review.

The depths at which the grab, split-spoon, and auger samples were recovered from the test holes are shown as G, SS, and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples and are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

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

Groundwater

Borehole BH 3-24 was fitted with a 51 mm diameter PVC groundwater monitoring well. Typical monitoring well construction details are described below:

- ☐ 1.5 m of slotted 51 mm diameter PVC screen at the base of the boreholes.
- ☐ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No. 3 silica sand backfill within annular space around screen.
- ☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

The other boreholes were fitted with flexible piezometers to allow groundwater level monitoring. Additionally, groundwater infiltration into the test pits was also recorded, where encountered. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

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

3.2 Field Survey

The current test hole locations, and ground surface elevation at each test hole location, were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The locations of the test holes, and ground surface elevation at each test hole location, are presented on Drawing PG7169-1 - 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. Moisture content tests were completed for all boreholes. The results are presented in the Soil Profile and Test Data Sheets in Appendix 1.

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. 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 Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by three shed structures, which are located near the central portion of the site. The northern and southern portions of the site are generally observed to be vegetated with mature trees and brushes.

The subject site is bordered to the north by Miikana Road, to the east by Bank Street, to the south by vacant land and to the west by residential buildings. The ground surface across the site is gently slopping downward from the south side to the northeast side at approximate geodetic elevations of 97 m to 95 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil, underlain by silt and glacial till deposit. Fill material was observed at the existing ground surface at borehole BH 1-24 and test pits TP 3-24, TP 4-24 and TP 5-24. The fill was observed to consist of compact brown silty sand with gravel, crushed stone, trace brick, asphalt, clay and organics. A layer of compact brown silt with trace organics, clay and gravel was observed underlying the top and/or fill layer at test pits TP 1-24, TP 2-24, TP 6-24 and TP 7-24 and extended to maximum depths of 0.4 to 0.6 m below existing ground surface. A deposit of compact to dense brown sandy silt layer was observed at test pits TP 4-24, TP 5-24 and TP 6-24 and extended to maximum depths of 0.6 to 0.8 m below the existing ground surface.

At all test hole locations, a glacial till deposit was encountered, consisting of compact to very dense, brown silty sand to sandy silt with gravel, cobbles and boulders and extended to the maximum depth in all test hole locations. Boulders were noted frequently throughout the investigation, and many were considered oversized (i.e.- encountered boulders up to 1 m in diameter).

Refusal to augering or excavation was encountered at all test pit locations, at approximate depths between 1.6 to 3.3 m.

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 hole location.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of sandstone of the Napean Formation and/or interbedded sandstone and dolomite of the March Formation, and is encountered at depths varying between 3 to 10 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring well installed at borehole BH 3-24. Groundwater infiltration into the open test pits was also observed and recorded. The observed groundwater levels are summarized in Table 1 below.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-24	95.80	0.67	95.13	June 28, 2024
BH 2-24	96.97	2.13	94.84	June 28, 2024
BH 3-24	95.90	2.64	93.26	June 14, 2024
TP 1-24	95.81	Dry	Dry	June 10, 2024
TP 2-24	96.58	Dry	Dry	
TP 3-24	95.16	2.60	92.56	
TP 4-24	96.08	Dry	Dry	
TP 5-24	95.77	Dry	Dry	
TP 6-24	95.77	Dry	Dry	
TP 7-24	97.48	Dry	Dry	

Note: The ground surface elevation at each test hole location was surveyed using a GPS referenced to a geodetic datum.

Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately **2.0 to 3.0 m** below the ground surface, and within the bedrock where less than 1.5 m of overburden is present. Groundwater levels are subject to seasonal fluctuations and therefore may vary at the time of construction.

It should be noted that groundwater levels are subject to seasonal fluctuations, therefore, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed residential buildings be founded on conventional spread footings bearing on clean surface sounded bedrock and/or compact to very dense glacial till.

Should existing fill encountered at the underside of footing elevation, it should be sub-excavated to the surface of the clean surface sounded bedrock or undisturbed compact to very dense glacial till. Engineered fill or lean concrete trench should then be placed from the excavated surface up to the underside of footing elevation.

It is anticipated that some bedrock removal and the removal of large boulders will be required for building construction and servicing installation. Therefore, the contractor should be prepared for bedrock removal and the presence of large boulders within the subject site.

Removal of concrete elements is likely to be encountered due to the demolition of the existing structure on site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic or deleterious materials, should be stripped from under the proposed building and other settlement sensitive structures.

If encountered, existing foundation walls and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants, such as foundation walls should be excavated to a minimum of 1 m below final grade.

The existing fill, where free of significant amounts of organics and deleterious materials, may be left in place as subgrade for the basement slab and pavement construction only. If considered suitable by the geotechnical consultant at the time of construction, the fill layer should be proof-rolled by a suitably sized vibratory roller making several passes and approved by Paterson personnel, as noted in Section 5.5 of the present report. Areas with poor performing fill should be removed and reinstated with a compacted engineered fill as detailed below.

Bedrock/Boulder Removal

Bedrock and/or boulder removal may be required at the subject site and can be accomplished by hoe ramming where the bedrock and/or boulders are weathered, and/or where only small quantities need to be removed. Sound bedrock and/or boulders may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations 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 or claims related to the blasting operations.

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

Vibration Considerations

Construction operations are also 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 a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations 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).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. This material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the propose building should be compacted to a minimum 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least 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 composite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

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 300 mm. Where this fill material is open-graded, a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement. This can be assessed at the time of construction.

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. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

5.3 Foundation Design

Bearing Resistance Values

Footings supported directly on clean, surface-sounded bedrock can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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.

Footings supported directly on clean, surface sounded bedrock and design for the bearing resistance values provided above will be subject to negligible post-construction total and differential settlements.

Footings placed on an undisturbed, compact to very dense glacial till can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **400 kPa.**

Footings designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

A disturbed 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.

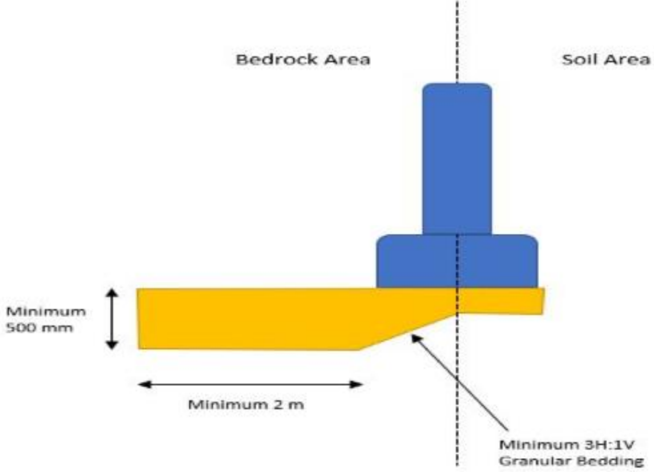
Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. 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. A soil bearing medium or a heavily fractured, weathered bedrock will require a lateral support zone of 1H:1V (or flatter).

Adequate lateral support is provided to the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in-situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended at the soil/bedrock and bedrock/soil transitions that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material, see below. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.



Bedrock/Soil Transition Treatment

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class is required (Class A or B) for the proposed residential buildings, a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the native soil and/or approved fill is considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

For structures with slab-on-grade construction, it is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone.

For structures with basement slabs, it is recommended that the upper 300 mm of sub-floor fill consists of 19 mm clear crush stone.

All backfill material within the footprint of the proposed buildings should be placed in a maximum 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

5.6 Pavement Design

For preliminary design purposes, the following pavement structures, presented in Tables 2 and 3, are recommended for car parking areas and access lanes.

Table 2 – Recommended Asphalt Pavement Structure – driveways

Thickness (mm)	Material Description
50	**Wear Course** – Sup0erpave 12.5 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
300	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Either fill, in situ soils or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Table 3 – Recommended Asphalt Pavement Structure – local roadways	
Thickness (mm)	**Material Description**
40	**Wear Course** – Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – Superpave 19.0 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
450	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Either fill, in situ soils or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

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.

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. The system should consist of a 100 mm to 150 mm diameter perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

If the proposed buildings include below-grade space, 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 geo-composite board, such as Delta Drain 6000, connected to the perimeter foundation drainage system.

If the proposed buildings do not include below-grade space, then backfill against the exterior sides of the foundation wall may consist of on-site excavated fill, provided it is maintained in an unfrozen state and at a suitable moisture content for compaction. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided in this regard.

Exterior unheated foundations, such as isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure, and require additional protection, such as soil cover of 2.1 m, or an equivalent combination of soil cover and foundation insulation.

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, foundation insulation will need to be provided or the frost susceptible bedrock will need to be removed and replaced with lean concrete (minimum 17 MPa 28-day strength).

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. For the proposed development, it is anticipated that sufficient room will be available for the greater pat of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Overburden

The excavation side slopes in the overburdened soils, above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. A flatter slope is required for excavation below groundwater level, such as 3H:1V. The subsurface soil at this site is considered to be mainly Type 2 and 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 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 is used 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.

6.4 Pipe Bedding and Backfill

A minimum 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials 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 site generated fill materials (moist, not wet) above the cover material if excavation and filling operations are carried out in dry and non-freezing weather conditions. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

Well fractured bedrock should be acceptable as backfill provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones 300 mm or larger in their longest dimension are removed. Where blast rock is used a blinding layer (OPSS Granular A crushed stone) or a geotextile may be required above the blast rock to reduce the loss of fine particles within the voids of the rockfill.

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) and above the cover material 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.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into the excavation should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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 Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to

5 months should be allowed for completion of the application 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 Persons as stipulated under O.Reg. 63/16.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the 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 using 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.

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (GU – General Use cement) would be appropriate for this site. The chloride content and pH of the sample indicate that they are not a significant factor in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a low to moderate aggressive corrosive environment.

6.8 Landscaping Considerations

Since the building is not anticipated to be founded on silty clay soils, the City of Ottawa's "*Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines",* which requires specific tree planting setbacks, does not apply for development at the subject site.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

☐ 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 provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 DCR/Phoenix Group of Companies, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Zubaida Al-Moselly, P.Eng.

David J. Gibert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
July 2, 2024
D. J. GILBERT
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PROVINCE OF ONTARIO

Report Distribution:

❑ DCR/Phoenix Group of Companies (email copy)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376200.314 **NORTHING:** 5019532.302 **ELEVATION:** 95.80

DATUM: Geodetic

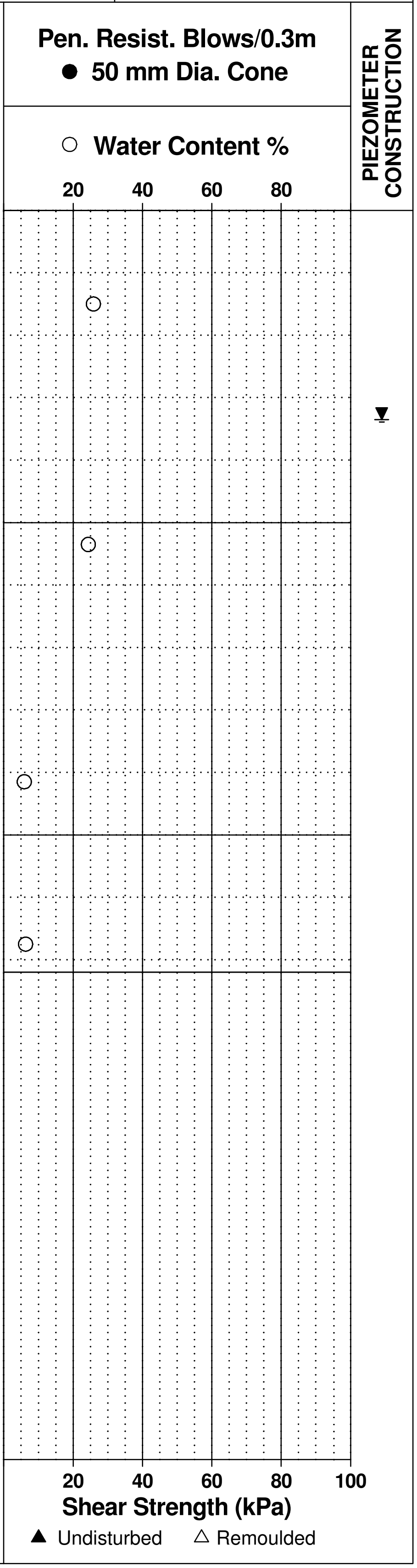
REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 10, 2024

FILE NO. PG7169

HOLE NO. BH 1-24

SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	95.80
TOPSOIL and organics	0.30						
FILL: Compact brown silty sand with crushed stone, trace gravel, topsoil and brick		AU	1				
	0.91						
GLACIAL TILL: Dense brown silty sand to sandy silt with gravel, cobbles and boulders - Boulder content increasing with depth		SS	2	58	4	1	94.80
		SS	3	58	+50	2	93.80
	2.44	SS	4	21	+50		
End of Borehole Practical refusal to augering at 2.44 m depth (GWL at 0.67 m - June 28, 2024)							



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376142.824 **NORTHING:** 5019480.99 **ELEVATION:** 96.97

DATUM: Geodetic

REMARKS:

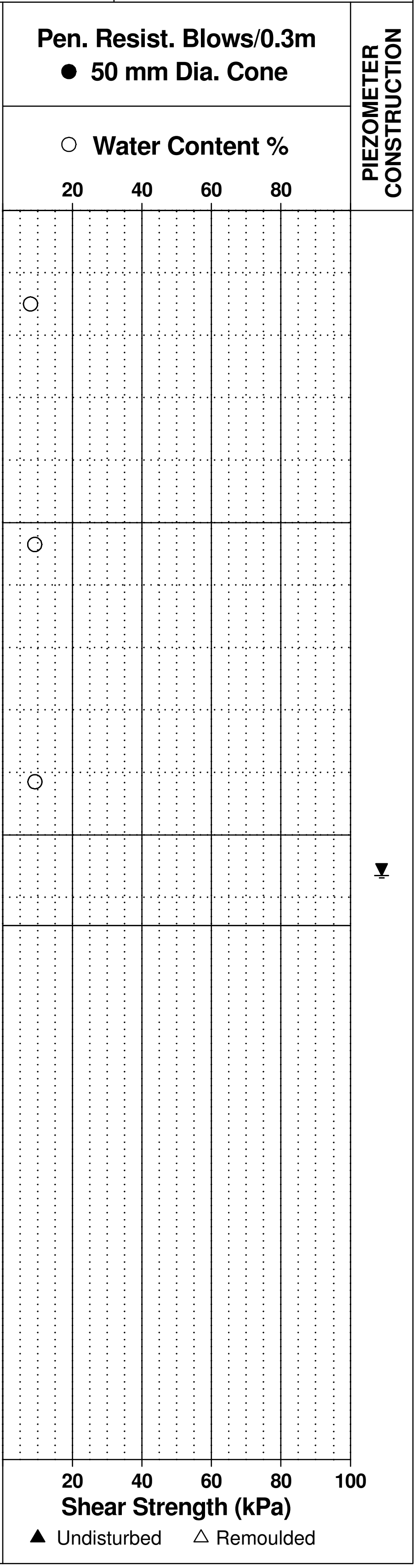
BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 10, 2024

FILE NO. PG7169

HOLE NO. BH 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
Ground Surface					
TOPSOIL and organics					
0.28					
GLACIAL TILL: Compact brown silty sand with gravel		AU	1		
0.61					
GLACIAL TILL: Dense to very dense brown silty sand to sandy silt with gravel, cobbles and boulders		SS	2	75	+50
		SS	3	17	+50
2.39		SS	4	0	+50
End of Borehole Practical refusal to augering on inferred boulder at 2.39 m depth (GWL at 2.13 m - June 28, 2024)					

DEPTH (m)	ELEV. (m)
0	96.97
1	95.97
2	94.97



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376100.246 **NORTHING:** 5019567.717 **ELEVATION:** 95.90

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** June 10, 2024

FILE NO. PG7169

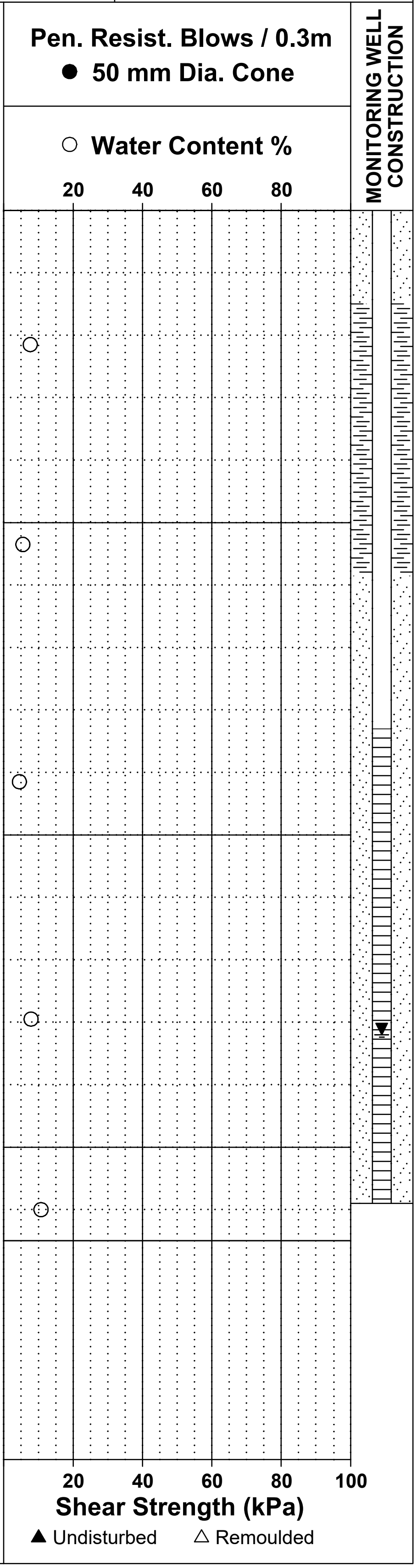
HOLE NO. BH 3-24

SAMPLE DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	95.90
TOPSOIL and organics — 0.25						
GLACIAL TILL: Compact brown silty sand with gravel — 0.69	AU	1				
GLACIAL TILL: Dense to very dense brown silty sand with gravel, cobbles and boulders, some shale fragments - Boulder content increasing with depth	SS	2	75	37	1	94.90
	SS	3	25	+50	2	93.90
	SS	4	4	+50		
3.30	SS	5	25	+50	3	92.90

End of Borehole

Practical refusal to augering at 3.3 m depth

(GWL at 2.64 m - June 14, 2024)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376096.516 **NORTHING:** 5019574.295 **ELEVATION:** 95.81

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 10, 2024

FILE NO. PG7169

HOLE NO. TP 1-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	PIEZOMETER CONSTRUCTION
Ground Surface						0	95.81		
TOPSOIL and organics 0.23		G	1						
Compact brown **SILT** with trace organics, clay and gravel 0.43		G	2						
GLACIAL TILL: Compact to dense brown silty sand with gravel, cobbles and boulders		G	3						
						1	94.81		
		G	4						
						2	93.81		
2.36									
End of Test Pit Practical refusal on cobbles and boulders at 2.36 m depth - Boulders observed to be approximately up to 1 m in its longest dimension (Test Pit dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376119.976 **NORTHING:** 5019545.873 **ELEVATION:** 96.58
DATUM: Geodetic
REMARKS:
BORINGS BY: Backhoe **DATE:** June 10, 2024

FILE NO. PG7169
HOLE NO. TP 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	PIEZOMETER CONSTRUCTION
Ground Surface						0	96.58		
TOPSOIL and organics 0.23		G	1						
Compact brown **SILT** with trace organics, clay and gravel 0.51		G	2						
GLACIAL TILL: Compact to dense brown silty sand with gravel, cobbles and boulders		G	3						
						1	95.58		
1.77		G	4						
End of Test Pit Practical refusal on cobbles and boulders at 1.77 m depth - Boulders observed to be approximately up to 1 m in its longest dimension (Test Pit dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376159.146 **NORTHING:** 5019570.496 **ELEVATION:** 95.16
DATUM: Geodetic
REMARKS:
BORINGS BY: Backhoe **DATE:** June 10, 2024

FILE NO. PG7169
HOLE NO. TP 3-24

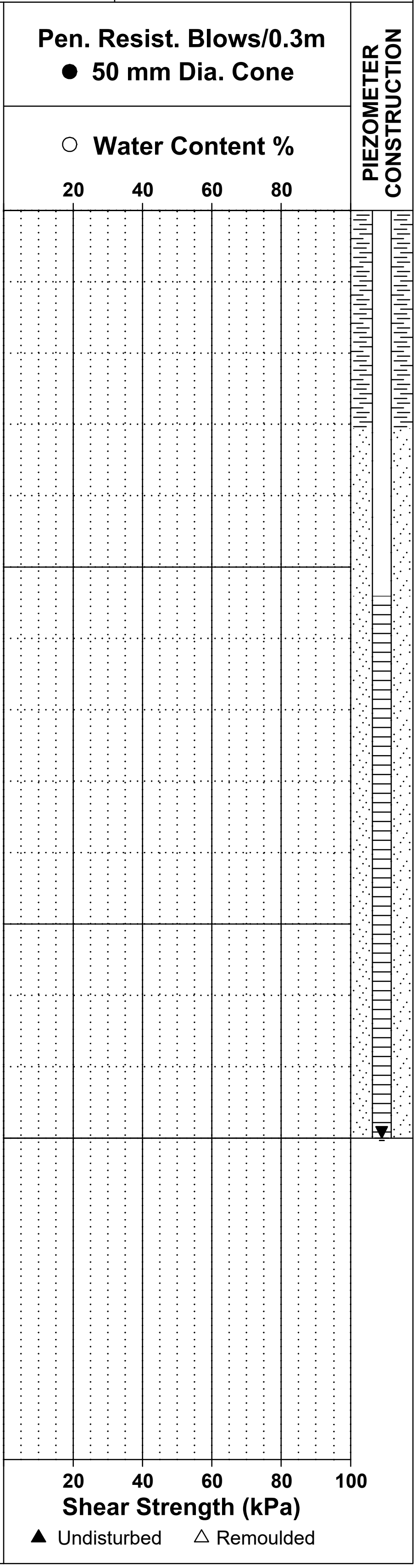
SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	95.16
FILL: Organics with brown silt with gravel — 0.10							
FILL: Compact brown silty sand with gravel, crushed stone, trace brick, asphalt and clay — 0.51		G	1				
TOPSOIL and organics — 0.76		G	2				
GLACIAL TILL: Compact brown silt with clay, some gravel — 1.00		G	3			1	94.16
GLACIAL TILL: Compact to dense brown silty sand with gravel, cobbles and boulders — 2.60		G	4			2	93.16

End of Test Pit

Practical refusal on inferred bedrock at 2.6 m depth

- Boulders observed to be approximately up to 0.8 m in its longest dimension

- Groundwater infiltration observed at 2.6 m depth on June 10, 2024



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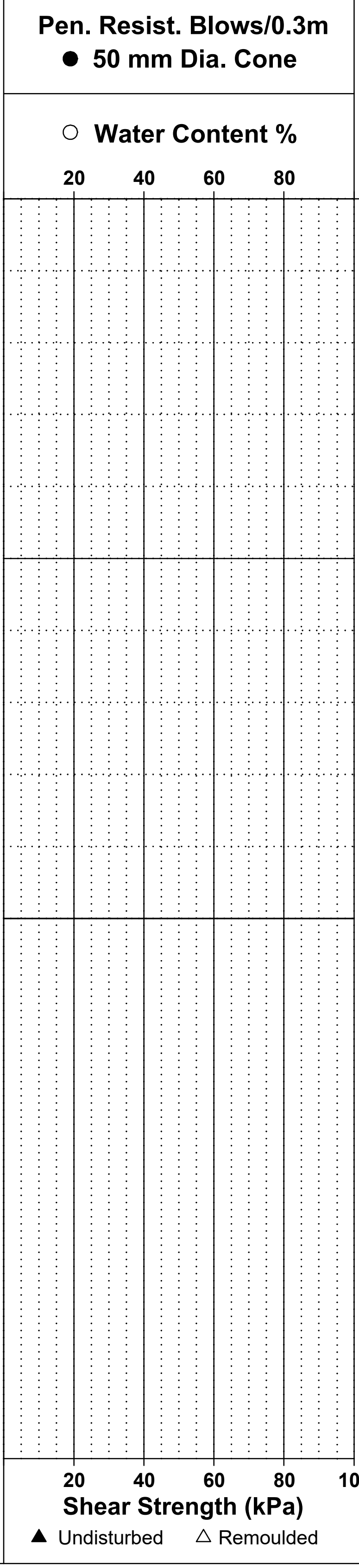
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376205.564 **NORTHING:** 5019518.876 **ELEVATION:** 96.08
DATUM: Geodetic
REMARKS:
BORINGS BY: Backhoe **DATE:** June 10, 2024

FILE NO. PG7169
HOLE NO. TP 4-24

SAMPLE DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	PIEZOMETER CONSTRUCTION
Ground Surface							0	96.08	
FILL: Compact brown silty sand with gravel, crushed stone, trace asphalt and clay	0.40		G	1					
Compact brown **SANDY SILT**, trace clay	0.66		G	2					
GLACIAL TILL: Compact brown silty sand with gravel, cobbles and boulders, trace clay - Clay content decreasing with depth			G	3			1	95.08	
	2.00		G	4			2	94.08	
End of Test Pit Practical refusal on boulders and cobbles at 2.0 m depth - Boulders observed to be approximately up to 1 m in its longest dimension (Test Pit dry upon completion)									



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376040.538 **NORTHING:** 5019593.93 **ELEVATION:** 95.77

DATUM: Geodetic

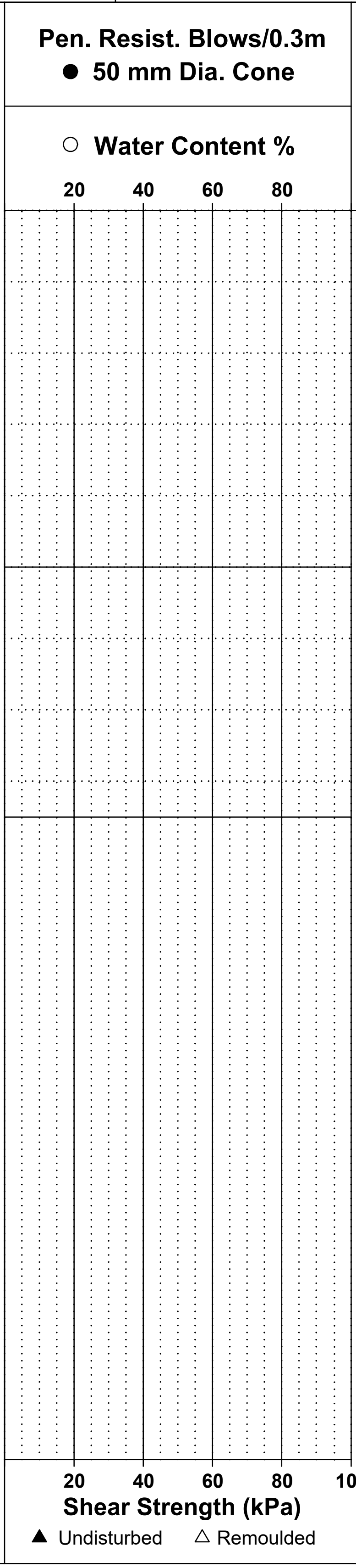
REMARKS:

BORINGS BY: Backhoe **DATE:** June 10, 2024

FILE NO. PG7169

HOLE NO. TP 5-24

SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	95.77
FILL: Organics with sand, gravel and crushed stone	0.20						
FILL: Compact crushed stone with sand	0.30	G	1				
Dense brown **SANDY SILT**, trace clay and gravel	0.63	G	2				
GLACIAL TILL: Compact to dense brown silty sand to sandy silt with gravel, cobbles and boulders		G	3			1	94.77
	1.70	G	4				
End of Test Pit Practical refusal on boulders at 1.7 m depth - Boulders observed to be approximately up to 1 m in its longest dimension (Test Pit dry upon completion)							



PIEZOMETER CONSTRUCTION

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376083.711 **NORTHING:** 5019498.515 **ELEVATION:** 95.77

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 10, 2024

FILE NO. PG7169

HOLE NO. TP 6-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	PIEZOMETER CONSTRUCTION
Ground Surface						0	95.77		
TOPSOIL and organics (to 0.25)		G	1						
Compact brown **SILT**, trace clay (to 0.61)		G	2						
Compact brown **SANDY SILT** (to 0.80)									
GLACIAL TILL: Compact to dense brown silty sand to sandy silt with gravel, cobbles and boulders (to 1.60)		G	3			1	94.77		
		G	4						
End of Test Pit									

End of Test Pit

Practical refusal at 1.6 m depth on boulders

- Boulders observed to be approximately up to 1 m in its longest dimension

(Test Pit dry upon completion)

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
4816 Bank Street
Ottawa, Ontario

EASTING: 376124.333 **NORTHING:** 5019501.728 **ELEVATION:** 97.48

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 10, 2024

FILE NO. PG7169

HOLE NO. TP 7-24

SAMPLE DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	97.48
TOPSOIL and organics	G	1				
0.20						
Compact brown **SILT**, trace clay						
0.45						
GLACIAL TILL: Compact to dense brown silty sand to sandy silt with gravel, cobbles and boulders	G	2				
	G	3				
					1	96.48
	G	4				
1.90						
End of Test Pit Practical refusal on cobbles and boulders at 1.9 m depth (Test Pit dry upon completion)						

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

○ **Water Content %**

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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 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

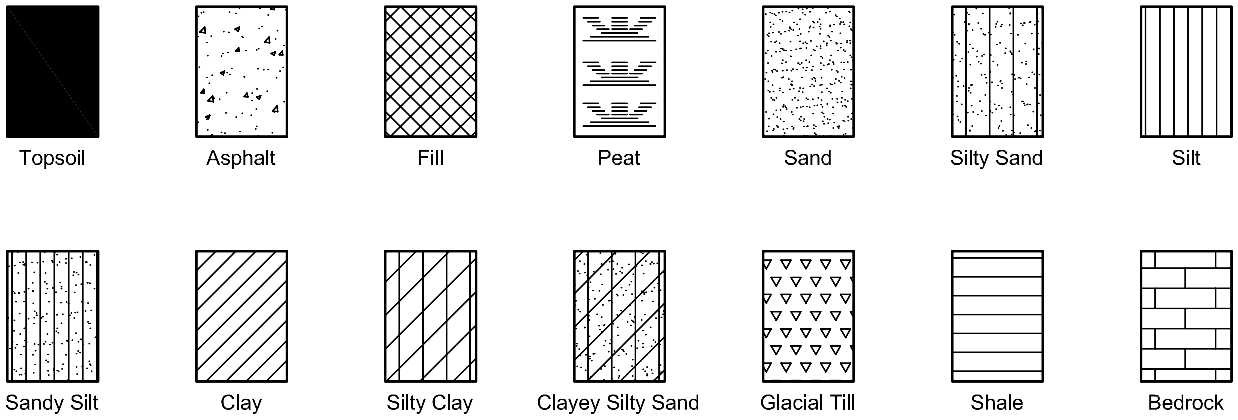
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

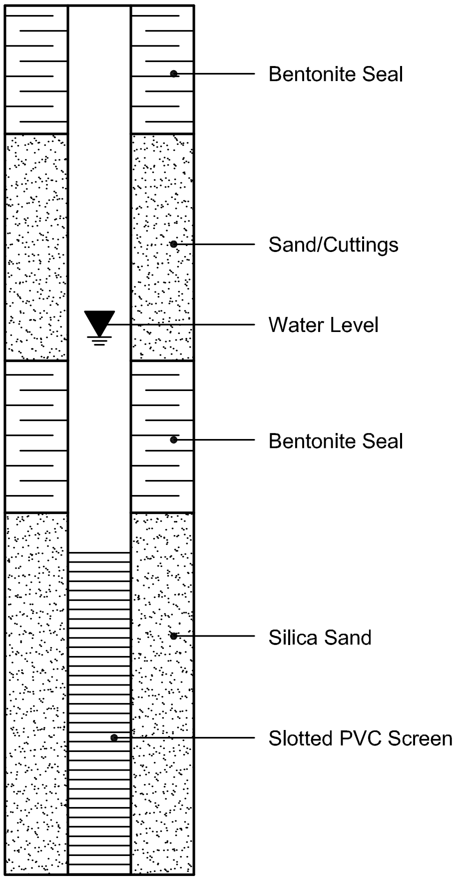
SYMBOLS AND TERMS (continued)

STRATA PLOT

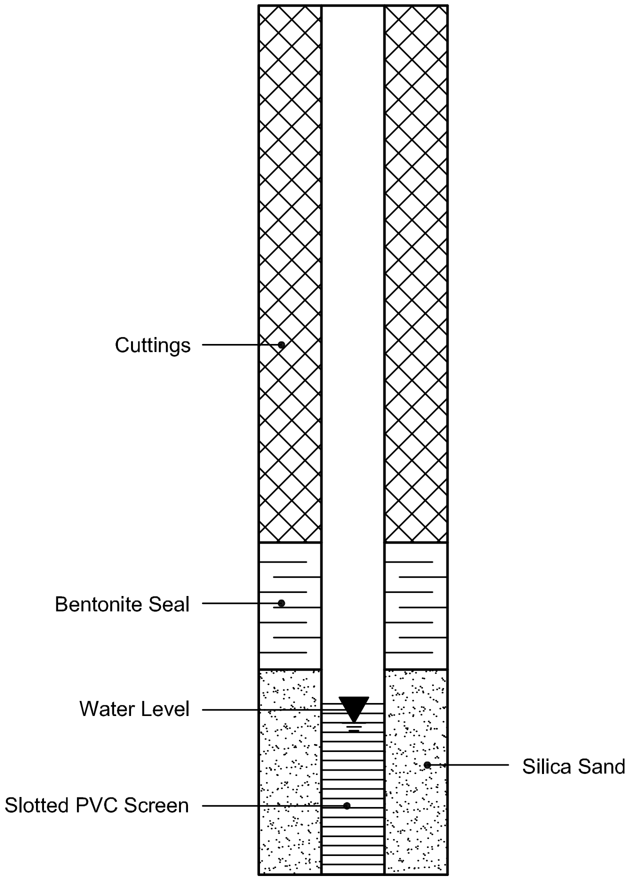


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Client ID:		BH3-24-SS3 (5'-7')	-	-	-	-	-
Sample Date:		10-Jun-24 09:00	-	-	-		
Sample ID:		2424310-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	87.4	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.61	-	-	-	-	-
Resistivity	0.1 Ohm.m	69.0	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	<10	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG7169-1 - TEST HOLE LOCATION PLAN

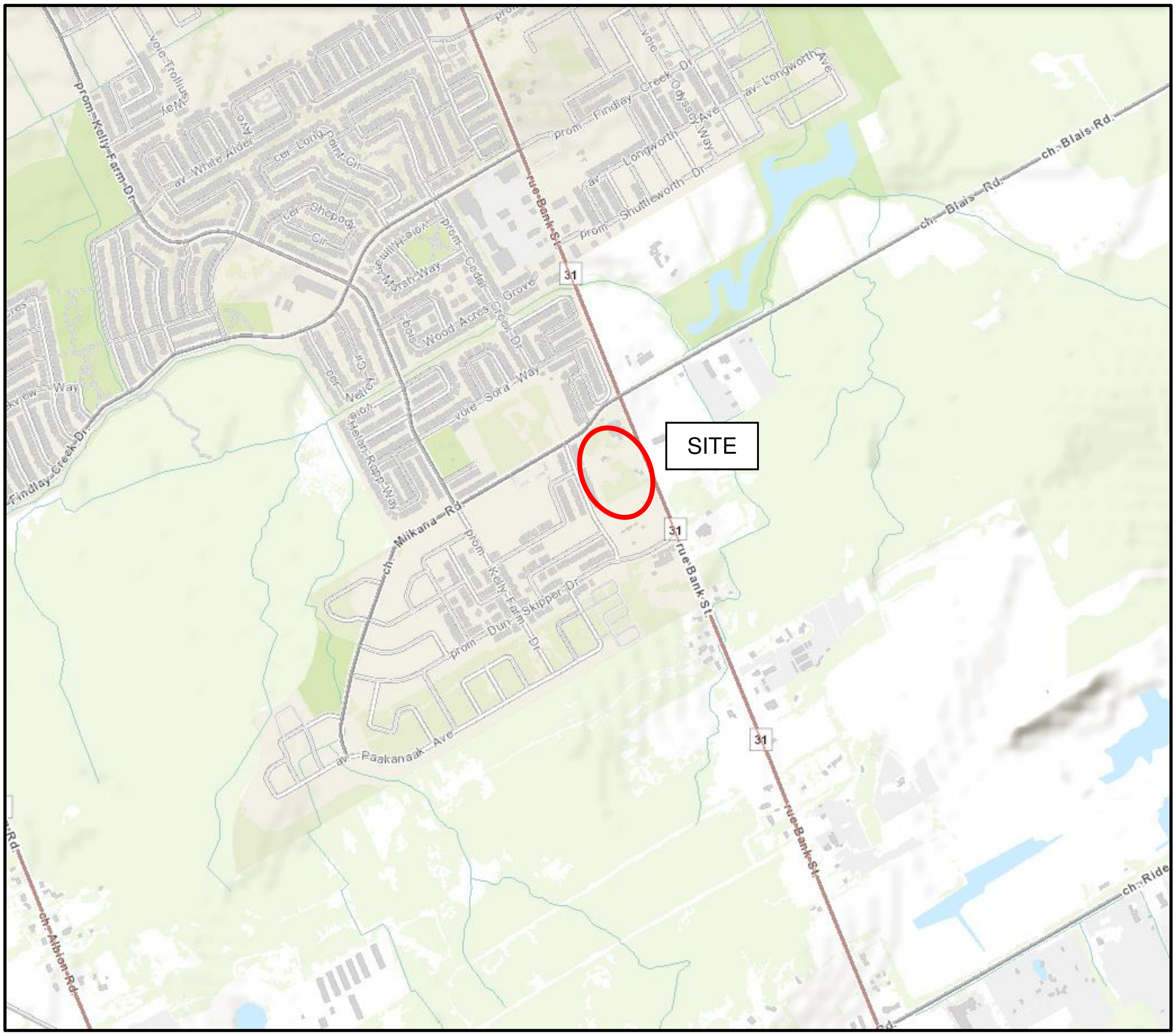
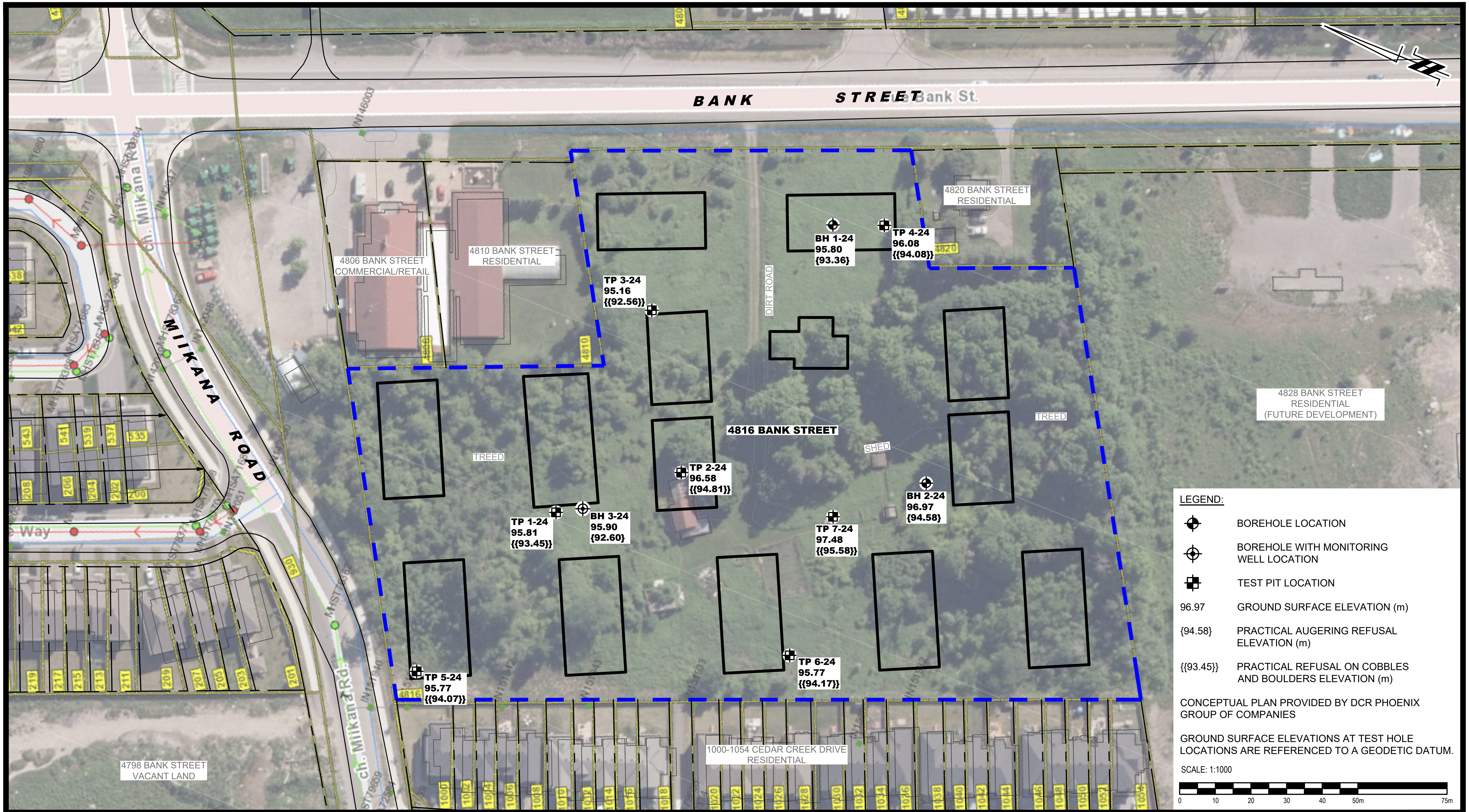


FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⊞	TEST PIT LOCATION
96.97	GROUND SURFACE ELEVATION (m)
{94.58}	PRACTICAL AUGERING REFUSAL ELEVATION (m)
{{93.45}}	PRACTICAL REFUSAL ON COBBLES AND BOULDERS ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY DCR PHOENIX GROUP OF COMPANIES

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

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

NO.	REVISIONS	DATE	INITIAL

DCR PHOENIX GROUP OF COMPANIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4816 BANK STREET

OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 06/2024
Drawn by: YA	Report No.: PG7169-1
Checked by: MR	Dwg. No.: PG7169-1
Approved by: ZAM	Revision No.:

11x17

p:\autocad drawings\geotechnical\pg71xx\pg7169\pg7169-1-test hole location plan.dwg