PATERSON GROUP

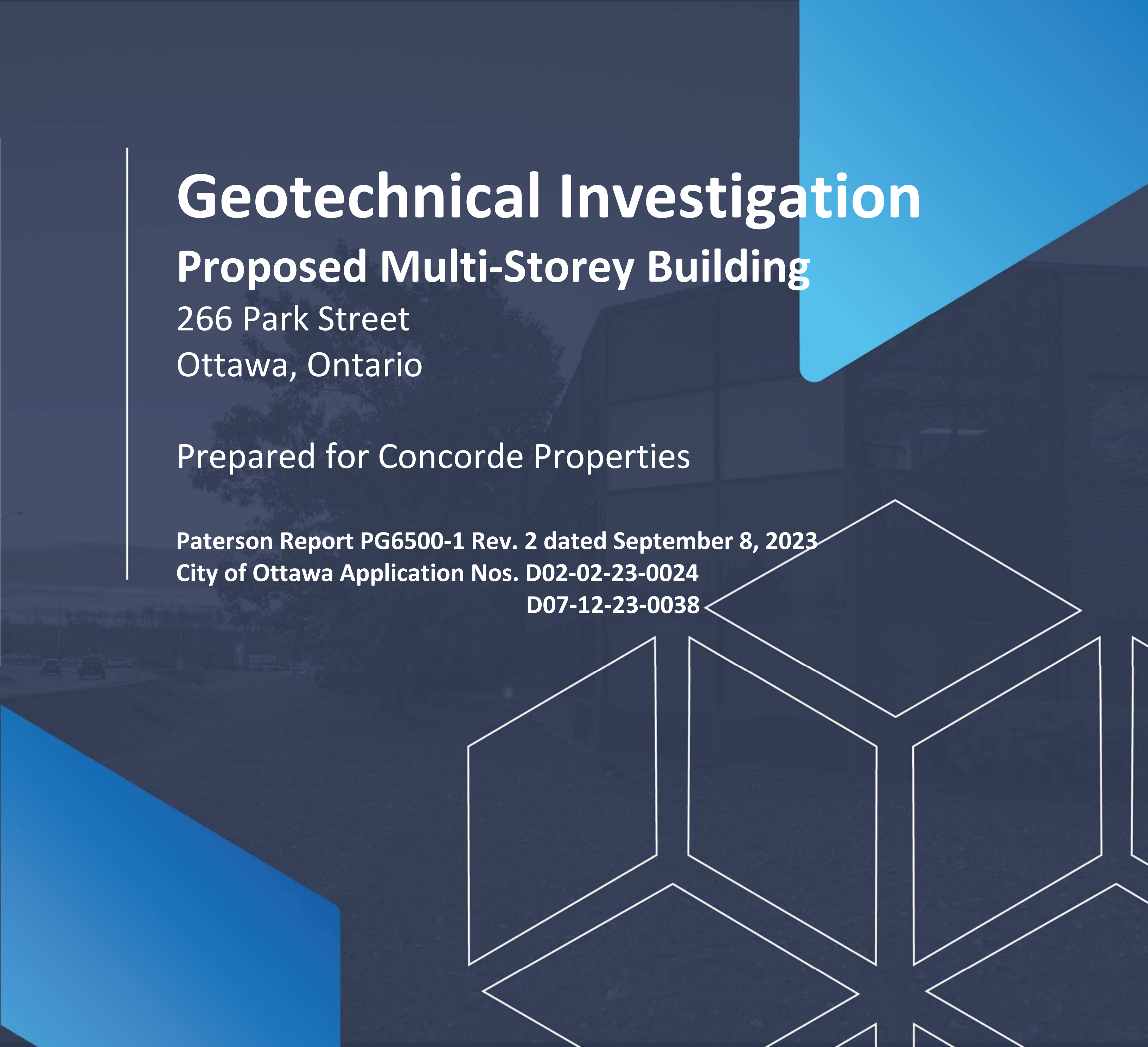
Geotechnical Investigation

Proposed Multi-Storey Building

266 Park Street
Ottawa, Ontario

Prepared for Concorde Properties

Paterson Report PG6500-1 Rev. 2 dated September 8, 2023
City of Ottawa Application Nos. D02-02-23-0024
D07-12-23-0038



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1.0 Introduction

Paterson Group (Paterson) was commissioned by Concorde Properties to prepare a Geotechnical Investigation Report for the proposed multi-storey building to be located at 266 Park Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the investigation were to:

- ☐ Determine the subsurface soil and groundwater conditions by means of boreholes.
- ☐ Provide geotechnical recommendations for 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. The report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the proposed development as understood at the time of this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey residential building with a slab-on-grade. Associated asphalt paved-parking areas and access lanes with landscaped margins are also anticipated surrounding the proposed building.

It is expected that the site will be municipally serviced by water, sanitary and storm services.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was conducted on April 11 and 12, 2022 and consisted of a total of 2 boreholes advanced to a maximum depth of 6.1 m below the existing ground surface. A supplemental investigation was conducted on August 30, 2023, and consisted of one (1) hand auger advanced to a maximum depth of 1.2 m for the purpose of collecting a sample for analytical testing. The test hole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG6500-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed with a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The test hole procedure consisted of augering and rock coring to the required depths at the selected locations, and sampling and testing the overburden and bedrock.

Sampling and In Situ Testing

Soil samples were collected from the test holes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results 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. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

Rock samples were recovered at borehole BH 3-22 using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from this borehole are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole log. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the test holes 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

A groundwater monitoring well was installed in borehole BH 3-22 to permit monitoring of the groundwater levels subsequent to the completion of the field program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG6500-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

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 samples were submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of an asphalt-paved parking lot, and is bordered by Park Street to the east, a residential building to the north, and asphalt-paved parking lots to the west and south. The existing ground surface at the site is relatively level at approximate geodetic elevation 60 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of fill underlain by glacial till and/or bedrock. The fill was generally observed to consist of a loose to compact, brown silty sand with gravel and occasional fragments of brick, wood and plastic, extending to depths ranging from 0.4 to 1.2 m below the existing ground surface.

A glacial till deposit was encountered underlying the fill, and was observed to consist of a dense, brown silty clay with varying amounts of sand, gravel, cobbles, and boulders.

Bedrock

Bedrock was encountered at depths ranging from 1.7 to 2.1 m below existing site grades. Bedrock was cored at borehole BH 3-22 and was observed to consist of a weathered, very poor to fair quality black shale. The bedrock was cored within this borehole to a depth of 6.1 m.

Based on available geological mapping, the bedrock at the subject site consists of shale of the Billings formation.

4.3 Groundwater

The groundwater level was measured in the monitoring well installed in borehole BH 3-22, on April 19, 2022. The observed groundwater level is summarized in Table 1 on the next page.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 3-22*	59.71	2.66	57.05	April 19, 2022

***Note:**
- Borehole elevations are understood to be referenced to a geodetic datum.
- Asterisk(*) denoted a groundwater monitoring well location.

The recorded groundwater levels are also indicated on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

Long-term groundwater levels can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is anticipated at approximate depths of 2.5 to 3.5 m below the existing ground surface.

However, it should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes and are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed building be founded on conventional spread footings bearing on the undisturbed, dense glacial till or weathered shale bedrock.

It is anticipated that bedrock removal may be required for the proposed building construction and/or site servicing installation. Therefore, all contractors should be prepared for bedrock removal within the subject site.

Should fill be encountered at the underside of footing elevation, it should be sub-excavated to the surface of the undisturbed, dense glacial till or weathered shale bedrock and replaced with engineered fill or lean concrete to the proposed founding elevation. The lateral limits of the engineered fill or lean concrete placement should be in accordance with our lateral support recommendations provided herein.

Expansive shale bedrock could be present on site. Precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock. This is discussed further in Section 6.7.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It is anticipated that the existing fill within the future building footprint, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprint outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled several times under dry conditions and above freezing temperatures and approved by Paterson personnel at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Bedrock Removal

Given the very poor to fair quality of the bedrock at the subject site, it is anticipated that bedrock removal can be accomplished by means of a hydraulic excavator and hoe ramming.

Vibration Considerations

Construction operations can be 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 bedrock removal or by other 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). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed building, where required, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material’s 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. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids.

If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 98% of their respective SPMDD.

If site-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 and sampled, reviewed and approved by Paterson prior to use throughout the subject site. The material is generally recommended to be placed in maximum 300 mm thick loose lifts and compacted using a suitably sized vibratory roller. Any site-excavated rock material greater than 300 mm in diameter should be segregated and hoe-rammed into acceptable fragments. Where the fill is open-graded, a blinding layer of finger granular fill or a geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements.

Lean Concrete Filled Trenches

As an alternative to placing engineered fill, where required, consideration should be given to excavating vertical trenches to the undisturbed, dense glacial till or weathered bedrock surface, and backfilling with lean concrete to the founding elevation (minimum 17 MPa 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. 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 glacial till or bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Conventional spread footings bearing on an undisturbed, dense glacial till or weathered shale bedrock bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applies to the bearing resistance value at ULS.

An undisturbed 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.

Footings placed on a soil bearing surface and 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.

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 the undisturbed, dense glacial till or weathered bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1.5H:1V (or flatter) passes through glacial till, weathered bedrock or a material of the same or higher capacity as the glacial till or weathered bedrock, such as concrete.

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), 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 presented 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 revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the existing fill, glacial till, and/or bedrock subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade construction. Where the slab-on-grade subgrade consists of the existing fill, a vibratory drum roller or plate compactor should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as OPSS Granular B Type II.

It is recommended that the upper 200 mm of sub-floor fill consist of OPSS Granular A crushed stone. All backfill materials required to raise grade within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Pavement Structure

Car only parking areas and access lanes are proposed as part of the development at this site. The proposed pavement structures are shown in Tables 2 and 3 below.

Table 2 - Recommended Pavement Structure – Car Only Parking Area

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 – Fill, in-situ soil, bedrock, or OPSS Granular B Type I or II material.	

Table 3 - Recommended Pavement Structure – Access Lanes

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** - HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE – Fill, in-situ soil, bedrock, or OPSS Granular B Type I or II material.	

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 99% of the material’s SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials, such as clean sand or OPSS Granular B Type I granular material. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for placement as backfill against the foundation walls.

However, a review and laboratory testing may be completed at the time of construction to determine if some of the excavated soils are suitable for reuse as backfill around the building foundation walls.

Should the proposed building contain occupied below-grade space, it is recommended that a perimeter foundation drainage system be provided for the below-grade areas. The system, where required, should consist of a 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 a positive outlet, such as a gravity connection to the storm sewer.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the proposed building 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 recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

Exterior unheated footings, 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.

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of excavations in the overburden materials and weathered bedrock 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. It is expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations), however, temporary shoring may be required on the north side of the proposed building due to its property to the northern side boundary.

Unsupported Excavations

The excavation side slopes in the overburden and weathered shale bedrock, above the groundwater level and extending to a maximum depth of 3 m, should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V. The subsoil at this site is considered to be mainly a 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 be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Temporary shoring may be required to support the overburden soil and/or weathered bedrock where insufficient room is available for open cut methods, such as along the northern side of the proposed building.

If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. 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. 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.

The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 4– Soil Parameters for Shoring System Design

Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-rest Earth Pressure Coefficient (K_o)	0.5
Total Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weights are 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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. 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 95% of the SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.5 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 225 mm thick loose lifts and compacted to a minimum of 98% of the material’s SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

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

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) will 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 Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Structures

Based on the subsurface conditions encountered at the subject site, it is anticipated that the adjacent structures are founded on bedrock or the glacial till deposit.

Therefore, no adverse effects from short term and/or long term dewatering are expected for the surrounding structures.

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.

6.7 Protection of Potentially Expansive Shale Bedrock

Upon being exposed to air and moisture, the shale may decompose into thin flakes along the bedding planes. Previous studies have concluded shales containing pyrite are subject to volume changes upon exposure to air. As a result, the formation of jarosite crystals by aerobic bacteria occurs under certain ambient conditions.

It has been determined that the expansion process does not occur or can be retarded when air (i.e. oxygen) is prevented from contact with the shale and/or the ambient temperature is maintained below 20^{o}C, and/or the shale is confined by pressures in excess of 70 kPa. The latter restriction on the heaving process is probably the major reason why damage to structures has, for the greater part, been confined to slabs-on-grade rather than footings.

The presence of expansive shale may be encountered at the subject site. To reduce the long term deterioration of the shale, exposure of the bedrock bearing surface to oxygen should be kept as low as possible. The bedrock bearing surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. A 50 to 75 mm thick concrete mud slab, consisting of minimum 15 MPa lean concrete, should be placed on the exposed bedrock bearing surface within a 48 hour period of being exposed. The excavated sides of the exposed bedrock should be sprayed with a bituminous emulsion or shotcrete to seal bedrock from exposure to air and dewatering.

Preventing the dewatering of the shale bedrock will also prevent the rapid deterioration and expansion of the shale bedrock. This can be accomplished by spraying bituminous emulsion as noted above.

The above-mentioned recommendations for protection of the potentially expansive shale will also mitigate the potential for sulphate attack. The placement of a concrete mud slab will prevent the oxidization of the shale bedrock therefore preventing the formation of sulphates.

6.8 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 (normal cement) would be appropriate for this site. The chloride content and the pH of the samples 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 an aggressive corrosive environment.

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 Paterson:

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

☐ Sampling and testing of the concrete and fill materials.

☐ Observation of the placement of the foundation insulation, if applicable.

☐ 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 and follow-up field density tests to determine the level of compaction achieved.

☐ 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 request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

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

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 Concorde Properties, 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.

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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
257-261 Montreal Road and 266 Park Street
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 12, 2022

FILE NO. PG6500

HOLE NO. BH 3-22

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	59.71
Asphaltic concrete 0.05	AU	1				
FILL: Dark brown to brown silty saned with gravel and crushed stone, trace brick and wood 1.07	SS	2	71	26	1	58.71
GLACIAL TILL: Dense, brown silty clay wtih sand, gravel, cobbles and boulders 1.73	SS	3	60	50+		
BEDROCK: Poor to very poor quality, black shale					2	57.71
	RC	1	91	29		
					3	56.71
	RC	2	92	41		
					4	55.71
					5	54.71
	RC	3	100	20		
6.07					6	53.71
End of Borehole						
(GWL @ 2.66m - April 19, 2022)						

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

Monitoring Well Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
257-261 Montreal Road and 266 Park Street
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 11, 2022

FILE NO. PG6500

HOLE NO. BH 7-22

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	60.16
Asphaltic concrete	0.05	AU	1				
FILL: Dark brown silty sand with gravel and crushed stone	0.36						
GLACIAL TILL: Compact, brown silty sand with gravel, trace clay, occasional cobbles	0.91						
		SS	2	88	28	1	59.16
GLACIAL TILL: Dense, brown silty clay with sand, gravel, cobbles and boulders	2.06	SS	3	93	50+	2	58.16
End of Borehole Practical refusal to augering at 2.06m depth.							

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

Monitoring Well Construction

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

Part of 261 Montreal Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** **NORTHING:** **ELEVATION:**

PROJECT: Proposed Residential Development

BORINGS BY: Hand Auger

REMARKS:

DATE: August 30, 2023

FILE NO. **PG6500**

HOLE NO. HA 1-23

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 25 50 75 100	Peak Shear Strength (kPa) 0 25 50 75 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 25 50 75 100	Piezometer Construction
Ground Surface											
TOPSOIL 0.1 m							0				No Data
FILL: Brown silty sand with gravel							1				
1.2 m		AU1	●								
End of Hand Auger Hole							2				

DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

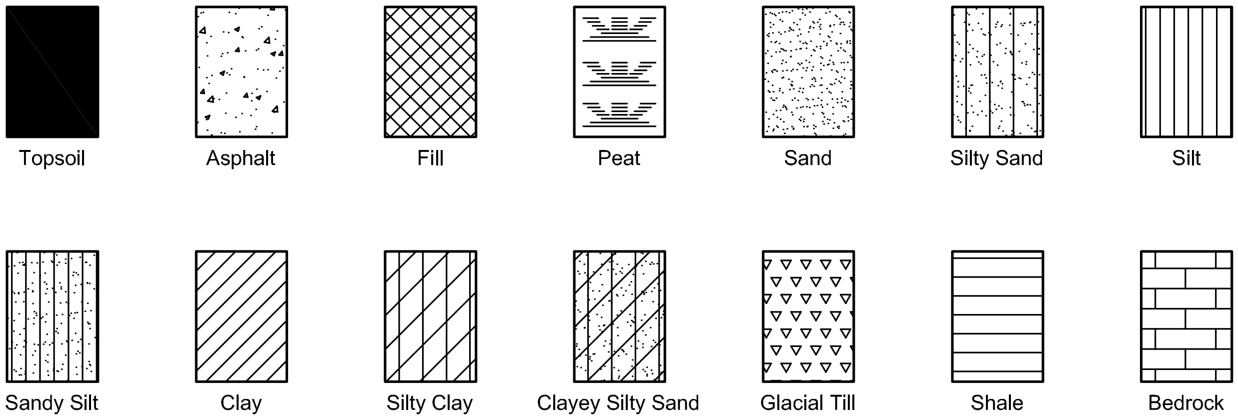
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

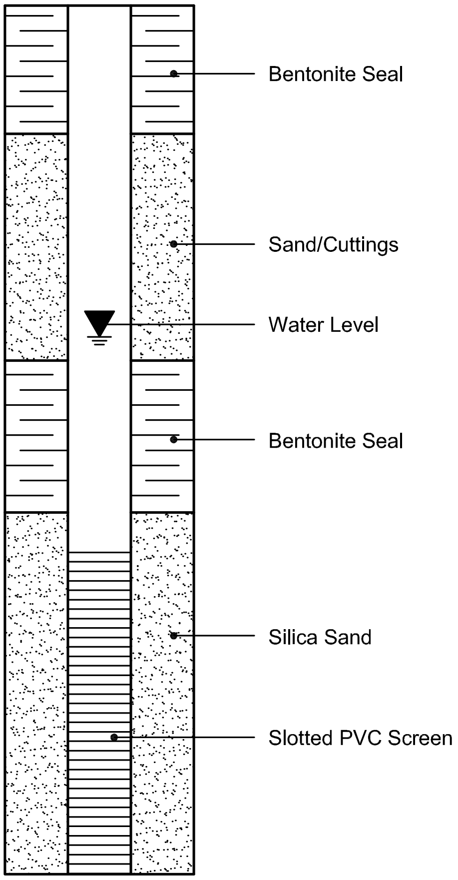
SYMBOLS AND TERMS (continued)

STRATA PLOT

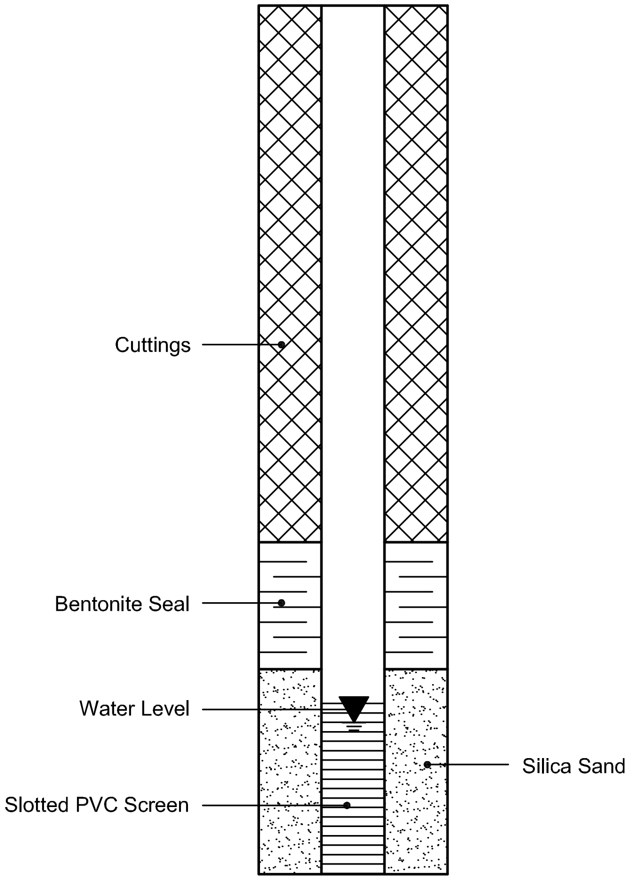


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 07-Sep-2023

Client: **Paterson Group Consulting Engineers**

Order Date: 31-Aug-2023

Client PO: 58273

Project Description: PG6500

Client ID:		HA1-23 1.2m	-	-	-		
Sample Date:		30-Aug-23 09:00	-	-	-	-	-
Sample ID:		2335392-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	92.0	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.67	-	-	-	-	-
Resistivity	0.1 Ohm.m	32.2	-	-	-	-	-
Anions							
Chloride	10 ug/g	35	-	-	-	-	-
Sulphate	10 ug/g	23	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG6500-1 - TEST HOLE LOCATION PLAN

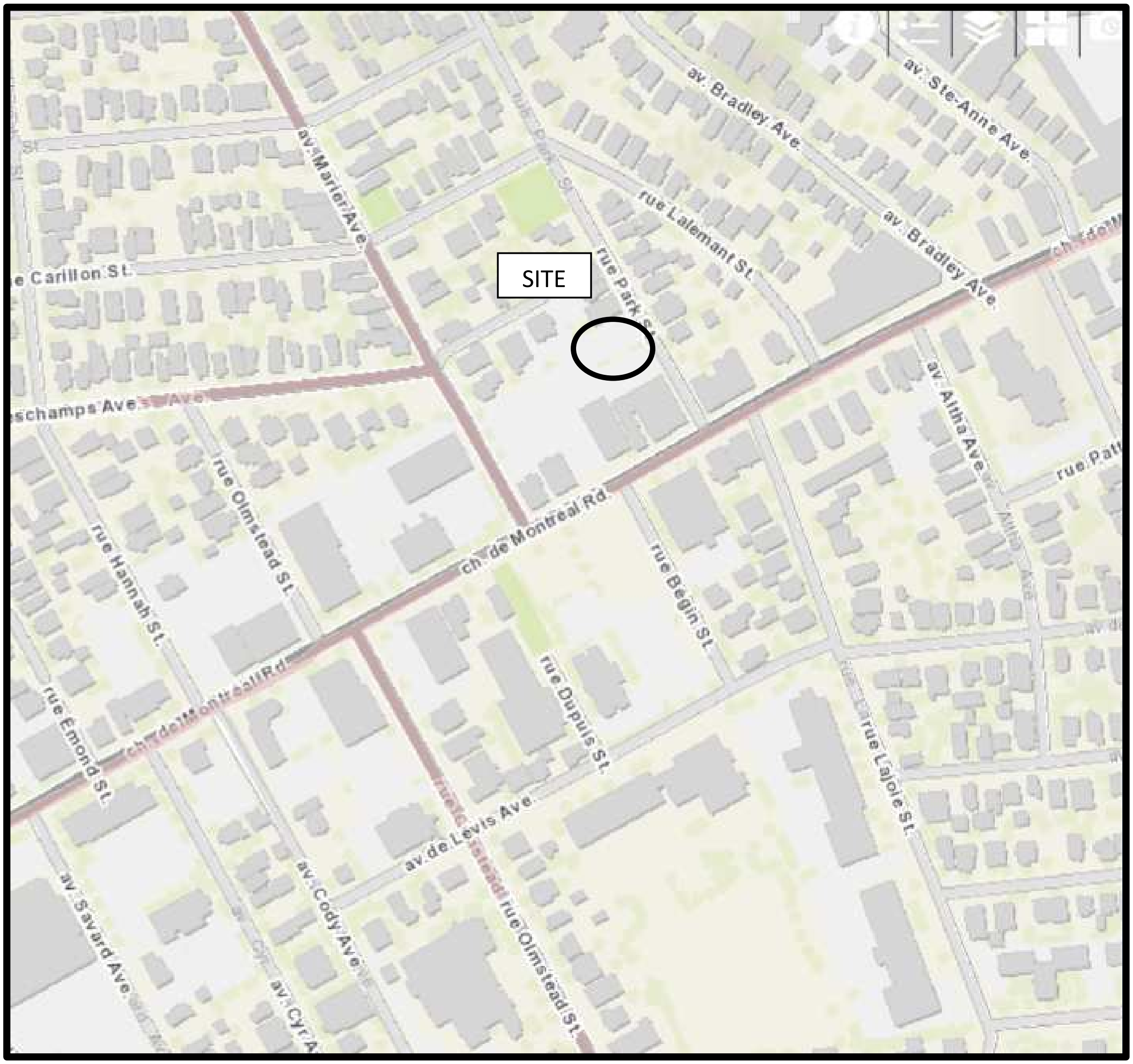
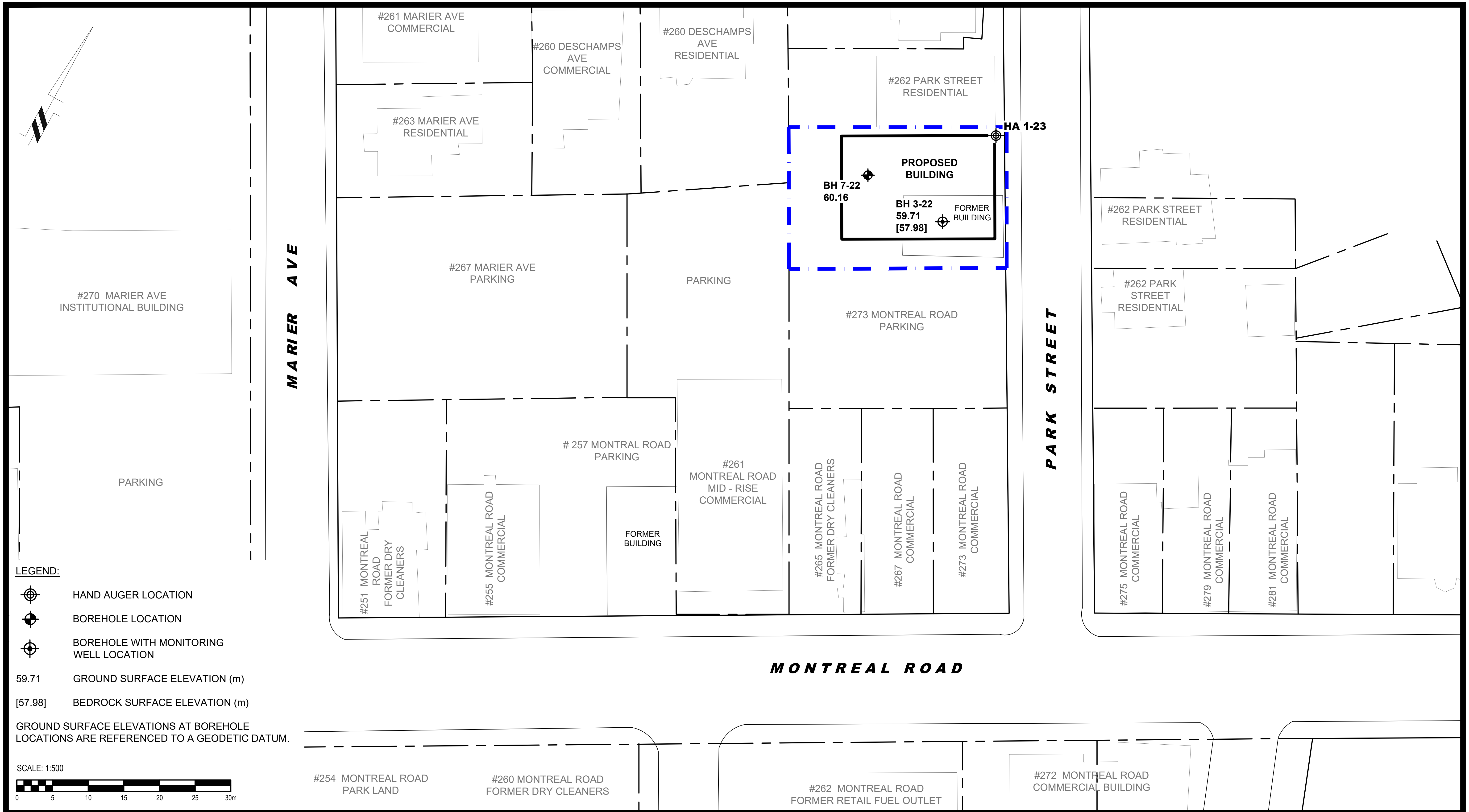


FIGURE 1

KEY PLAN



LEGEND:

⌖	HAND AUGER LOCATION
⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
59.71	GROUND SURFACE ELEVATION (m)
[57.98]	BEDROCK SURFACE ELEVATION (m)

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

SCALE: 1:500

0 5 10 15 20 25 30m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED CLIENT INFORMATIN	07/09/2023	SD

CONCORDE PROPERTIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
266 PARK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION

Scale: 1:500
Drawn by: YA
Checked by: NS
Approved by: SD
Date: 09/2023
Report No.: PG6500-1
Dwg. No.: **PG6500-1**
Revision No.:

11x17

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