PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

240 Presland Road
Ottawa, Ontario

Prepared for CAHDCO

Report PG7188-1 Revision 1 dated April 28, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by CAHDCO to conduct a geotechnical investigation for the proposed development to be located at 240 Presland Road 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 geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site 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. 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.

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed development will consist of a 6-storey residential building which will mostly have a slab-on-grade, but which will also have a service basement located at the north end of the proposed building.

The remainder of the site would be immediately surrounded by access lanes, parking areas, and paved walkways with landscaped margins. It is further understood that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on July 3, 2024, and consisted of a total of 2 boreholes sampled to a maximum depth of 6.8 m below ground surface. The borehole locations were distributed in a manner to provide general coverage of the proposed development, taking into consideration underground utilities and site features. The locations of the boreholes are shown on Drawing PG7188-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

Sampling and In Situ Testing

The borehole samples were recovered from the auger flights using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The 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.

Groundwater

The boreholes were instrumented with standpipe piezometer to allow for groundwater level monitoring. The groundwater observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG7188-1 - 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.

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 sample. 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 vacant, as the apartment building which was previously located on the site has been recently demolished. The site is bordered by Presland Road to the north, and residential properties to the east, west, and south.

The subject site is relatively level at approximate geodetic elevation 60 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of asphalt and fill overlaying a glacial till deposit. The fill encountered at borehole BH 1-24 was noted to consist of loose silty sand with gravel and crushed stone.

The glacial till was encountered at approximate depths of 0.15 to 0.7 m, and was observed to consist of compact to very dense, brown silty sand with shale fragments.

Severely weathered shale bedrock was encountered underlying the glacial till at depths of about 2.1 to 4.5 m. Practical refusal to augering in the shale bedrock was encountered in boreholes BH 1-24 and BH 2-24 at depths of 6.8 m and 6.6 m, respectively, below the existing ground surface.

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

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of Shale of the Carlsbad Formation. The drift thickness at the subject site ranges from 5 m to 10 m.

4.3 Groundwater

Groundwater levels were recorded at each monitoring well location on July 9, 2024. The measured groundwater levels are presented in Table 1 on the next page, and on the applicable Soil Profile and Test Data sheets presented in Appendix 1.

Table 1 - Summary of Groundwater Level Readings				
Test Hole Number	**Ground Surface Elevation (m)**	**Groundwater Level (m)**	**Groundwater Elevation (m)**	**Recording Date**
BH 1-24	59.92	3.66	56.26	July 9, 2024
BH 2-24	60.85	3.22	57.63	July 9, 2024
Note: - Ground surface elevations at borehole locations were surveyed by Paterson and are 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 approximate depths of 3 to 4 m below the existing ground surface.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, 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 considered suitable for the proposed development. It is recommended that the proposed building be founded on conventional spread footings placed on the undisturbed, compact to very dense glacial till or weathered shale bedrock.

Expansive shale bedrock may be encountered during the proposed building excavation at this site. Precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock, which are discussed further in Section 6.8.

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

5.2 Site Grading and Preparation

Stripping Depth

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

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

Bedrock Removal

Bedrock removal, where required, can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming where large quantities of bedrock need to be removed.

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 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 caused by construction operations could also 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 used for grading beneath the building area should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This 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 proposed building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials 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 95% of their respective SPMDD.

Lean Concrete Filled Trenches

Where footings are designed to be supported on the undisturbed, compact to very dense glacial till or clean, surface sounded bedrock which is encountered below the underside of footing elevation, zero-entry vertical trenches should be excavated to the undisturbed, compact to very dense glacial till or clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength).

Typically, the excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of 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 clean, surface sounded 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

Footings placed on the undisturbed, compact to very dense glacial till and/or weathered shale bedrock 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 applied to the above noted bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

Footings bearing on an undisturbed soil or weathered bedrock bearing surface and designed using the bearing resistance values provided 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 glacial till or weathered bedrock bearing media when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Soil/Bedrock Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long-term total and differential settlements.

Also, at the soil/bedrock and bedrock/soil transitions, it is recommended 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. 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.

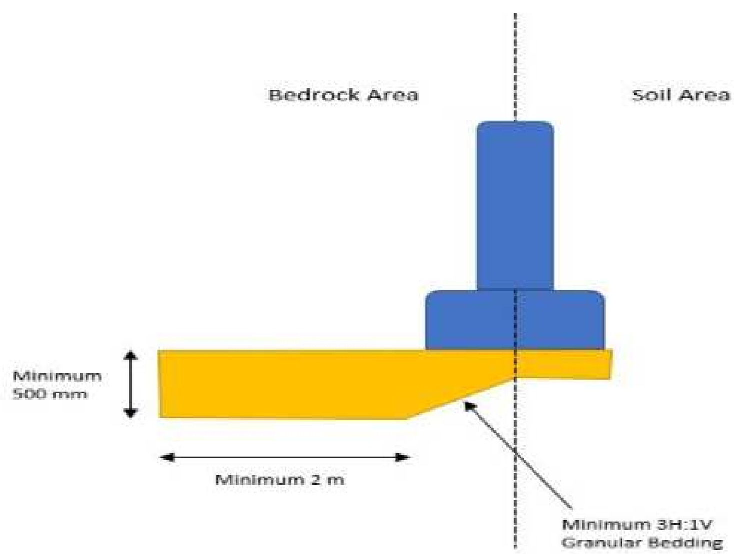

Figure 1 – Soil/ Bedrock Transition Treatment

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class X_C**. If a higher seismic site class is required (such as **Class X_A or X_B**), a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic classification for foundation design of the proposed building, as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2024 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 existing fill, undisturbed glacial till, or weathered bedrock are considered to be acceptable subgrades on which backfill for slab on grade construction or basement slab construction.

However, as noted above in Section 5.2, a vibratory drum roller should complete several passes over the existing fill or undisturbed glacial till subgrades as a proof-rolling program, which should be observed and approved by Paterson. Any poor performing areas in the slab on grade subgrade should be removed and reinstated with an engineered fill such as OPSS Granular A, Granular B Type II with a maximum particle size of 50 mm.

For finished lower basement areas which will require the construction of a concrete floor slab, such as mechanical, electrical or storage rooms, the upper 300 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Elsewhere, where the proposed building will have a slab-on-grade, the upper 200 mm of sub-floor fill should consist of OPSS Granular A crushed stone. All backfill materials within the footprint of the proposed building should be placed in a maximum of 300 mm thick loose layers and compacted to at least 98% of the SPMDD.

For any below-grade space, an underslab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented in the following page.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.357g according to the OBC 2024. Note that the vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

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

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

5.7 Pavement Design

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3 on the next page.

Table 2 - Recommended Pavement Structure – Driveways and Access Lanes

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

Table 3 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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

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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Perimeter Drainage

For the portion of the proposed building with a basement level, a perimeter foundation drainage system is recommended. The system 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 perimeter of the basement. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit.

Underslab Drainage System

Further, for the portion of the proposed building with a basement level, underslab drainage will be required to control water infiltration below the lowest level floor slab. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres underlying the basement floor slab. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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

Sidewalks and Walkways

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

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. Generally, 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

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. It is expected that sufficient room will be available for the most of the excavation to be undertaken by open-cut methods (i.e., unsupported excavations), however, temporary shoring may be required for the portion of the proposed building with a basement, particularly the side nearest to the property line.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. Excavations below the groundwater level should be cut back at a maximum slope of 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

Where sufficient space is not available to slope the excavation, a temporary shoring system would be required to support the excavation. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event 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 designer prior to implementation.

The temporary shoring system may consist of a soldier pipe and lagging system which could be cantilevered, anchored or braced. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

The earth pressure acting on the shoring system may be calculated using the following parameters.

Table 4 – Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight , kN/m^3	21
Submerged Unit Weight , 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 table.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent material specifications and standard detail drawings from the department of public works and services, infrastructure services branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding of private 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 98% of the SPMDD.

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. 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 excavation should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all 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) 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.

Impacts to Neighbouring Properties

It is not anticipated that the proposed building will be founded below the long-term groundwater level. Accordingly, short-term groundwater lowering during construction, and long-term groundwater lowering following construction, is not anticipated. Therefore, adverse effects are not anticipated for neighboring properties as a result of groundwater lowering.

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 severe to aggressive corrosive environment.

6.8 Protection of Potentially Expansive Shale Bedrock

Upon being exposed to air and moisture, 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.

Based on the borehole logs, expansive shale may be encountered within the depth of excavation for the proposed building. To reduce the long term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The weathered bedrock surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. A 50 mm thick concrete mud slab, consisting of minimum 17 MPa lean concrete, should be placed on the exposed bedrock surface within a 48 hour period of being exposed. The excavated sides of the exposed bedrock should be sprayed with a bituminous emulsion to seal bedrock from exposure to air and dewatering.

7.0 Recommendations

It is a requirement for the foundation data provided herein to be applicable that the following materials 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 bituminous concrete including mix design reviews.
- ❑ 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 request, following the completion of a satisfactory material testing and observation program by geotechnical consultant.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management***.

8.0 Statement of Limitations

The recommendations provided herein 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 pit 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 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 CAHDCO 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.

LICENSED PROFESSIONAL ENGINEER
April.28, 2025
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

PATERSON GROUP
9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 240 Presland Road
Ottawa, Ontario

EASTING:	370861.672
NORTHING:	5031739.633
ELEVATION:	59.92
DATUM:	Geodetic
REMARKS:	
BORINGS BY:	CME-55 Low Clearance Drill
DATE:	2024 July 3
FILE NO.	PG7188
HOLE NO.	BH 1-24

SAMPLE DESCRIPTION	Depth to base (m)
GROUND SURFACE	
ASPHALT	0.25
FILL: Loose silty sand with gravel and crushed stone	0.69
GLACIAL TILL: Compact to dense brown silty sand, trace shale fragments - Trace gravel at 1.5 m depth - Shale content increasing with depth	4.50
BEDROCK: Very poor quality, weathered shale bedrock	6.76

End of Borehole

Practical refusal to augering at 6.76 m depth

(GWL at 3.66 m - July 9, 2024)

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	71	25
SS	3	71	38
SS	4	83	31
SS	5	75	+50
SS	6	83	40
SS	7	67	38
SS	8	75	+50
SS	9	75	+50

DEPTH (m)	ELEV. (m)
0	59.92
1	58.92
2	57.92
3	56.92
4	55.92
5	54.92
6	53.92

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

○ Water Content %

20 40 60 80

PIEZOMETER CONSTRUCTION

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 240 Presland Road
Ottawa, Ontario

EASTING: 370863.919 **NORTHING:** 5031780.711 **ELEVATION:** 60.85

DATUM: Geodetic

REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** 2024 July 3

FILE NO. PG7188

HOLE NO. BH 2-24

SAMPLE DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
ASPHALT	0.15				
Loose to compact **SILTY SAND**, trace gravel and cobbles	1.45	AU	1		
		SS	2	50	15
GLACIAL TILL: Compact, silty sand, with shale bedrock	2.10	SS	3	83	29
BEDROCK: Very poor quality, weathered shale bedrock - Trace sand at 2.3 m depth - Shale at 3.8 m depth	6.63	SS	4	75	37
		SS	5	75	45
		SS	6	17	+50
		SS	7	67	26
		SS	8	92	45
		SS	9	62	+50

End of Borehole

Practical refusal to augering at 6.63 m depth

(GWL at 3.22 m - July 9, 2024)

DEPTH (m)	ELEV. (m)
0	60.85
1	59.85
2	58.85
3	57.85
4	56.85
5	55.85
6	54.85

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

○ Water Content %

20 40 60 80

PIEZOMETER CONSTRUCTION

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

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.

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

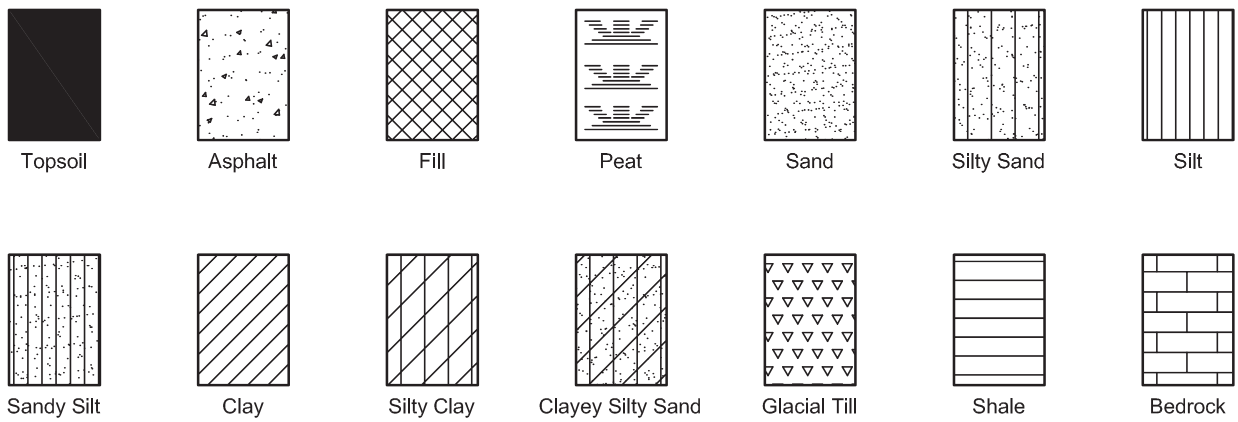
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

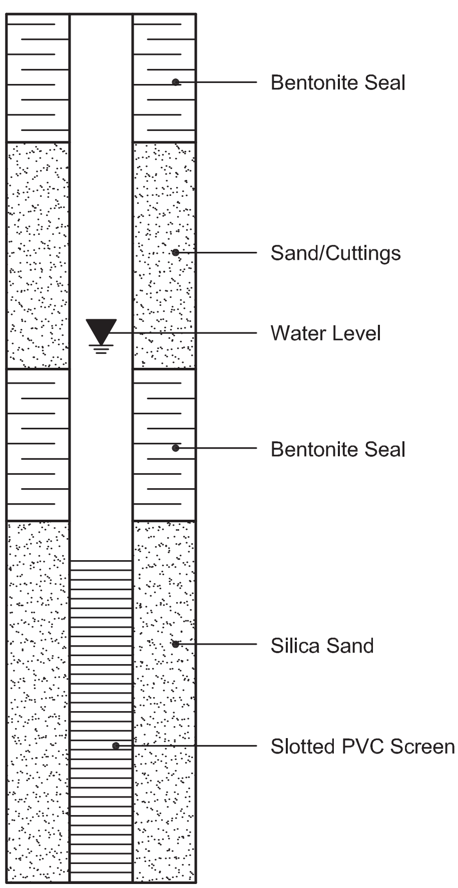
SYMBOLS AND TERMS (continued)

STRATA PLOT

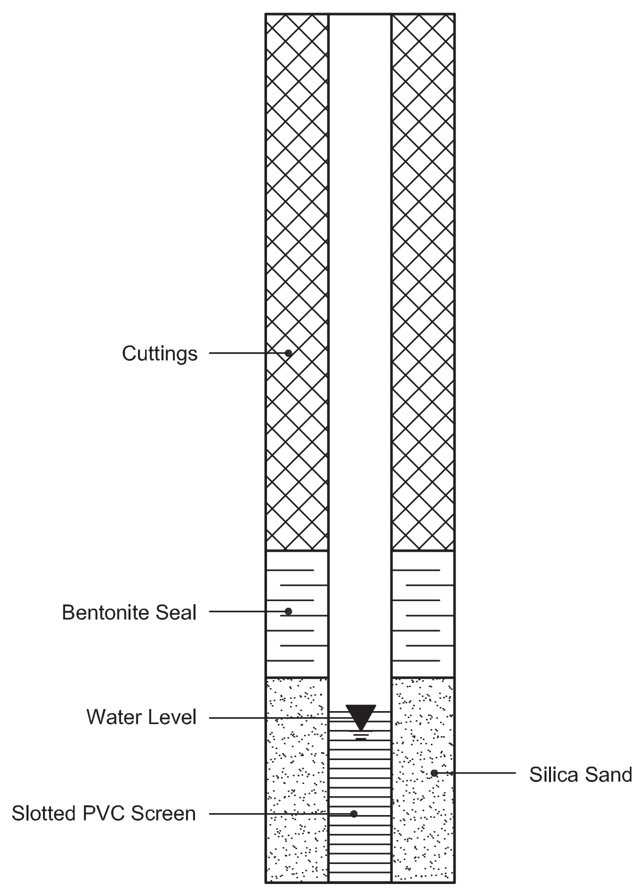


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL
LABORATORIES LTD.

Order #: 2427437

Certificate of Analysis

Report Date: 11-Jul-2024

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

Order Date: 5-Jul-2024

Client PO: 60594

Project Description: PG7188

	Client ID:	BH1-24 SS5	-	-	-	-	-
	Sample Date:	03-Jul-24 09:00	-	-	-		
	Sample ID:	2427437-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	94.8	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	8.22	-	-	-	-	-
Resistivity	0.1 Ohm.m	11.7	-	-	-	-	-
Anions							
Chloride	10 ug/g	171	-	-	-	-	-
Sulphate	10 ug/g	623	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG7188-1 - TEST HOLE LOCATION PLAN

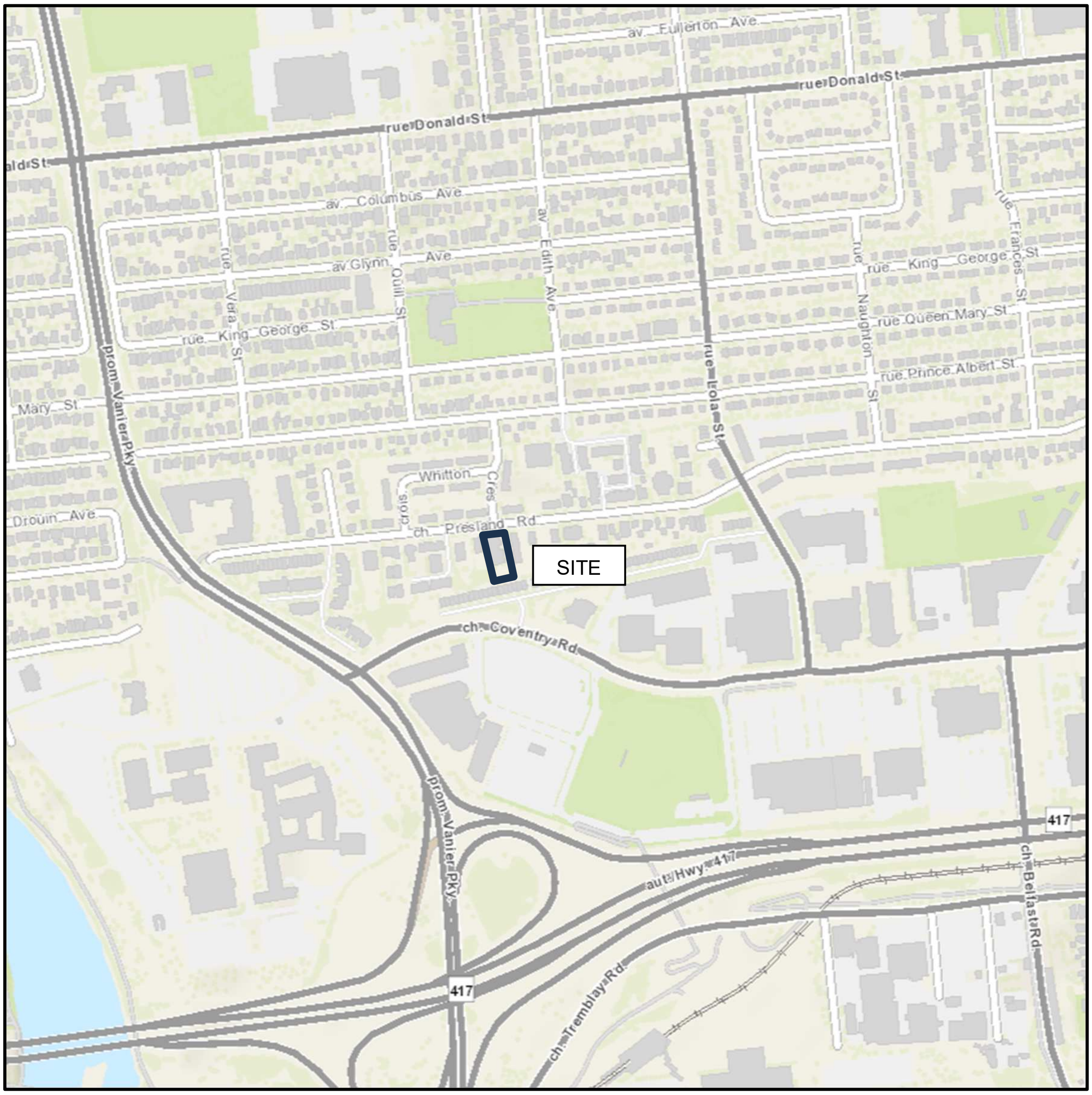
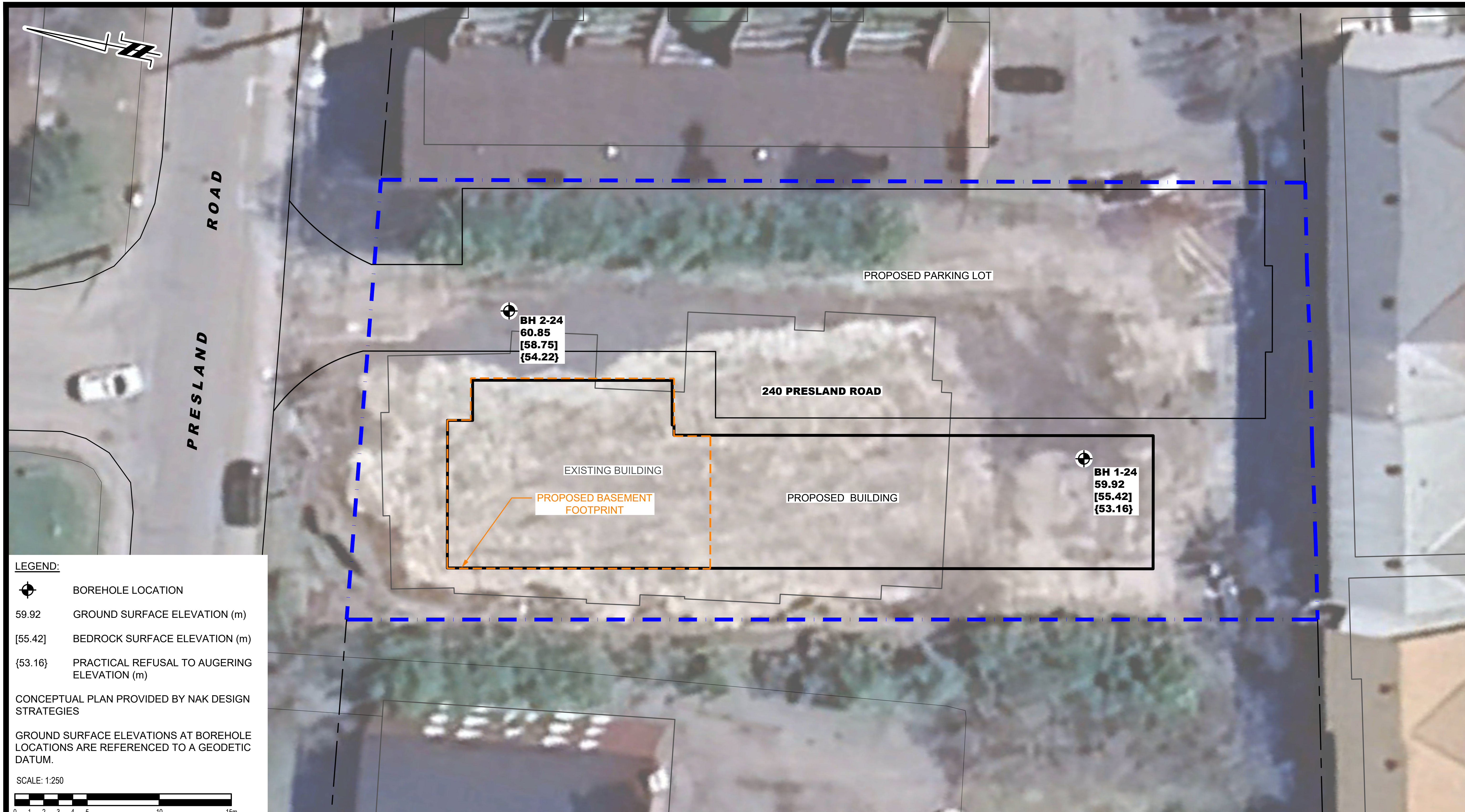


FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

(symbol)	BOREHOLE LOCATION
59.92	GROUND SURFACE ELEVATION (m)
[55.42]	BEDROCK SURFACE ELEVATION (m)
{53.16}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NAK DESIGN STRATEGIES

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

SCALE: 1:250

0 1 2 3 4 5 10 15m

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NO.	REVISIONS	DD/MM/YYYY	INITIAL
1	UPDATED TO NEW CONCEPTUAL PLAN	28/04/2025	SD

CAHDCO
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
240 PRESLAND ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:250	Date:	07/2024
Drawn by:	GK	Report No.:	PG7188-1
Checked by:	KS	Dwg. No.:	PG7188-1
Approved by:	SD	Revision No.:	1

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11x17