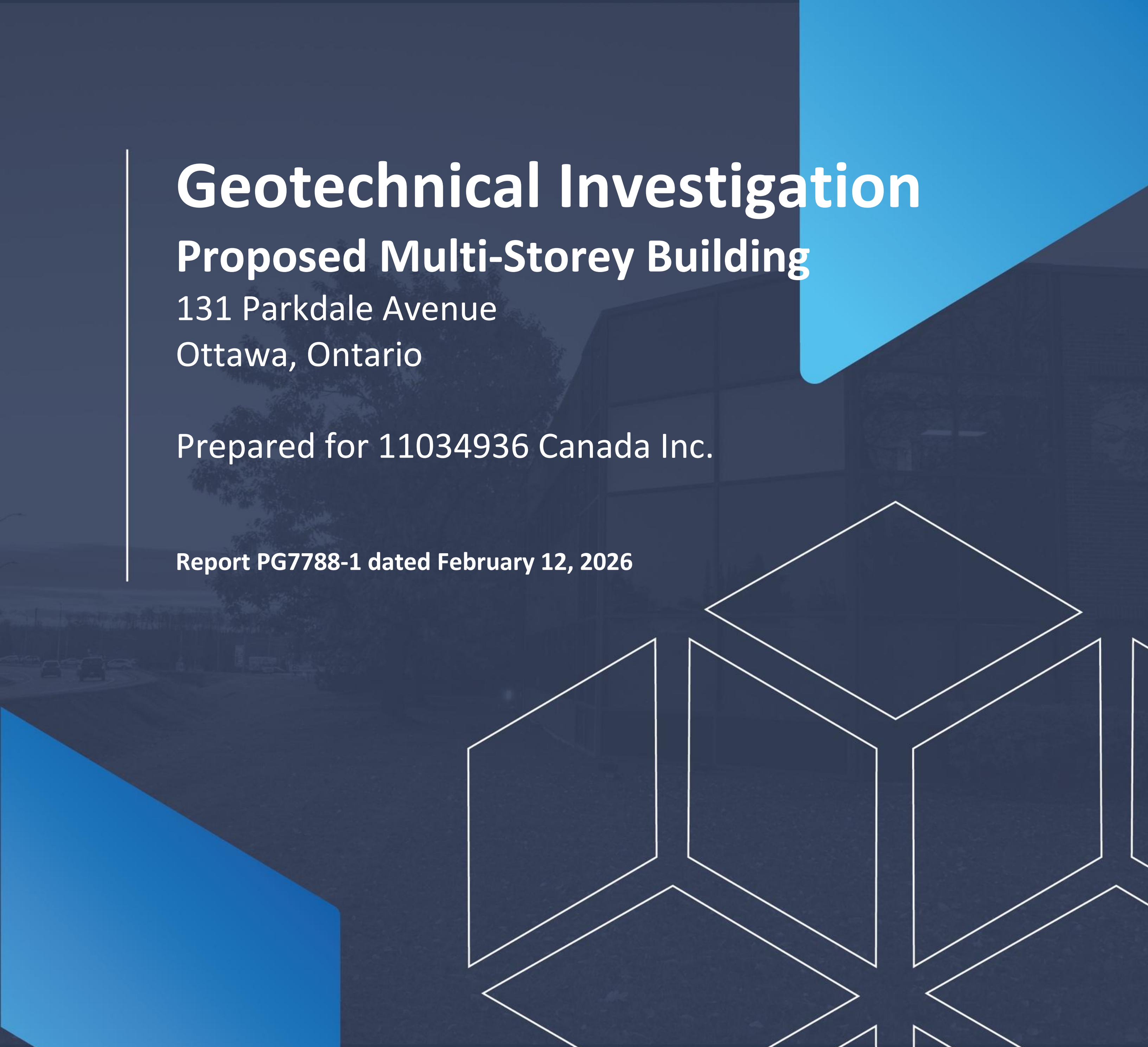


Geotechnical Investigation

Proposed Multi-Storey Building

131 Parkdale Avenue
Ottawa, Ontario

Prepared for 11034936 Canada Inc.

Report PG7788-1 dated February 12, 2026

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 11034936 Canada Inc. to conduct a geotechnical investigation for the proposed mixed-use development, to be located at 131 Parkdale Avenue, 139 Parkdale Avenue, and 122 Forward Avenue in the City of Ottawa, Ontario (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 pertaining to the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

Detailed plans were not available at the time of preparing this report. However, it is understood that the proposed development will consist of a 40-storey high-rise apartment building with up to 4 levels (potentially 5 levels) of underground parking level. The building footprint is anticipated to occupy the majority of the subject site and will be connected to a 6-storey podium. At-grade landscaping will primarily consist of streetscape-related works. An amenity terrace is anticipated on the roof of the podium, with a potential additional amenity terrace at the top of the tower.

Associated at-grade access lanes, pedestrian pathways, and landscaped areas are also anticipated. The proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was conducted from December 1 to 2, 2025. The investigation consisted of a total of four (4) boreholes (BH 1-25 to BH 4-25) advanced to a maximum depth of 11.8 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the test holes are shown on Drawing PG7788-1 - Test Hole Location Plan included in Appendix 2.

In addition, a phase II environmental site assessment was conducted by others on December 3, 2025. At that time, a total of seven (7) boreholes (BH 5-25 to BH 11-25) were advanced to a maximum depth of 0.76 m below existing grade at the aforementioned site. The locations of the test holes are shown on Drawing PG7788-1- Test Hole Location Plan included in Appendix 2.

The borehole was completed using a low-clearance 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 from the geotechnical division. The drilling procedure consisted of drilling to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

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

Rock samples were recovered using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory.

The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality. The subsurface conditions observed at the test pits were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

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

Groundwater

Groundwater monitoring wells were installed in BH 1-25, BH 2 -25, BH 3 -25, and BH 4-25 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1. Typical monitoring well construction details are described below:

Monitoring Well Installation

Typical monitoring well construction details are described below:

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

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

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 ground surface elevation at each test hole location are presented on Drawing PG7788-1 - Test Hole Location Plan in Appendix 2.

Sample Storage

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

3.3 Laboratory Review

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Moisture contents were also determined on split samples collected from the boreholes. The results of the moisture content testing have been included in the Soil Profile and Test Data Sheets enclosed in Appendix I.

4.0 Observations

4.1 Surface Conditions

The subject site is located at 131 Parkdale Avenue, 139 Parkdale Avenue, and 122 Forward Avenue in the City of Ottawa. The site is currently occupied by existing low-rise residential buildings, associated paved access areas, and limited landscaped/grass-covered areas.

The site is bordered to the west by Parkdale Avenue, to the east by Forward Avenue, to the north by Burnside Avenue and adjacent residential dwellings, and to the south by residential properties. The existing ground surface across the site is generally at grade relative to the surrounding streets, with a moderate slope descending from the eastern portion of the site (Forward Avenue) toward the western portion (Parkdale Avenue).

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of asphaltic concrete underlain by fill. The fill was generally observed to consist of brown silty sand with crushed stone, rock fragments, brick, trace to some clay, and organics. The fill layer thickness ranges between approximately 0.5 to 1.3 m depth.

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

Bedrock

Bedrock was encountered underlying the fill at approximate geodetic elevations of 59.5 m to 61.1 m. Based on the recovered bedrock cores, the bedrock consists of grey limestone. Further, based on the RQD values of the recovered bedrock cores, the bedrock varies from poor to fair in quality in the upper 2 m, generally becoming excellent in quality below these depths.

It is expected that existing services might have been installed in trenches extending into the rock.

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic limestone of the Bobcaygeon formation.

4.3 Groundwater

Groundwater levels were recorded at each borehole location instrumented with a monitoring device. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. The measured water levels within the monitoring wells are presented in Table 1 below:

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-25*	60.84	4.80	56.04	December 17, 2025
BH 2-25*	60.64	3.35	57.29	
BH 3-25*	61.60	5.00	56.60	
BH 4-25*	61.59	3.35	58.24	

Note: Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.
*- Monitoring Well

It should be noted that surface water can be trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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 suitable for the proposed development. The proposed building is recommended to be founded on conventional spread footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the underground level. Hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

Based on the bedrock quality the upper portion of the rock excavation will require stabilization measures to ensure safety of the workers.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the depth of the bedrock at the subject site and the anticipated founding level for the proposed multi-storey building, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed multi-storey building.

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

Bedrock Removal

As noted above, bedrock removal 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.

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 proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

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

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause or the source of detrimental vibrations at the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity (PPV) and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline for structures and pipelines, as per S.P. No: F-1201, the peak particle velocity should be less than 20 mm/s for frequencies less than 40 Hz and 50 mm/s for frequencies above 40 Hz.

These guidelines are at perceivable human level and, in some cases, could be very disturbing to some people. A pre-blast survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery.

The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Horizontal Rock Anchors

Horizontal rock anchors may be required at specific locations to prevent pop-outs and slippage of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations.

Bedrock Stabilization

A weathered bedrock layer was noted within the upper 2 m of the bedrock on site. It is therefore expected that bedrock stabilization and protection measures will be required on site. Those measures should consist of an anchored chainlink fencing and geotextile placed on the exposed excavation face to ensure that small rock fragments do to not fall onto workers in close proximity of the excavation walls.

Paterson will need to complete a full review of the excavation face during rock removal to review the stabilization requirements. At this point given the known information about the project, the following recommendations are made for budgetary purposes. Final field requirements will depend on the exact bedrock conditions encountered during construction.

Bedrock grinding is expected for this site to allow installation of the proposed drainage and waterproofing system. Once completed a review of the upper portion of the bedrock layer should be completed by Paterson to establish the stabilization measures required.

Bedrock stabilization shall consist of chain link fencing with a woven geotextile liner, such as Terratrack 200 or equivalent, placed over the upper weathered bedrock in order to stabilize any loose rock present. The top of the chain link fencing should be secured by a series of bolts extended into the top of the bedrock bench. The weathered layer of rock was noted to range from 1 to 3 m in depth. The chain link fencing should be anchored below the weathered layer in good-quality rock using a series of minimum 150 mm length rock wedge bolts spaced no greater than 1.5 m apart.

Rock bolts consisting of swellex bolts or equivalent might be required to stabilize seams or large blocks of bedrock. The anchor spacing and length should be established on site following the bedrock review. It is expected that seams can be stabilized using 1.8 to 3.0 m long anchor spaced approximately 2.4 m apart. As for block stabilization, anchors should extend a minimum of 1.5 to 2.0 m into good quality rock to retain larger bedrock pieces.

Bedrock cavities and vertical seams

Any significant cavities or vertical seams (wider or deeper than 300 mm) should be in-filled with 30 MPa concrete or shotcrete. Any cavities will need to be cleaned by compressed air or hand tools, and any loose debris and rock should be removed. The rock surface should be clean for the concrete to adhere properly.

Reinforcement bar consisting of a 600 mm long 15M bar should be epoxied a minimum of 300 mm into the rock face at a minimum spacing of 500 mm in a grid pattern. The cavity should then be formed and infilled with a minimum of 30 MPa concrete or shotcrete.

Following the setting of the concrete, install a Swellex rock bolt(s) through the mass of concrete at each cavity and vertical seam location as discussed and marked on site during inspection. The bolt length will depend on the cavity size and can range from 1.8 to 3 m long. The rock bolts should be installed at approximately 25 to 30° below the horizontal. The rock bolts should then be stressed to the manufacturer's specified pressure.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock can be designed using a bearing resistance value at ultimate limits states (ULS) of **5,000 kPa**. A geotechnical factor of 0.5 was applied to the above noted bearing resistance value.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures, or open joints which can be detected from surface sounding with a rock hammer.

Settlement

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

The preliminary site class for seismic site response can be taken as **Class X_C** for foundations bearing directly on the clean, surface-sounded bedrock, although this can be finalized once the project design becomes more advanced. A site-specific shear wave velocity test will be conducted in the spring of 2026 to accurately determine the applicable seismic site classification for the foundation design of the proposed development, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024.

The proposed building foundations bearing directly on clean, surface sounded bedrock will not be subject to liquefaction.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the basement floor slab. It is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone.

In consideration of the groundwater conditions at the site, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the 19 mm clear crushed stone layer under the lower basement floor. This is discussed further in Subsection 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 of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

However, the majority of the basement walls are to be poured against a composite drainage blanket which will be placed over the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 23.5 kN/m^3 (effective 15.5 kN/m^3) where this condition occurs. Further, a seismic earth pressure component will not be applicable for the foundation wall which is poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slab, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective unit weight of the retained soil or bedrock should be utilized, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.465 g according to NBCC. 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 NBCC.

5.7 Pavement Design

Should they be required for the proposed development, the pavement structures presented in the Tables 2 and 3 could be used for the design of car parking areas and access lanes.

Table 2 – Recommended Pavement Structure – Car Only Parking Areas

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

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 – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or concrete fill.	

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

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) could be reduced to 300 mm. The upper 300 mm of the bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised to ensure that the bedrock subgrade does not have depressions that will trap water.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 10 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.

Where insufficient room is available for exterior backfill, it is suggested that the composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the vertical bedrock face extending to a series of drainage sleeves inlets through the building foundation wall at the footing/foundation wall interface. The drainage sleeves should be at least 150 mm diameter and be spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the underslab drainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the lower underground area.

Sub-slab Drainage

Underslab drainage may be required to control water infiltration below the lowest level slab. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at 3 to 6 m centres. 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

Where sufficient space is available for conventional backfilling, backfill against the exterior sides of the foundation wall should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided to the elevator pit (pit bottom and walls).

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 proper 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

Excavations for buildings can be carried out using conventional excavation techniques and in accordance with the Occupational Health and Safety Act, Regulations for Construction projects. The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Where space restrictions exist, temporary shoring of the overburden material will be needed.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Rock Stabilization

Excavation side slopes in sound bedrock can be completed with almost vertical side walls. A minimum of 1 m horizontal ledge should be left between the bottom of the overburden and the top of the bedrock surface to provide an area for potential sloughing.

Horizontal rock anchors may be required at specific locations to stabilize the bedrock excavation face and to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface.

The requirements for horizontal rock anchors and bedrock stabilization measures will be evaluated during the excavation operations and determined by Paterson at the time of construction.

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 for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, should be placed 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.

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

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 225 mm thick loose lifts and compacted to a minimum of 98% of the material’s SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, 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 subgrades, regardless of the source, to prevent disturbance to the founding medium.

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

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.

Long-term Groundwater Control for building use

Any groundwater which breaches the building’s perimeter groundwater infiltration control system will be directed to the sump pit. Provided that the recommendations detailed in subsection 6.1 are successfully implemented and approved by the geotechnical consultant at the time of construction, it is expected that long-term groundwater flow will be very low to negligible (i.e., less than 30,000 L/day).

Impacts on Neighbouring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring 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.

Caution should also be taken to avoid ice formation on vertical rock face. This ice formation can become loose and fall during warmer weather or under impact.

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 (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to very aggressive corrosive environment.

7.0 Recommendations

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

- Review of the as built grading plan, from a geotechnical perspective.
- Review of the contractor's design of the temporary shoring system.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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*.

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 Paterson.

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 11034936 Canada Inc., 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
Feb. 12, 2026
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Zubaida Al-Moselly, Ph.D., P.Eng.

Joey R. Villeneuve, M.A.Sc., P.Eng., ing.

Report Distribution:

❑ 11034936 Canada Inc. (email copy)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

RECORD OF BOREHOLE LOGS BY OTHERS

ANALYTICAL TESTING RESULTS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

131 Parkdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364824.26 **NORTHING:** 5030056.93 **ELEVATION:** 60.84

PROJECT: 131 Parkdale Avenue, Ottawa, Ontario

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: December 1, 2025

FILE NO. : PG7788

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE					
ASPHALT 0.05m [60.79m]		AU 1			9
FILL: Brown silty sand, with crushed stone and rock fragments, some clay 1.30m [59.54m]		SS 2	85	8-18-50-/ 68/0.18	6
BEDROCK: Poor to excellent quality limestone		RC 1	100	RQD 39	
		RC 2	100	RQD 100	
		RC 3	100	RQD 100	
		RC 4	100	RQD 97	
		RC 5	100	RQD 100	
		RC 6	100	RQD 95	
11.66m [49.18m]		RC 7	100	RQD 100	

End of Borehole

(GWL at 4.80 m depth - December 17, 2025)

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

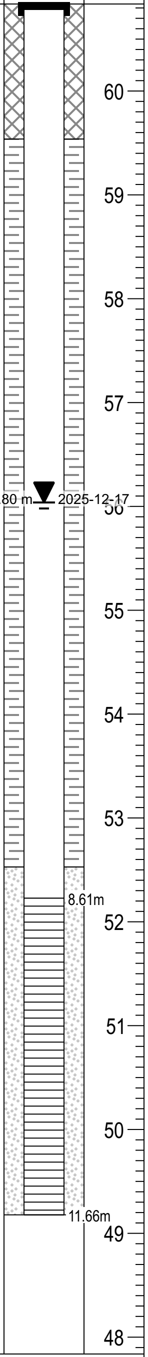
△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

STRATA PLOT

DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12 13

ELEVATION (m): 60 59 58 57 56 55 54 53 52 51 50 49 48



P:/AutoCAD Drawings/Test Hole Data Files/PG77xx/PG7788/data.sqlite 2026-01-02, 10:30 Paterson_Template MA

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

131 Parkdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364831.08 **NORTHING:** 5030097.29 **ELEVATION:** 60.64

PROJECT: 131 Parkdale Avenue, Ottawa, Ontario

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: December 1, 2025

FILE NO. : PG7788

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	Depth / Elevation
GROUND SURFACE	
ASPHALT	0.05m [60.59m]
FILL: Brown silty sand, with crushed stone, trace clay, brick and organics	0.84m [59.80m]
BEDROCK: Excellent quality limestone	11.68m [48.96m]
End of Borehole	
(GWL at 3.35 m depth - December 17, 2025)	

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			13
RC 1	100	RQD 100	
RC 2	100	RQD 97	
RC 3	100	RQD 97	
RC 4	100	RQD 100	
RC 5	100	RQD 98	
RC 6	100	RQD 100	
RC 7	100	RQD 97	
RC 8	100	RQD 100	

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

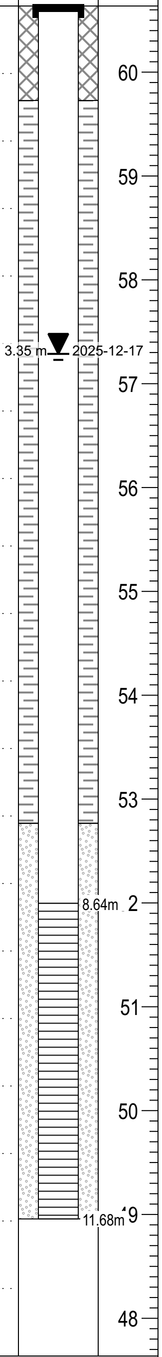
△ REMOULDED SHEAR STRENGTH (kPa)

▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12 13

MONITORING WELL CONSTRUCTION



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

131 Parkdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364868.15 **NORTHING:** 5030098.69 **ELEVATION:** 61.60

PROJECT: 131 Parkdale Avenue, Ottawa, Ontario

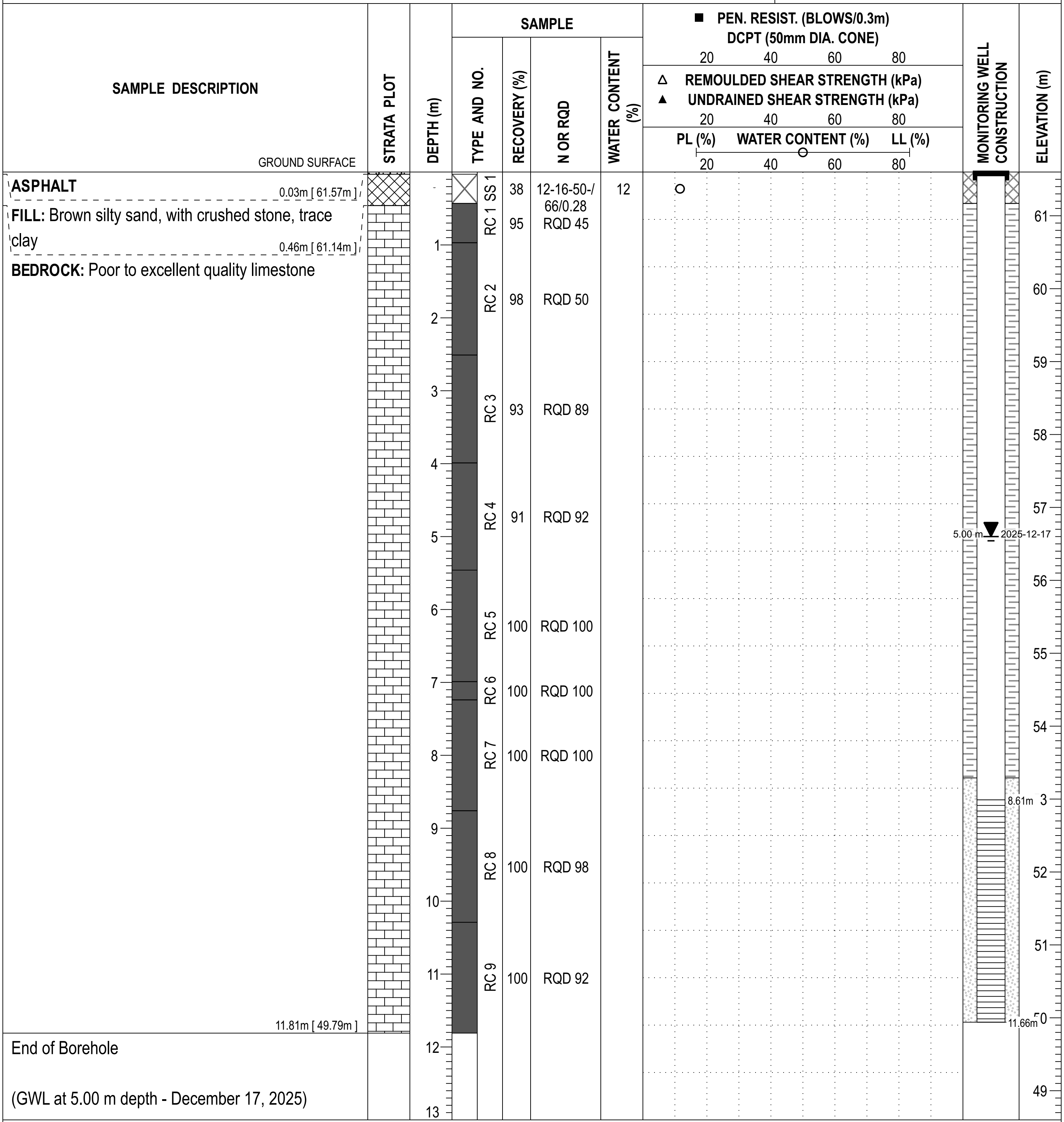
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: December 2, 2025

FILE NO. : PG7788

HOLE NO. : BH 3-25



SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE					
ASPHALT 0.03m [61.57m]		SS 1	38	12-16-50-/66/0.28	12
FILL: Brown silty sand, with crushed stone, trace clay 0.46m [61.14m]		RC 1	95	RQD 45	
BEDROCK: Poor to excellent quality limestone		RC 2	98	RQD 50	
		RC 3	93	RQD 89	
		RC 4	91	RQD 92	
		RC 5	100	RQD 100	
		RC 6	100	RQD 100	
		RC 7	100	RQD 100	
		RC 8	100	RQD 98	
		RC 9	100	RQD 92	
11.81m [49.79m]					

End of Borehole

(GWL at 5.00 m depth - December 17, 2025)

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

131 Parkdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364851.99 **NORTHING:** 5030054.95 **ELEVATION:** 61.59

PROJECT: 131 Parkdale Avenue, Ottawa, Ontario

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: December 2, 2025

FILE NO. : PG7788

HOLE NO. : BH 4-25

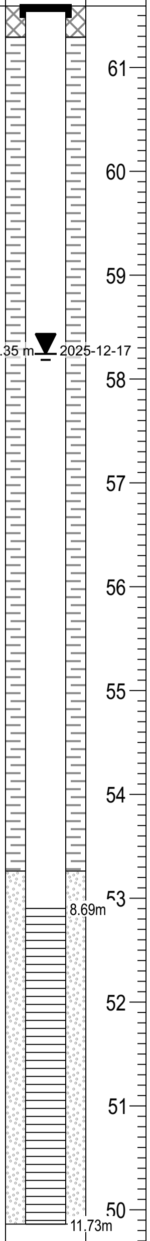
SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							61.59
FILL: Brown silty sand, with crushed stone, some brick, trace clay			SS 1	48	6-20-19-50 39	13	
0.81m [60.78m]		0.81					60.78
BEDROCK: Poor to excellent quality limestone			RC 1	78	RQD 0		
			RC 2	196	RQD 25		
			RC 3	63	RQD 85		
			RC 4	100	RQD 100		
			RC 5	100	RQD 100		
			RC 6	100	RQD 95		
			RC 7	100	RQD 100		
			RC 8	97	RQD 98		
11.73m [49.86m]		11.73					49.86

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

MONITORING WELL CONSTRUCTION



End of Borehole

(GWL at 3.35 m depth - December 17, 2025)

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

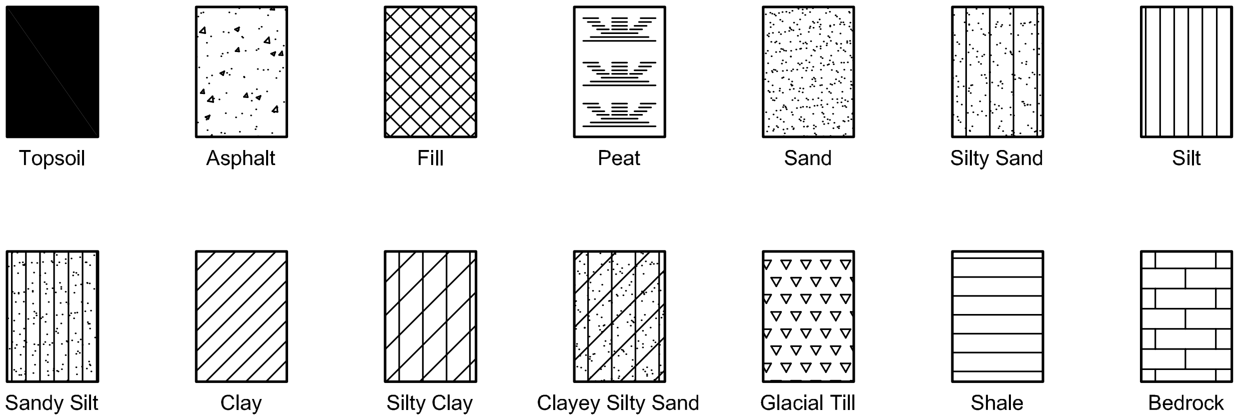
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

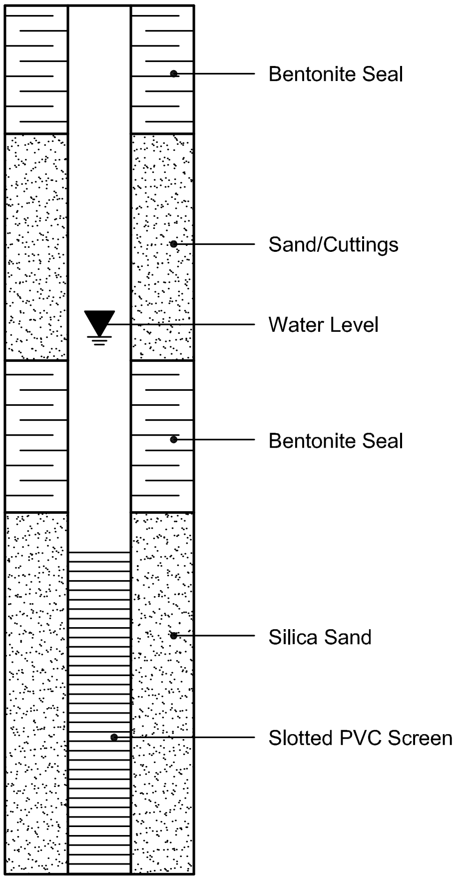
SYMBOLS AND TERMS (continued)

STRATA PLOT

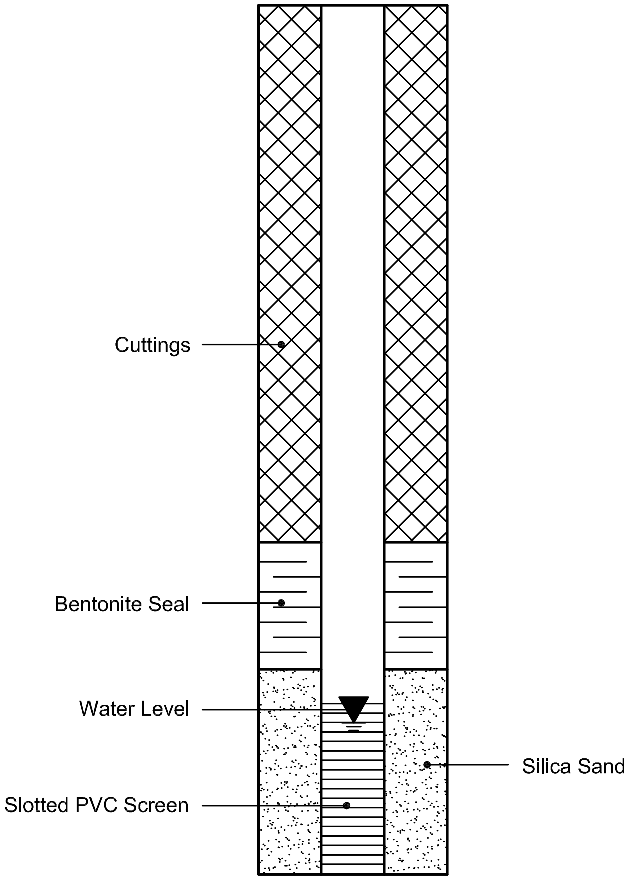


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



LOPERS & ASSOCIATES

Lopers & Associates
30 Lansfield Way
Ottawa, Ontario K2G3V8

BH5-25

PAGE 1 OF 1

CLIENT 11034936 Canada Inc.

PROJECT NAME Phase Two Environmental Site Assessment

PROJECT NUMBER LOP25-035B

PROJECT LOCATION 131 Parkdale Avenue, Ottawa, Ontario

DATE STARTED 25-12-3 **COMPLETED** 25-12-3

GROUND ELEVATION

HOLE SIZE 20 cm

DRILLING CONTRACTOR George Downing Estate Drilling

GROUND WATER LEVELS:

DRILLING METHOD Truck Mounted CME 55

AFTER DRILLING ---

LOGGED BY L. Lopers **CHECKED BY** D. Plenderleith

NOTES

DEPTH (m)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	ENVIRONMENTAL DATA	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
	AU 1		Vapor = 0		Silty Sand and Gravel with some silt (Fill). Grey, compact, dry. 0.76	

Auger Refusal on inferred Limestone Bedrock @ 0.76 m BGS.
Bottom of hole at 0.76 m.

LOPERS & ASSOCIATES

Lopers & Associates
30 Lansfield Way
Ottawa, Ontario K2G3V8

BH6-25

PAGE 1 OF 1

CLIENT 11034936 Canada Inc.

PROJECT NAME Phase Two Environmental Site Assessment

PROJECT NUMBER LOP25-035B

PROJECT LOCATION 131 Parkdale Avenue, Ottawa, Ontario

DATE STARTED 25-12-3 **COMPLETED** 25-12-3

GROUND ELEVATION **HOLE SIZE** 20 cm

DRILLING CONTRACTOR George Downing Estate Drilling

GROUND WATER LEVELS:

DRILLING METHOD Truck Mounted CME 55

AFTER DRILLING ---

LOGGED BY L. Lopers **CHECKED BY** D. Plenderleith

NOTES

DEPTH (m)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	ENVIRONMENTAL DATA	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
	AU 1		Vapor = 0		0.25 Silty Sand and Gravel with some metal pieces (Fill). Grey, loose, dry. Auger Refusal on inferred Limestone Bedrock @ 0.25 m BGS.	

Bottom of hole at 0.25 m.

LOPERS & ASSOCIATES

Lopers & Associates
30 Lansfield Way
Ottawa, Ontario K2G3V8

BH7-25

PAGE 1 OF 1

CLIENT 11034936 Canada Inc.

PROJECT NAME Phase Two Environmental Site Assessment

PROJECT NUMBER LOP25-035B

PROJECT LOCATION 131 Parkdale Avenue, Ottawa, Ontario

DATE STARTED 25-12-3 **COMPLETED** 25-12-3

GROUND ELEVATION

HOLE SIZE 20 cm

DRILLING CONTRACTOR George Downing Estate Drilling

GROUND WATER LEVELS:

DRILLING METHOD Truck Mounted CME 55

AFTER DRILLING ---

LOGGED BY L. Lopers **CHECKED BY** D. Plenderleith

NOTES

DEPTH (m)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	ENVIRONMENTAL DATA	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
	AU 1		Vapor = 0	0.61	Silty Sand and Gravel with some metal pieces (Fill). Grey, loose, dry.	

Auger Refusal on inferred Limestone Bedrock @ 0.61 m BGS.
Bottom of hole at 0.61 m.

BH8-25

CLIENT 11034936 Canada Inc.

PROJECT NAME Phase Two Environmental Site Assessment

PROJECT NUMBER LOP25-035B

PROJECT LOCATION 131 Parkdale Avenue, Ottawa, Ontario

DATE STARTED 25-12-3 **COMPLETED** 25-12-3

GROUND ELEVATION ______ **HOLE SIZE** 20 cm

DRILLING CONTRACTOR George Downing Estate Drilling

GROUND WATER LEVELS:

DRILLING METHOD Truck Mounted CME 55

AFTER DRILLING ---

LOGGED BY L. Lopers **CHECKED BY** D. Plenderleith

NOTES ______

DEPTH (m)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	ENVIRONMENTAL DATA	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
	AU 1		Vapor = 0		0.35 Silty Sand and Gravel (Fill). Grey, compact, dry.	

Auger Refusal on inferred Limestone Bedrock @ 0.35 m BGS.
Bottom of hole at 0.35 m.

LOPERS & ASSOCIATES

Lopers & Associates
30 Lansfield Way
Ottawa, Ontario K2G3V8

BH9-25

PAGE 1 OF 1

CLIENT 11034936 Canada Inc.

PROJECT NAME Phase Two Environmental Site Assessment

PROJECT NUMBER LOP25-035B

PROJECT LOCATION 131 Parkdale Avenue, Ottawa, Ontario

DATE STARTED 25-12-3 **COMPLETED** 25-12-3

GROUND ELEVATION **HOLE SIZE** 20 cm

DRILLING CONTRACTOR George Downing Estate Drilling

GROUND WATER LEVELS:

DRILLING METHOD Truck Mounted CME 55

AFTER DRILLING ---

LOGGED BY L. Lopers **CHECKED BY** D. Plenderleith

NOTES

DEPTH (m)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	ENVIRONMENTAL DATA	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
	AU 1		Vapor = 0	0.35	Silty Sand and Gravel (Fill). Grey, compact, dry.	

Auger Refusal on inferred Limestone Bedrock @ 0.35 m BGS.
Bottom of hole at 0.35 m.

LOPERS & ASSOCIATES

Lopers & Associates
30 Lansfield Way
Ottawa, Ontario K2G3V8

BH10-25

PAGE 1 OF 1

CLIENT 11034936 Canada Inc.

PROJECT NAME Phase Two Environmental Site Assessment

PROJECT NUMBER LOP25-035B

PROJECT LOCATION 131 Parkdale Avenue, Ottawa, Ontario

DATE STARTED 25-12-3 **COMPLETED** 25-12-3

GROUND ELEVATION **HOLE SIZE** 20 cm

DRILLING CONTRACTOR George Downing Estate Drilling

GROUND WATER LEVELS:

DRILLING METHOD Truck Mounted CME 55

AFTER DRILLING ---

LOGGED BY L. Lopers **CHECKED BY** D. Plenderleith

NOTES

DEPTH (m)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	ENVIRONMENTAL DATA	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
	AU 1		Vapor = 1	0.51	Silty Sand and Gravel with Brick and Glass pieces (Fill). Brown to black, compact, dry.	

Auger Refusal on inferred Limestone Bedrock @ 0.51m BGS.
Bottom of hole at 0.51 m.

LOPERS & ASSOCIATES

Lopers & Associates
30 Lansfield Way
Ottawa, Ontario K2G3V8

BH11-25

PAGE 1 OF 1

CLIENT 11034936 Canada Inc.

PROJECT NAME Phase Two Environmental Site Assessment

PROJECT NUMBER LOP25-035B

PROJECT LOCATION 131 Parkdale Avenue, Ottawa, Ontario

DATE STARTED 25-12-3 **COMPLETED** 25-12-3

GROUND ELEVATION **HOLE SIZE** 20 cm

DRILLING CONTRACTOR George Downing Estate Drilling

GROUND WATER LEVELS:

DRILLING METHOD Truck Mounted CME 55

AFTER DRILLING ---

LOGGED BY L. Lopers **CHECKED BY** D. Plenderleith

NOTES

DEPTH (m)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	ENVIRONMENTAL DATA	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
	AU 1		Vapor = 0		0.30 Silty Sand and Gravel with some silt (Fill). Brown to black, loose, dry. Brick and glass pieces mixed with fill materials. Auger Refusal on inferred Limestone Bedrock @ 0.3m BGS. Bottom of hole at 0.31 m.	

Certificate of Analysis

Report Date: 24-Dec-2025

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

Order Date: 18-Dec-2025

Client PO: 64748

Project Description: PG7788

Client ID:		BH4-25-SS1	-	-	-		
Sample Date:		18-Dec-25 08:39	-	-	-	-	-
Sample ID:		2551408-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	88.9	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	10.29	-	-	-	-	-
Resistivity	0.1 Ohm.m	21.2	-	-	-	-	-
Anions							
Chloride	10 ug/g	52	-	-	-	-	-
Sulphate	10 ug/g	233	-	-	-	-	-

APPENDIX 2

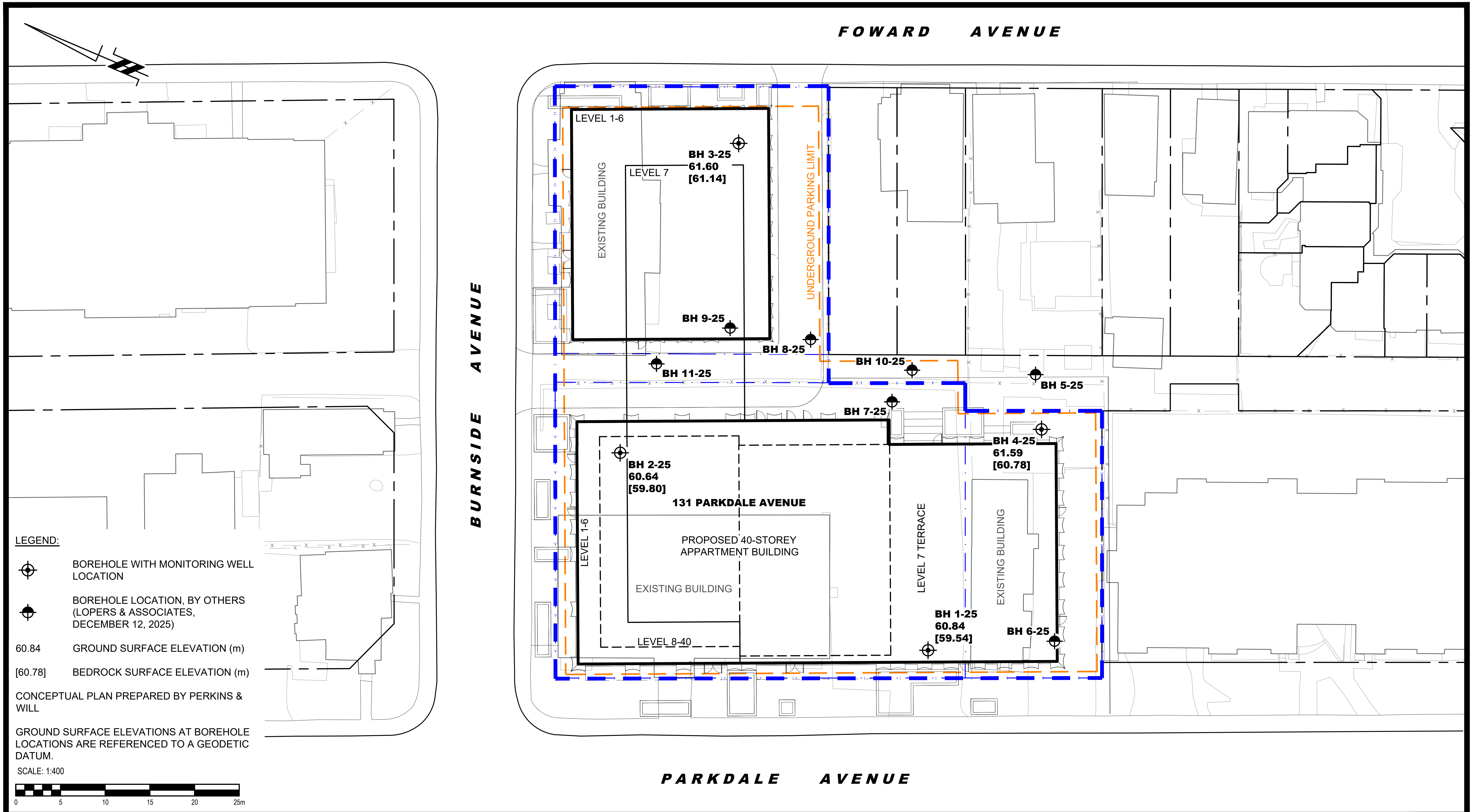
FIGURE 1 - KEY PLAN

DRAWING PG7788-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
●	BOREHOLE LOCATION, BY OTHERS (LOPERS & ASSOCIATES, DECEMBER 12, 2025)
60.84	GROUND SURFACE ELEVATION (m)
[60.78]	BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PREPARED BY PERKINS & WILL

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

PATERSON GROUP

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

NO.	REVISIONS	DD/MM/YYYY	INITIAL

11034936 CANADA INC.
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
131 PARKDALE AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 12/2025
Drawn by: ZS
Report No.: PG7788-1
Checked by: ZA
Dwg. No.: **PG7788-1**
Approved by: JV
Revision No.:

11x17

p:\autocad drawings\geotechnical\pg77xx\pg7788\pg7788-1-test hole location plan.dwg