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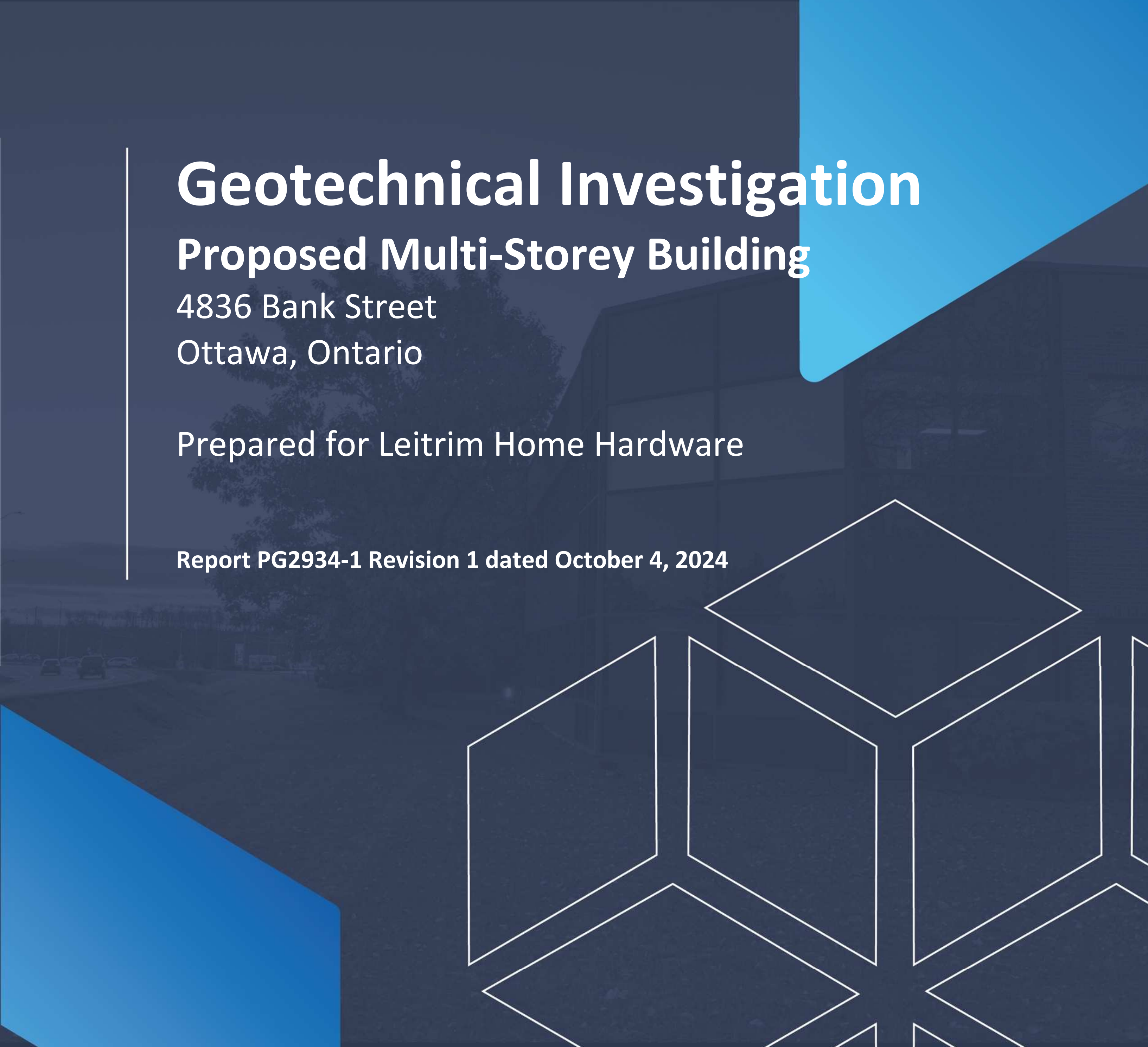

# Geotechnical Investigation

## Proposed Multi-Storey Building

4836 Bank Street
Ottawa, Ontario

Prepared for Leitrim Home Hardware

**Report PG2934-1 Revision 1 dated October 4, 2024**

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# 1.0 Introduction

Paterson Group (Paterson) was commissioned by Leitrim Home Hardware to conduct a geotechnical investigation for the proposed development to be located at 4836 Bank in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

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

This 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 site plan, it is understood that the proposed development at the subject site will consist of a multi-storey building with 2 levels of underground parking.

Associated at-grade access lanes and parking areas are to be located to the west of the proposed building, with paved walkways to the east and landscaped margins to the north and south. It is also expected that the proposed building will be municipally serviced.

# 3.0 Method of Investigation

## 3.1 Field Investigation

### Field Program

The current field investigation was carried out on September 13 and 16, 2024, which consisted of a total of 7 boreholes advanced at the subject site to a maximum depth of 6.7 m below the existing ground surface.

The initial geotechnical investigation was conducted on November 5, 2019, which included 3 test pits (TP 13, TP 14 and TP 16) located at the subject site. These test pits were excavated to a maximum depth of 2.65 m below ground surface.

The boreholes were advanced using a low-clearance track-mounted auger drill rig operated by a two-person crew. The drilling procedure consisted of augering and rock coring to the required depths at the selected borehole locations, and sampling and testing the soil and bedrock.

The test pit procedure consisted of excavating with a backhoe to depth at the selected locations and sampling the overburden. Field notes were logged as per ASTM D5434.

All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division.

The test hole locations were determined by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations are shown on Drawing PG2934-1 - Test Hole Location Plan attached in Appendix 2.

### Sampling and In Situ Testing

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

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.

Soil samples from the test pits were recovered from the side walls of the open excavation, and all soil samples were initially classified on-site. The depths at which the grab samples were recovered from the test pits are shown as G on the Soil Profile and Test Data sheets in Appendix 1.

The subsurface conditions observed in the test holes 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 boreholes BH 1B-24, BH 2-24, and BH 3-24, to permit long-term groundwater measurement subsequent to the field investigation. Flexible polyethylene standpipes were installed in the remaining boreholes to permit further groundwater measurement.

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

The groundwater observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

### 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 and elevations were surveyed in the field by Paterson. The ground surface elevations at the test hole locations were referenced to a geodetic datum. The test hole locations, and the ground surface elevations at the test hole locations are presented on Drawing PG2934-1 - Test Hole Location Plan in Appendix 2.

## 3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless we are otherwise directed.

## 3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The 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, with a grassed surface across the western portion of the site, and an asphalt-paved surface nearest to Bank Street. The ground surface is relatively level and at-grade with the surrounding roads.

A commercial building was previously located at the site but was demolished in recent years.

## 4.2 Subsurface Profile

**Overburden**

Generally, the soil profile profile encountered at the test hole locations consists of fill underlain by glacial till. The fill was generally observed to consist of silty sand with some gravel, extending to approximate depths of 0.6 to 2.95 m below the existing ground surface.

The glacial till, encountered directly underlying the fill, was observed to consist of compact to very dense, brown to grey silty sand with gravel, cobbles, and boulders.

Practical refusal to augering was encountered at depths ranging from 2.5 to 6 m below the existing ground surface.

**Bedrock**

Bedrock was cored at borehole BH 1B-24 from approximate depths of 5.2 to 6.7 m. The bedrock was observed to consist of excellent quality sandstone.

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

Based on available geological mapping, the site is located in an area where the bedrock consists of interbedded sandstone and dolomite of the March formation with a drift thickness ranging from about 3 to 5 m in depth.

## 4.3 Groundwater

Groundwater levels were recorded at each monitoring well location on September 27, 2024. The measured groundwater level readings are presented in Table 1 below and are also shown on the Soil Profile and Test Data sheets in Appendix 1.

**Table 1 – Summary of Groundwater Levels**

| Test hole Number | Ground Surface Elevation (m) | Measured Groundwater Level | | Date Recorded |
|---|---|---|---|---|
| | | Depth (m) | Elevation (m) | |
| BH 1B-24 | 100.55 | 4.58 | 95.97 | September 27, 2024 |
| BH 2-24 | 100.52 | 4.79 | 95.73 | September 27, 2024 |
| BH 3-24 | 100.12 | 1.17 | 98.95 | September 27, 2024 |

**Note:**
-The ground surface elevation at each test hole location was surveyed using a high precision GPS and are referenced to a geodetic datum.

The test pits, excavated to a maximum depth of 2.65 m, were noted to be dry upon the completion of excavation.

Based on these observations, the long-term groundwater level is expected to range between approximately 3.5 to 4.5 m below 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 suitable for the proposed development. The proposed building is recommended to be founded on conventional spread footings bearing on the undisturbed glacial till or clean, surface sounded bedrock.

Bedrock removal is expected in order to complete the excavation for the underground parking levels. Boulders should also be anticipated within the glacial till.
The above and other considerations are further discussed in the following sections.

## 5.2 Site Grading and Preparation

### Stripping Depth

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

Existing foundation walls and other demolished debris should be completely removed from the proposed building perimeter and within the lateral support zones of the foundation. Under paved area, existing construction remnants, such as foundation walls should be excavated to a minimum of 1 m below final grade.

### Fill Placement

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

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill materials should be spread in thin lifts and at a minimum compacted by the tracks of the spreading equipment to minimize voids. If the non-specified backfill is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 98% of the material’s SPMDD.

## Bedrock Removal

The proposed building excavation will likely extend into the bedrock. 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 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 are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

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

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

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

### Lean Concrete Filled Trenches

Where bedrock overbreak occurs below the design underside of footing elevation (USF), lean concrete (minimum **17 MPa** 28-day compressive strength) can be used to reinstate grades from the clean-surface sounded bedrock up to the USF elevation. Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying 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

### Footings on Glacial Till

Footings placed on an undisturbed glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance value at ULS.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

### Footings on Bedrock

A factored bearing resistance value at SLS and ULS of **3,000 kPa,** incorporating a geotechnical resistance factor of 0.5, can be used for the design of conventional spread footings bearing on the clean, surface sounded bedrock.

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

Footings supported on clean, surface-sounded bedrock, and designed for the bearing resistance values provided herein, will be subjected to negligible 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 horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A soil bearing medium, or a heavily fractured, weathered bedrock bearing medium, will require a lateral support zone of 1H:1V (or flatter).

## 5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If the footings are bearing on bedrock, and a higher seismic site class (Class A or B) is required for the proposed building, then a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The soils underlying the site are not susceptible to liquefaction.

## 5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the glacial till or bedrock will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. It is understood that the lowest level of the proposed building will be mostly parking and the recommended pavement structures noted in Section 5.7 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

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

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the subfloor fill under the lowest level floor slab. This is discussed further in Section 5.5.

## 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/$m^3$.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/$m^3$, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

### Lateral Earth Pressures

The static horizontal earth pressure (po) can be calculated using a triangular earth pressure distribution equal to Ko·γ·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 ($\Delta P_{AE}$).

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

$a_c$ = $(1.45\text{-}a_{max}/g)a_{max}$
$\gamma$ = 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 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component ($P_o$) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

## 5.7 Pavement Design

### Lowest Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the lowest underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 2 below.

**Table 2 - Recommended Rigid Pavement Structure – Underground Parking Level**

| Thickness (mm) | Material Description |
|---|---|
| 150 | **Exposure Class C2** – **32 MPa Concrete** (5 to 8% Air Entrainment) |
| 300 | **BASE** - OPSS Granular A Crushed Stone |
| **SUBGRADE** – Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock. | |

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level.

The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

### Pavement Structure On Podium Deck

The pavement structures presented in Tables 3 and 4 should be used for car only parking areas, at grade access lanes and heavy loading parking areas located over the top of the podium structure.

**Table 3 - Recommended Pavement Structure - Car Only Parking Areas Over Podium Deck**

| Thickness (mm) | Material Description |
|---|---|
| 50 | **Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete |
| 200* | **BASE** - OPSS Granular A Crushed Stone |
| See below** | **Thermal Break** -** Rigid Insulation (See Following Paragraph) |
| n/a | **Waterproofing Membrane and IKO Protection Board** |

**SUBGRADE** – Reinforced concrete podium deck

* Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph

** If specified by others, not required from a geotechnical perspective

**Table 4 - Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Heavy Loading Areas Over Podium Deck**

| 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 |
| See below** | **Thermal Break** -** Rigid Insulation (See Following Paragraph) |
| n/a | **Waterproofing Membrane and IKO Protection Board** |

**SUBGRADE** – Reinforced concrete podium deck

* Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph

** If specified by others, not required from a geotechnical perspective

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section.

For this transition, a 5H:1V is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 500 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 60 (HI-60), or High-Load 40 (HI-40). The base layer thickness will be dependent on the grade of insulation considered for this project and should be reassessed by the geotechnical consultant once pertinent design details have been prepared.

The higher grade of insulation have more resistance to deformation under wheel-loading and require less granular cover to avoid being crushing by vehicular loading. It should be noted that SM (Styrofoam) rigid insulation is **not** considered suitable for this application.

### Pavement Structure on Overburden Soils

Beyond the podium deck, the following pavement structures in Tables 5 and 6 may be used for car only parking and heavy traffic areas on overburden.

**Table 5 - 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 in-situ soils, existing imported fill or OPSS Granular B Type I or II material placed over in-situ soil or bedrock. | |

**Table 6 - Recommended Pavement Structure – Access Lanes, Heavy Traffic and Loading 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 |
| 150 | **BASE** - OPSS Granular A Crushed Stone |
| 450 | **SUBBASE –** OPSS Granular B Type II |
| **SUBGRADE** – Either in-situ soils, existing imported fill or OPSS Granular B Type I or II material placed over in-situ soil or bedrock. | |

### Other Considerations

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 Waterproofing

### Waterproofing and Composite Drainage Board

For the proposed underground parking levels, it is anticipated that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a drainage and waterproofing system which is fastened to the shoring system.

Waterproofing of the foundation walls is recommended and the membrane is to be installed from 3 m below finished grade down the foundation walls, to the bottom of foundation. The waterproofing membrane is recommended to consist of Tremco Paraseal, or an approved equivalent.

It is also recommended that a composite drainage board, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation walls, extending from the exterior finished grade to the founding elevation (USF). The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 100 mm or 150 mm diameter sleeves at 3 m centres be cast in the foundation walls at the perimeter footing interface, to allow the infiltration of water to flow to an interior perimeter underslab drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

A waterproofing system should also be provided for any elevator pits (pit bottom and walls).

### Perimeter and Underslab Drainage System

The perimeter and underslab drainage system is recommended to control water infiltration below the underground parking level slab and to re-direct water from the buildings foundation drainage system to the building’s sump pit(s). For preliminary design purposes, it is recommended that 100 mm or 150 mm perforated pipes provided with a geosock, surrounded on all sides by a minimum 150 mm thick layer of 19 mm clear crushed stone, be placed at approximate 6 m centres underlying the underground parking level slab.

The perimeter drainage system should be mechanically connected to the 150 mm drainage sleeves and gravity connected to the underslab drainage system, which in turn is connected to the building’s sump pit(s).

The spacing of the underslab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

**Foundation Backfill**

The exterior foundation walls can then be backfilled with the site excavated materials, provided that they are maintained in an unfrozen state and at a suitable moisture content for compaction. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

## 6.2 Protection of Footings Against Frost Action

Perimeter 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 exterior 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 a combination of soil cover and foundation insulation.

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

## 6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

**Unsupported Side Slopes**

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

Excavation side slopes carried out for the building footprint are recommended to be provided with surface protection from erosion by rain and surface water runoff, where shoring is not anticipated to be implemented.

This can be accomplished by covering the entire surface of the excavation side slopes with tarps secured between the top and bottom of the overburden excavation, and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of the side slopes.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

**Temporary Shoring**

Temporary shoring may be required for the overburden soil to complete the required excavations, where insufficient room is available for open cut methods. The shoring requirements, designed by a structural engineer specializing in those works, will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of the temporary shoring system will be the responsibility of the excavation contractor and their design team.

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

The temporary shoring system could consist of a soldier pile and lagging system. Any additional loading due to street traffic, neighbouring buildings, construction equipment, adjacent structures and facilities, etc., should be included in the earth pressures described below.

The earth pressures acting on the temporary shoring system may be calculated with the parameters presented in Table 7, presented in the page below.

**Table 7 – 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 |
| Dry Unit Weight (γ), kN/$m^3$ | 20 |
| Effective Unit Weight (γ), kN/$m^3$ | 13 |

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated to full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

### Bedrock Stabilization

Excavation side slopes in sound bedrock can be carried out using vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

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

The requirement for temporary rock anchors, shotcrete, and/or chainlink fencing should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage of the project.

## 6.4 Pipe Bedding and Backfill

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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material for areas over a soil subgrade. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at a minimum to the spring line of the pipe.

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

Generally, it should be possible to re-use the moist (not wet) silty sand to sandy silt and glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions.

Wet sub-excavated soil should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used. All stones greater than 300 mm in their greatest dimension should be removed prior to reuse of site-generated glacial till.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should consist of 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 SPMDD.

## 6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

### Dewatering Permit

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 Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

### Impacts on Neighbouring Properties

It is understood that 2 levels of underground parking are planned for the proposed building, with the lower portion of the foundation walls having a groundwater infiltration control system in place. In addition, given the shallow glacial till present at, and in the vicinity of, the subject site, the neighbouring structures are expected to be founded on glacial till which is not susceptible to settlement from dewatering. Therefore, no issues are expected with respect to groundwater lowering that would cause damage to adjacent structures surrounding the proposed development.

## 6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

## 6.7 Corrosion Potential and Sulphate

The analytical test results of the soil sample indicate that the sulphate content is less than 0.1%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (Type GU) would be appropriate for this site. The chloride content and the pH of the sample indicate they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a low to moderate aggressive environment.

# 7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- Review of the geotechnical aspects of the foundation drainage systems prior to construction, if applicable.
- Review of the geotechnical aspects of the excavation contractor’s shoring design prior to construction, if applicable.

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

- Review and inspection of the installation of the foundation drainage and waterproofing systems.
- 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 and follow-up field density tests to determine the level of compaction achieved.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

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

# 8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Leitrim Home Hardware, 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
Oct. 4, 2024
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Puneet Bandi, M.Eng.

Scott Dennis, P.Eng.

**Report Distribution:**

☐ Leitrim Home Hardware
☐ Paterson Group

# APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

PATERSON GROUP

# SOIL PROFILE AND TEST DATA

## Geotechnical Investigation

4836 Bank Street, Ottawa, Ontario

**COORD. SYS.:** MTM ZONE 9 **EASTING:** 376313.80 **NORTHING:** 5019255.98 **ELEVATION:** 100.54

**PROJECT:** Proposed High-rise development

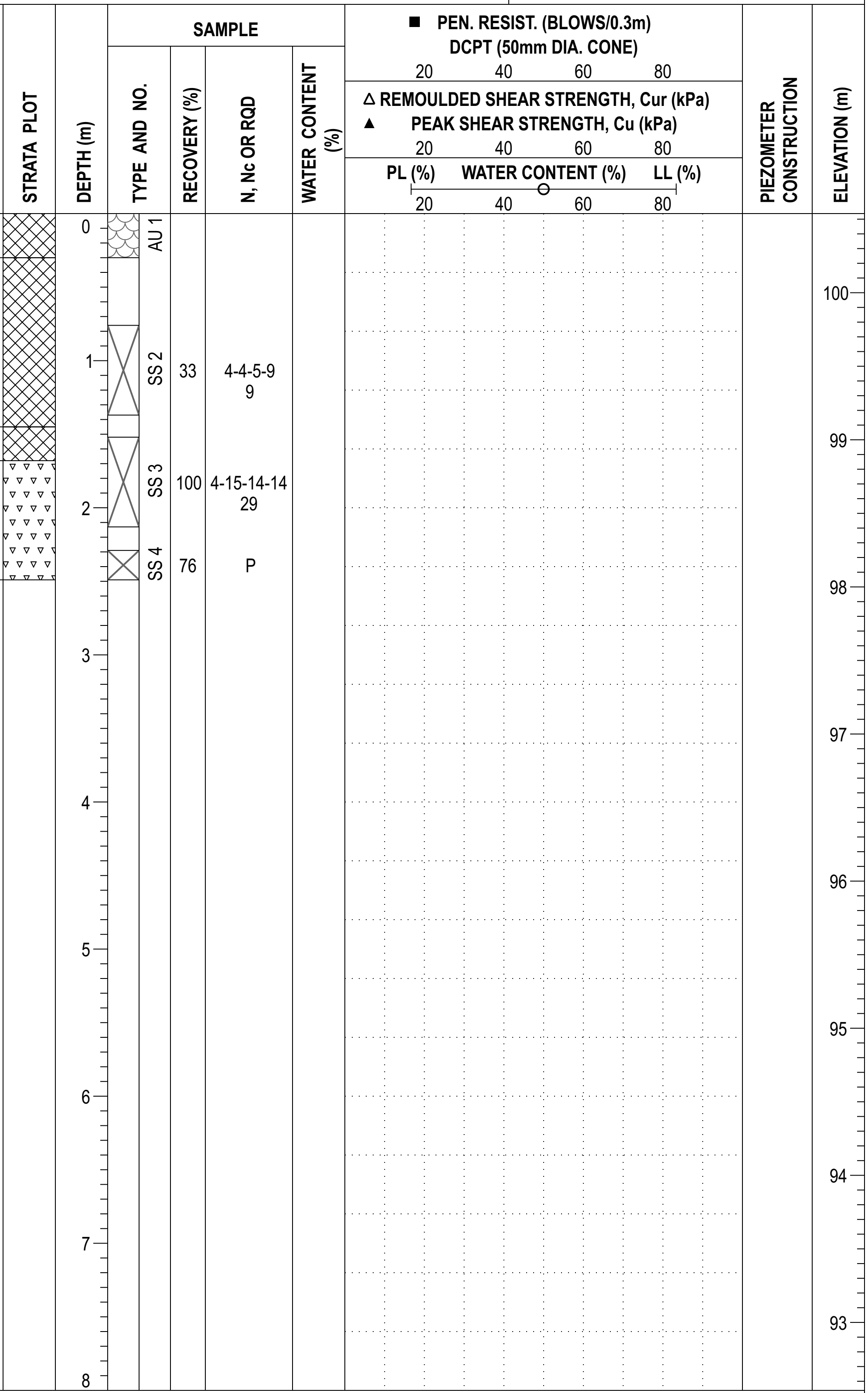
**BORINGS BY:** CME-55 Low Clearance Drill

**REMARKS:**

**DATE:** September 13, 2024

**FILE NO. :** PG2934

**HOLE NO. :** BH 1-24



**SAMPLE DESCRIPTION**

GROUND SURFACE

**FILL:** Granular and crushed stone

0.30m [ 100.24m ]

**FILL:** Loose, brown silty fine sand, with clay, trace gravel

1.45m [ 99.09m ]

**FILL:** Loose, grey sandy silt, trace brown silty sand

1.68m [ 98.86m ]

**GLACIAL TILL:** Compact, brown silty sand, with gravel, cobbles and boulders

- Very dense by 2.44 m depth

2.49m [ 98.05m ]

End of Borehole

Practical refusal to augering at 2.49 m depth

| TYPE AND NO. | RECOVERY (%) | N, Nc OR RQD |
|---|---|---|
| AU 1 | | |
| SS 2 | 33 | 4-4-5-9<br>9 |
| SS 3 | 100 | 4-15-14-14<br>29 |
| SS 4 | 76 | P |

# SOIL PROFILE AND TEST DATA

## Geotechnical Investigation

4836 Bank Street, Ottawa, Ontario

**COORD. SYS.:** MTM ZONE 9 **EASTING:** 376312.01 **NORTHING:** 5019256.37 **ELEVATION:** 100.67

**PROJECT:** Proposed High-rise development

**BORINGS BY:** CME-55 Low Clearance Drill

**REMARKS:**

**DATE:** September 13, 2024

**FILE NO. :** PG2934

**HOLE NO. :** BH 1A-24

**SAMPLE DESCRIPTION**

GROUND SURFACE

**FILL:** Granular and crushed stone

0.30m [ 100.37m ]

**FILL:** Loose, brown silty fine sand, with clay, trace gravel

1.45m [ 99.22m ]

**FILL:** Loose, grey sandy silt, trace brown silty sand

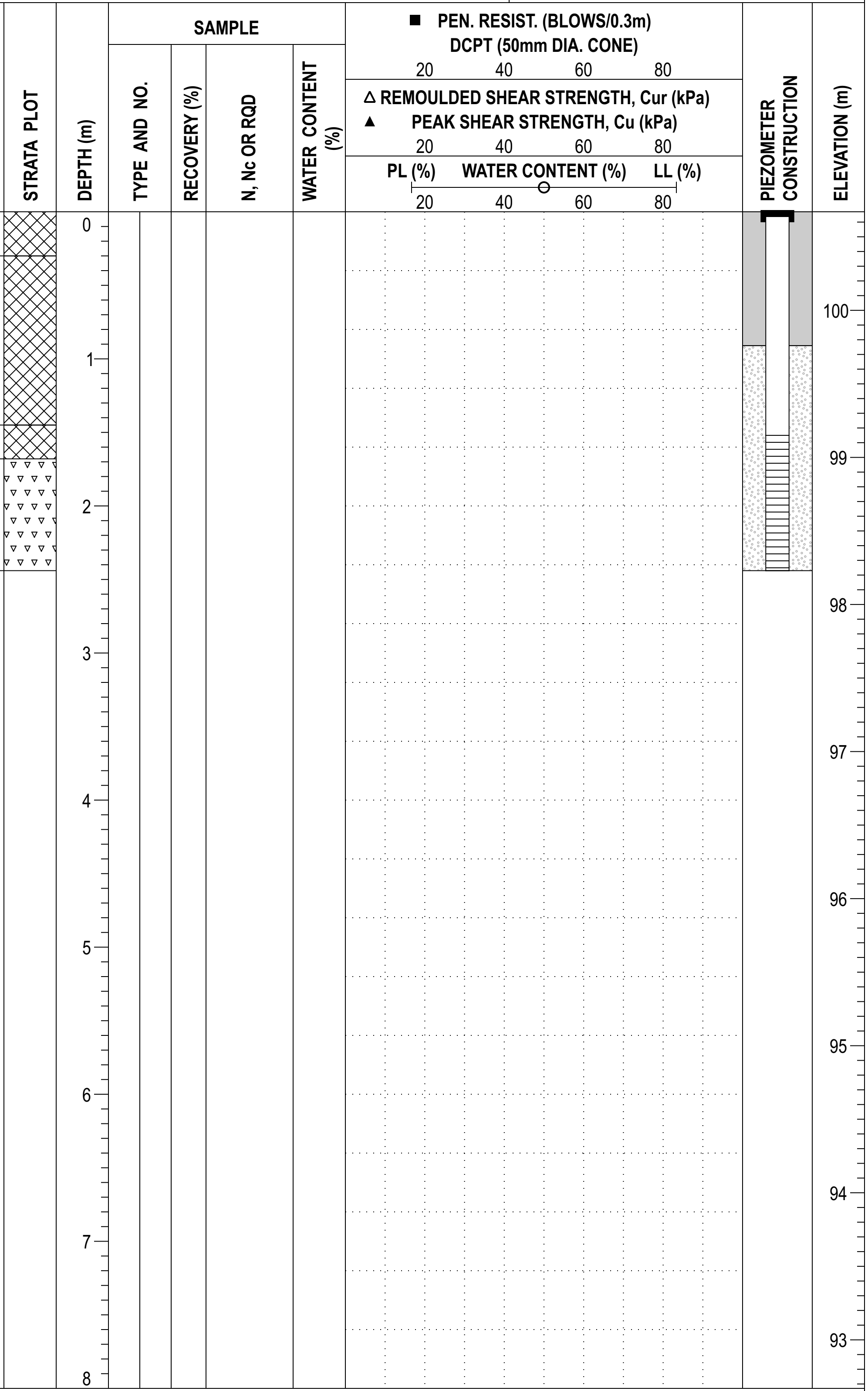
1.68m [ 98.99m ]

**GLACIAL TILL:** Compact, brown silty sand, with gravel, cobbles and boulders

2.44m [ 98.23m ]

End of Borehole

Practical refusal to augering at 2.44 m depth



# SOIL PROFILE AND TEST DATA

## Geotechnical Investigation

4836 Bank Street, Ottawa, Ontario

**COORD. SYS.:** MTM ZONE 9 **EASTING:** 376313.42 **NORTHING:** 5019255.24 **ELEVATION:** 100.55

**PROJECT:** Proposed High-rise development

**BORINGS BY:** CME-55 Low Clearance Drill

**REMARKS:**

**DATE:** September 16, 2024

**FILE NO. :** PG2934

**HOLE NO. :** BH 1B-24

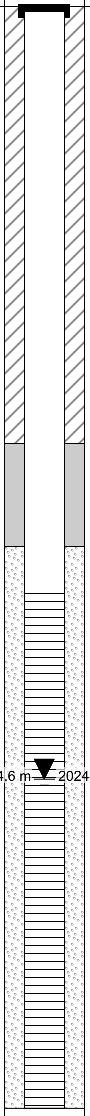
| SAMPLE DESCRIPTION | DEPTH (m) | SAMPLE TYPE AND NO. | RECOVERY (%) | N, Nc OR RQD | WATER CONTENT (%) |
|---|---|---|---|---|---|
| GROUND SURFACE | 0 | | | | |
| **FILL:** Granular and crushed stone<br>0.30m [ 100.25m ] | | | | | |
| **FILL:** Loose, brown silty fine sand, with clay, trace gravel<br>1.45m [ 99.10m ] | 1 | | | | |
| **FILL:** Loose, grey sandy silt, trace brown silty sand<br>1.68m [ 98.87m ] | | | | | |
| **GLACIAL TILL:** Very dense, brown silty fine sand to sandy silt, with gravel, cobbles and boulders | 2 | | | | |
| | 3 | SS 1 | 99 | 25-50-/-/<br>50/0.03 | |
| 3.66m [ 96.89m ] | | | | | |
| **GLACIAL TILL:** Very dense, grey silty fine sand, with gravel, cobble sand boulders | 4 | SS 2 | 91 | 20-50-/-/<br>50/0.13 | |
| | 5 | SS 3 | 94 | 10-30-40-50<br>70 | |
| 5.18m [ 95.37m ] | | | | | |
| **BEDROCK:** Excellent quality sandstone | 6 | RC 1 | 95 | RQD 92 | |
| 6.71m [ 93.84m ] | | | | | |
| End of Borehole | 7 | | | | |
| (GWL at 4.58 m depth - September 27, 2024 | 8 | | | | |

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

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

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

MONITORING WELL CONSTRUCTION



| ELEVATION (m) |
|---|
| 100 |
| 99 |
| 98 |
| 97 |
| 96 |
| 95 |
| 94 |
| 93 |

PATERSON GROUP

# SOIL PROFILE AND TEST DATA

## Geotechnical Investigation

4836 Bank Street, Ottawa, Ontario

**COORD. SYS.:** MTM ZONE 9 **EASTING:** 376269.35 **NORTHING:** 5019252.07 **ELEVATION:** 100.52

**PROJECT:** Proposed High-rise development

**BORINGS BY:** CME-55 Low Clearance Drill

**REMARKS:**

**DATE:** September 13, 2024

**FILE NO. :** PG2934

**HOLE NO. :** BH 2-24

| SAMPLE DESCRIPTION | DEPTH / ELEVATION |
|---|---|
| GROUND SURFACE | |
| **ASPHALT** | 0.04m [ 100.48m ] |
| **FILL:** Compact, brown silty fine sand, with crushed stone | 0.61m [ 99.91m ] |
| **FILL:** Compact, grey sandy silt, trace gravel and brown sandy silt | 0.91m [ 99.61m ] |
| **GLACIAL TILL:** Dense to very dense, brown silty fine sand to sandy silt, with gravel, cobbles and boulders<br>- Sand content increasing with depth | 4.11m [ 96.41m ] |
| **GLACIAL TILL:** Very dense to compact, grey sandy silt, with gravel, cobbles and boulders<br>- Some clay by 5.26 m depth | 5.97m [ 94.55m ] |

End of Borehole

Practical refusal to augering at 5.97 m depth

(GWL at 4.79 m depth - September 27, 2024)

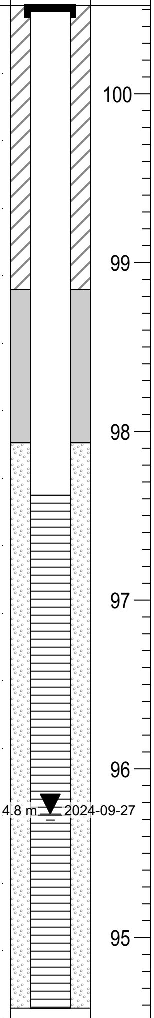
| TYPE AND NO. | RECOVERY (%) | N, Nc OR RQD | WATER CONTENT (%) |
|---|---|---|---|
| AU 1 | | | |
| SS 2 | 75 | 9-17-17-29<br>34 | |
| SS 3 | 91 | 11-50-50-/<br>100/0.23 | |
| SS 4 | 91 | 13-50-/-/<br>50/0.1 | |
| SS 5 | 100 | 7-40-50-/<br>90/0.18 | |
| SS 6 | 50 | 17-22-30-35<br>52 | |
| SS 7 | 75 | 6-16-32-16<br>48 | |
| SS 8 | 86 | 3-4-20-50<br>24 | |

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

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

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

MONITORING WELL CONSTRUCTION



DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7, 8

ELEVATION (m): 100, 99, 98, 97, 96, 95, 94, 93

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# SOIL PROFILE AND TEST DATA

## Geotechnical Investigation

4836 Bank Street, Ottawa, Ontario

PATERSON GROUP

**COORD. SYS.:** MTM ZONE 9 **EASTING:** 376304.22 **NORTHING:** 5019230.26 **ELEVATION:** 100.12

**PROJECT:** Proposed High-rise development

**BORINGS BY:** CME-55 Low Clearance Drill

**REMARKS:**

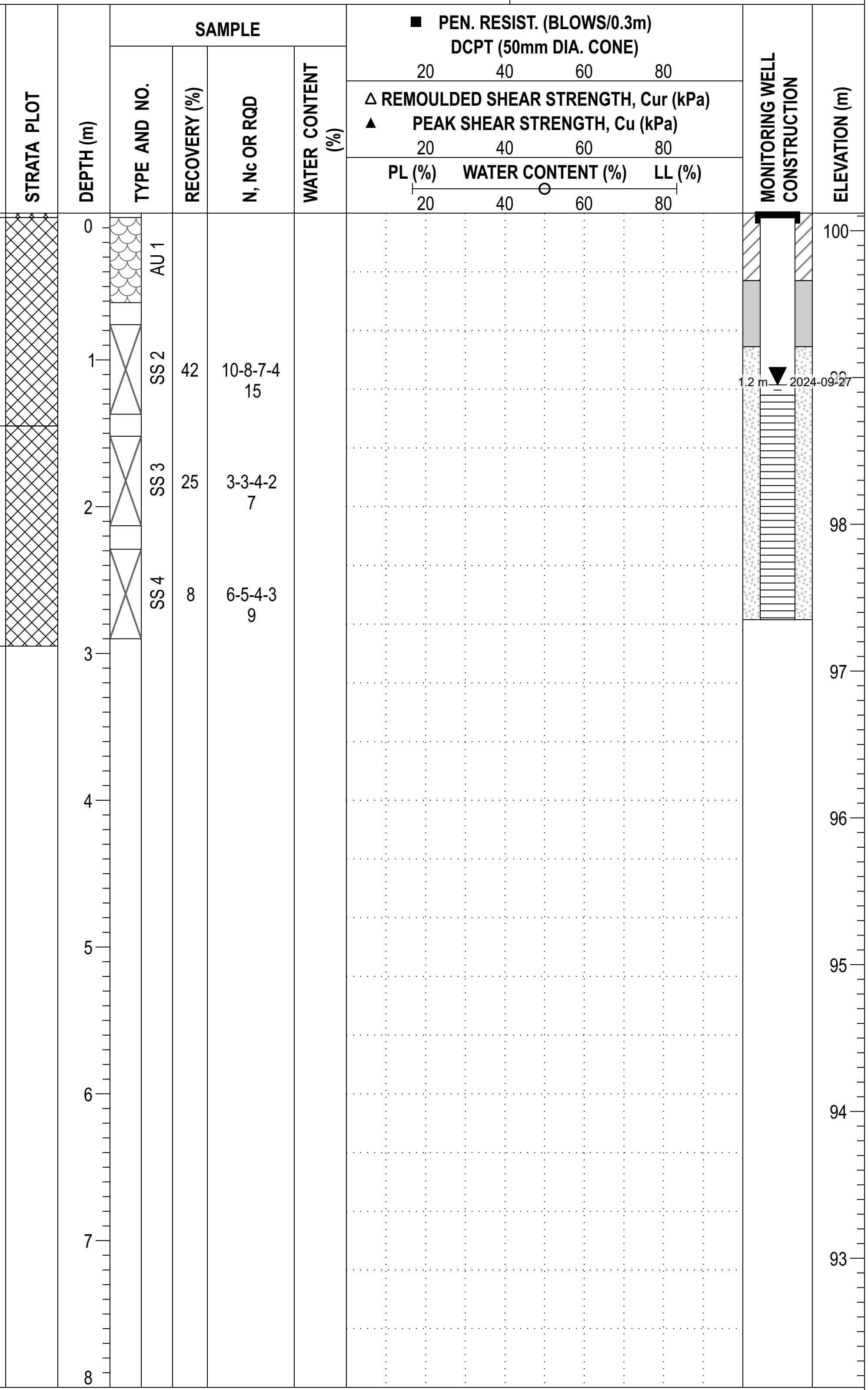
**DATE:** September 13, 2024

**FILE NO. :** PG2934

**HOLE NO. :** BH 3-24

| SAMPLE DESCRIPTION |
|---|
| GROUND SURFACE |
| **FILL:** Granular and crushed stone 0.03m [ 100.09m ] |
| **FILL:** Compact to loose, brown silty sand, with gravel and crushed stone 1.45m [ 98.67m ] |
| **FILL:** Loose, brown, medium sand with gravel 2.95m [ 97.17m ] |
| End of Borehole<br>Practical refusal to augering at 2.95 m depth<br>(GWL at 1.17 m depth - September 27, 2024) |

| SAMPLE TYPE AND NO. | RECOVERY (%) | N, Nc OR RQD | WATER CONTENT (%) |
|---|---|---|---|
| AU 1 | | | |
| SS 2 | 42 | 10-8-7-4<br>15 | |
| SS 3 | 25 | 3-3-4-2<br>7 | |
| SS 4 | 8 | 6-5-4-3<br>9 | |



PATERSON GROUP

# SOIL PROFILE AND TEST DATA

## Geotechnical Investigation

4836 Bank Street, Ottawa, Ontario

**COORD. SYS.:** MTM ZONE 9 **EASTING:** 376245.10 **NORTHING:** 5019216.58 **ELEVATION:** 101.47

**PROJECT:** Proposed High-rise development

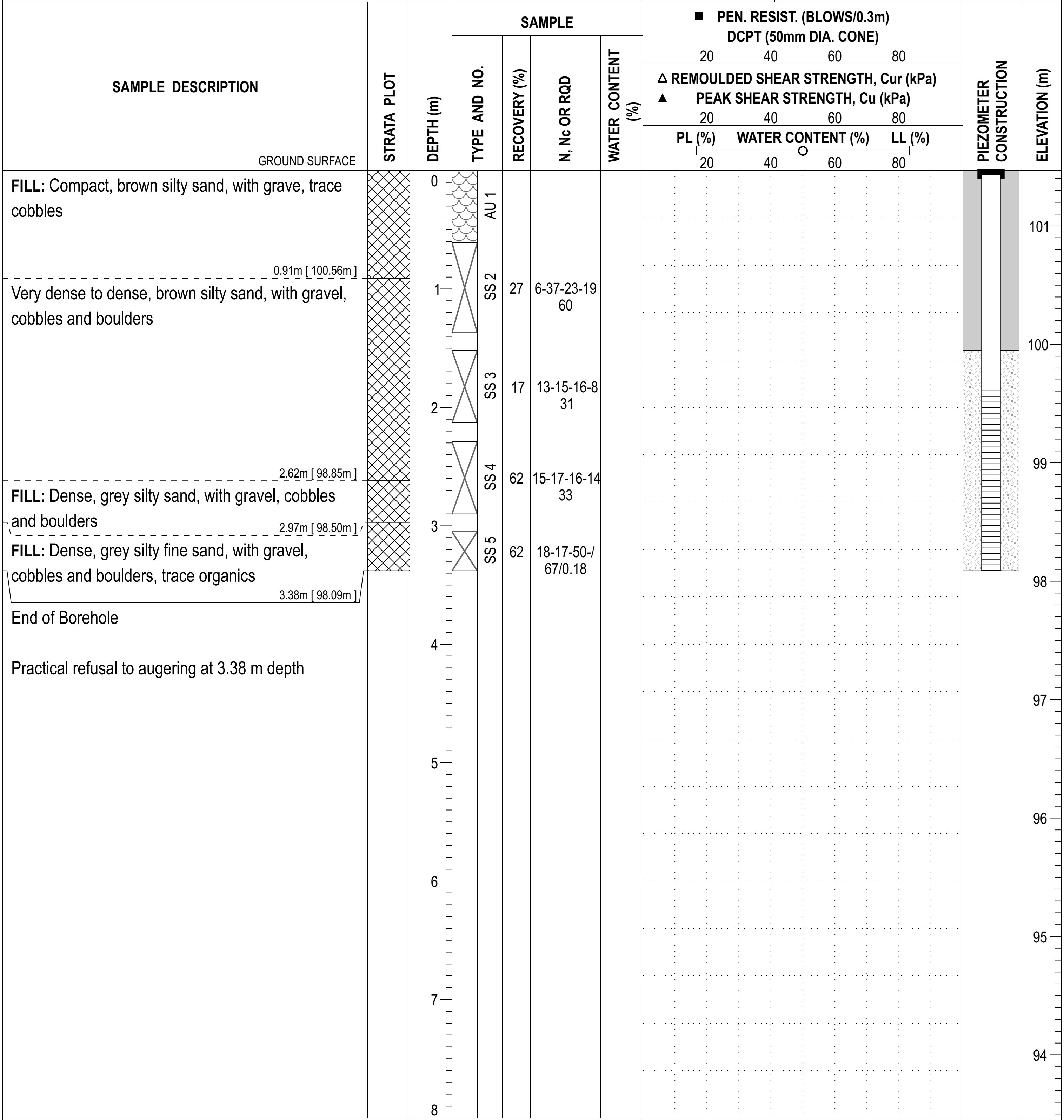
**BORINGS BY:** CME-55 Low Clearance Drill

**REMARKS:**

**DATE:** September 13, 2024

**FILE NO. :** PG2934

**HOLE NO. :** BH 4-24



| SAMPLE DESCRIPTION |
|---|
| GROUND SURFACE |
| **FILL:** Compact, brown silty sand, with grave, trace cobbles |
| 0.91m [ 100.56m ] |
| Very dense to dense, brown silty sand, with gravel, cobbles and boulders |
| 2.62m [ 98.85m ] |
| **FILL:** Dense, grey silty sand, with gravel, cobbles and boulders |
| 2.97m [ 98.50m ] |
| **FILL:** Dense, grey silty fine sand, with gravel, cobbles and boulders, trace organics |
| 3.38m [ 98.09m ] |
| End of Borehole |
| Practical refusal to augering at 3.38 m depth |

| TYPE AND NO. | RECOVERY (%) | N, Nc OR RQD | WATER CONTENT (%) |
|---|---|---|---|
| AU 1 | | | |
| SS 2 | 27 | 6-37-23-19<br>60 | |
| SS 3 | 17 | 13-15-16-8<br>31 | |
| SS 4 | 62 | 15-17-16-14<br>33 | |
| SS 5 | 62 | 18-17-50-/<br>67/0.18 | |

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# SOIL PROFILE AND TEST DATA

## Geotechnical Investigation

4836 Bank Street, Ottawa, Ontario

**COORD. SYS.:** MTM ZONE 9 **EASTING:** 376295.46 **NORTHING:** 5019206.85 **ELEVATION:** 99.77

**PROJECT:** Proposed High-rise development

**BORINGS BY:** CME-55 Low Clearance Drill

**REMARKS:**

**DATE:** September 13, 2024

**FILE NO. :** PG2934

**HOLE NO. :** BH 5-24

**SAMPLE DESCRIPTION**

GROUND SURFACE

**FILL:** Compact, brown silty fine sand, with gravel

0.91m [ 98.86m ]

**GLACIAL TILL:** Dense to very dense brown silty fine sand, with gravel, cobbles and boulders

1.68m [ 98.09m ]

**GLACIAL TILL:** Very dense to dense, grey silty fine sand, with gravel, cobbles and boulders

- Trace clay by 2.29 m depth

- Sand content increasing

5.31m [ 94.46m ]

End of Borehole

Practical refusal to augering at 5.31 m depth

| TYPE AND NO. | RECOVERY (%) | N, Nc OR RQD | WATER CONTENT (%) |
|---|---|---|---|
| AU 1 | | | |
| SS 2 | 100 | 9-12-18-19<br>30 | |
| SS 3 | 67 | 12-27-25-17<br>52 | |
| SS 4 | 100 | 6-18-30-32<br>48 | |
| SS 5 | 51 | 33-50-/-/<br>50/0.05 | |
| SS 6 | 100 | 14-22-29-31<br>51 | |
| SS 7 | 139 | 19-22-26-50<br>48 | |

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

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

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

PIEZOMETER CONSTRUCTION

DEPTH (m): 0 1 2 3 4 5 6 7 8

ELEVATION (m): 99 98 97 96 95 94 93 92

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Prop. Commercial Development - 4836 Highway 31**
**Ottawa, Ontario**

**DATUM**

**REMARKS**

**BORINGS BY** Backhoe

**DATE** 2019 November 5

**FILE NO.** PG2934

**HOLE NO.** TP13

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE | | | | DEPTH (m) | ELEV. (m) | Pen. Resist. Blows/0.3m<br>● 50 mm Dia. Cone<br>○ Water Content % | Piezometer Construction |
|---|---|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | TYPE | NUMBER | % RECOVERY | N VALUE or RQD | | | 20 40 60 80 | |
| Asphaltic concrete 0.15 | | | | | | 0 | | | |
| **FILL:** Brown silty sand with gravel, trace organics 0.75 | | G | 1 | | | | | | |
| **GLACIAL TILL:** Loose to dense, brown silty sand with gravel, cobbles and boulders 2.50 | | G<br>G | 2<br>3 | | | 1<br>2 | | | |
| End of Test Pit<br>(TP dry upon completion) | | | | | | | | | |

20 40 60 80 100

**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 4836 Highway 31
Ottawa, Ontario

**DATUM**

**REMARKS**

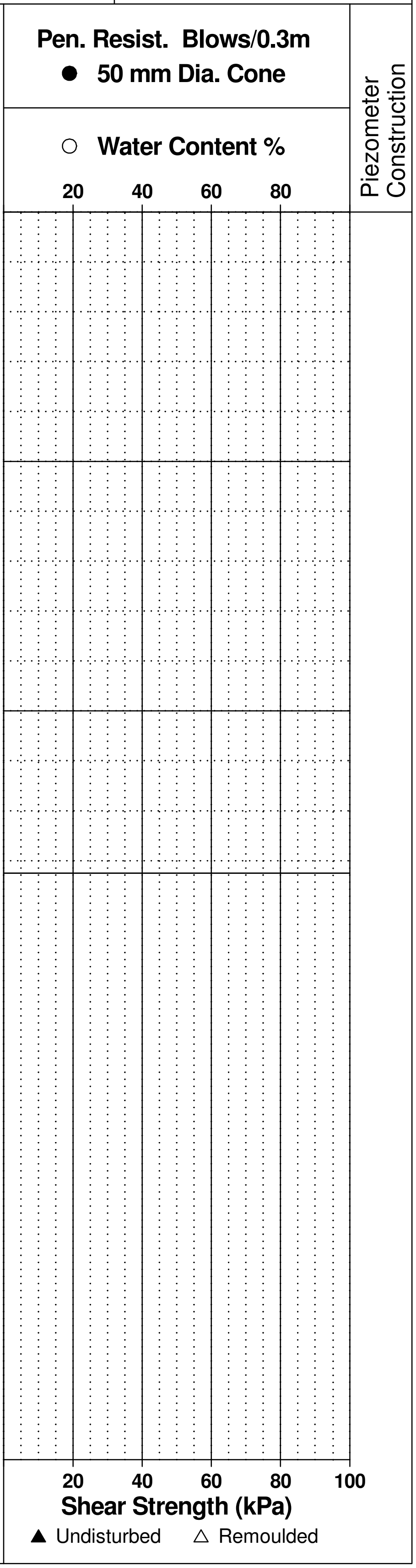
**BORINGS BY** Backhoe

**DATE** 2019 November 5

**FILE NO.** PG2934

**HOLE NO.** TP14

| SOIL DESCRIPTION | | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | | 0 | |
| Asphaltic concrete | 0.10 | | G | 2 | | | | |
| **FILL:** Brown silty sand with gravel, trace organics | 0.60 | | G | 1 | | | | |
| | | | | | | | 1 | |
| **GLACIAL TILL:** Loose to dense, brown silty sand with gravel, cobbles and boulders | | | | | | | 2 | |
| | 2.65 | | G | 3 | | | | |
| End of Test Pit<br>(TP dry upon completion) | | | | | | | | |



**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Prop. Commercial Development - 4836 Highway 31**
**Ottawa, Ontario**

**DATUM**

**REMARKS**

**BORINGS BY** Backhoe

**DATE** 2019 November 5

**FILE NO.** PG2934

**HOLE NO.** TP16

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) | Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone<br>○ Water Content %<br>20 40 60 80 | Piezometer Construction |
|---|---|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | | | |
| **FILL:** Gravel, trace sand | | G | 1 | | | | | | |
| 0.25 | | | | | | | | | |
| **FILL:** Brown silty sand, trace clay and gravel | | G | 2 | | | 1 | | | |
| 1.80 | | | | | | | | | |
| **GLACIAL TILL:** Loose to dense, brown silty sand with gravel, cobbles and boulders | | G | 3 | | | 2 | | | |
| 2.55 | | | | | | | | | |
| End of Test Pit | | | | | | | | | |

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. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

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

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

# SYMBOLS AND TERMS (continued)

## SOIL DESCRIPTION (continued)

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

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

## ROCK DESCRIPTION

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

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

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

## SAMPLE TYPES

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

## SYMBOLS AND TERMS (continued)

### PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

### CONSOLIDATION TEST

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

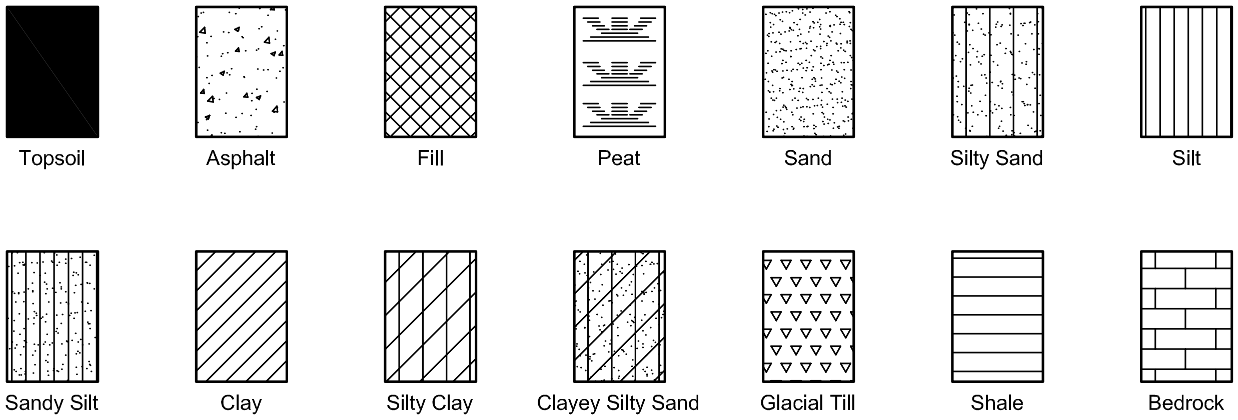
OC Ratio Overconsolidaton ratio = $p'_c / p'_o$
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

### PERMEABILITY TEST

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

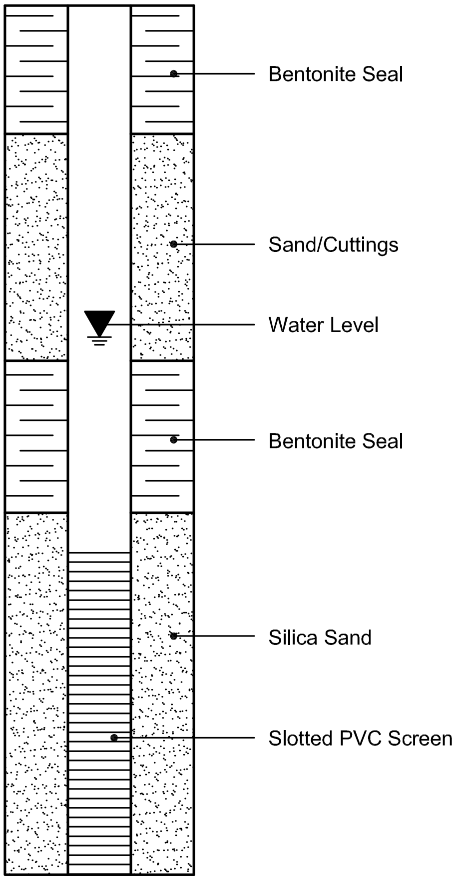
# SYMBOLS AND TERMS (continued)

## STRATA PLOT

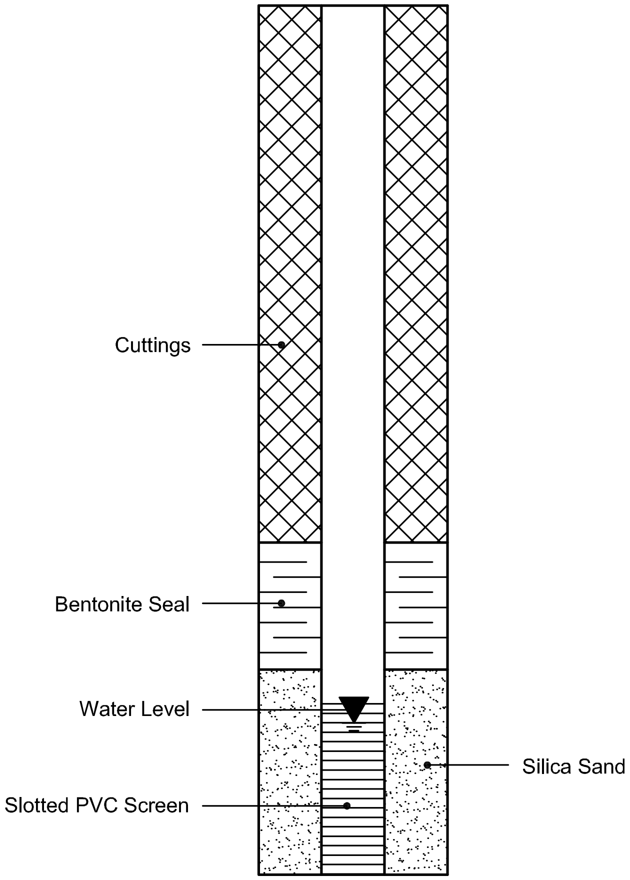


## MONITORING WELL AND PIEZOMETER CONSTRUCTION

### MONITORING WELL CONSTRUCTION



### PIEZOMETER CONSTRUCTION



Certificate of Analysis
**Client: Paterson Group Consulting Engineers**
**Client PO: 26475**

Report Date: 03-Oct-2019
Order Date: 30-Sep-2019
**Project Description: PG2934**

| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | GS1 | - | - | - |
| | **Sample Date:** | 26-Sep-19 09:00 | - | - | - |
| | **Sample ID:** | 1940106-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 89.8 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 6.89 | - | - | - |
| Resistivity | 0.10 Ohm.m | 102 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 6 | - | - | - |
| Sulphate | 5 ug/g dry | 6 | - | - | - |

# APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG2934-1 – TEST HOLE LOCATION PLAN

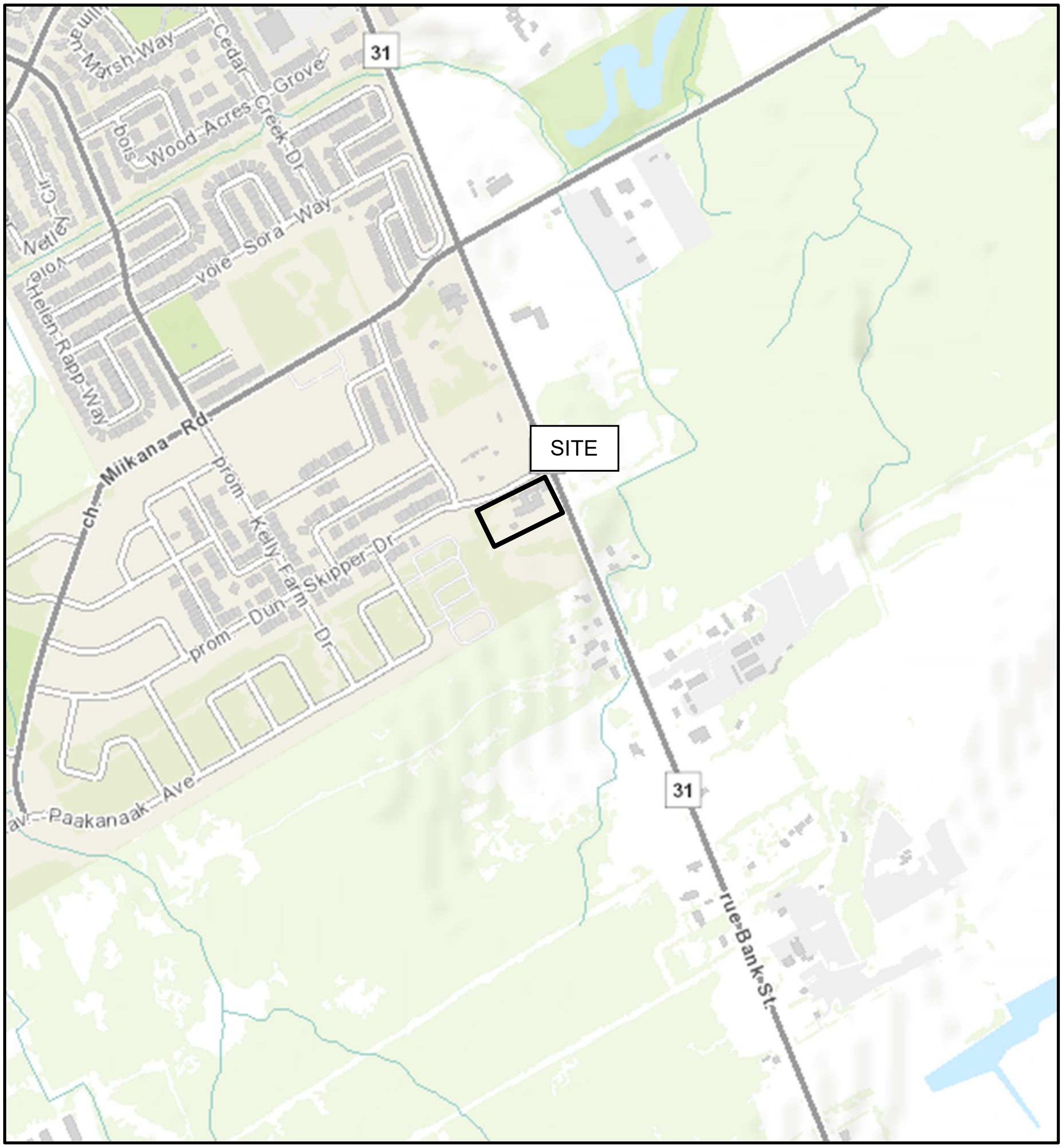
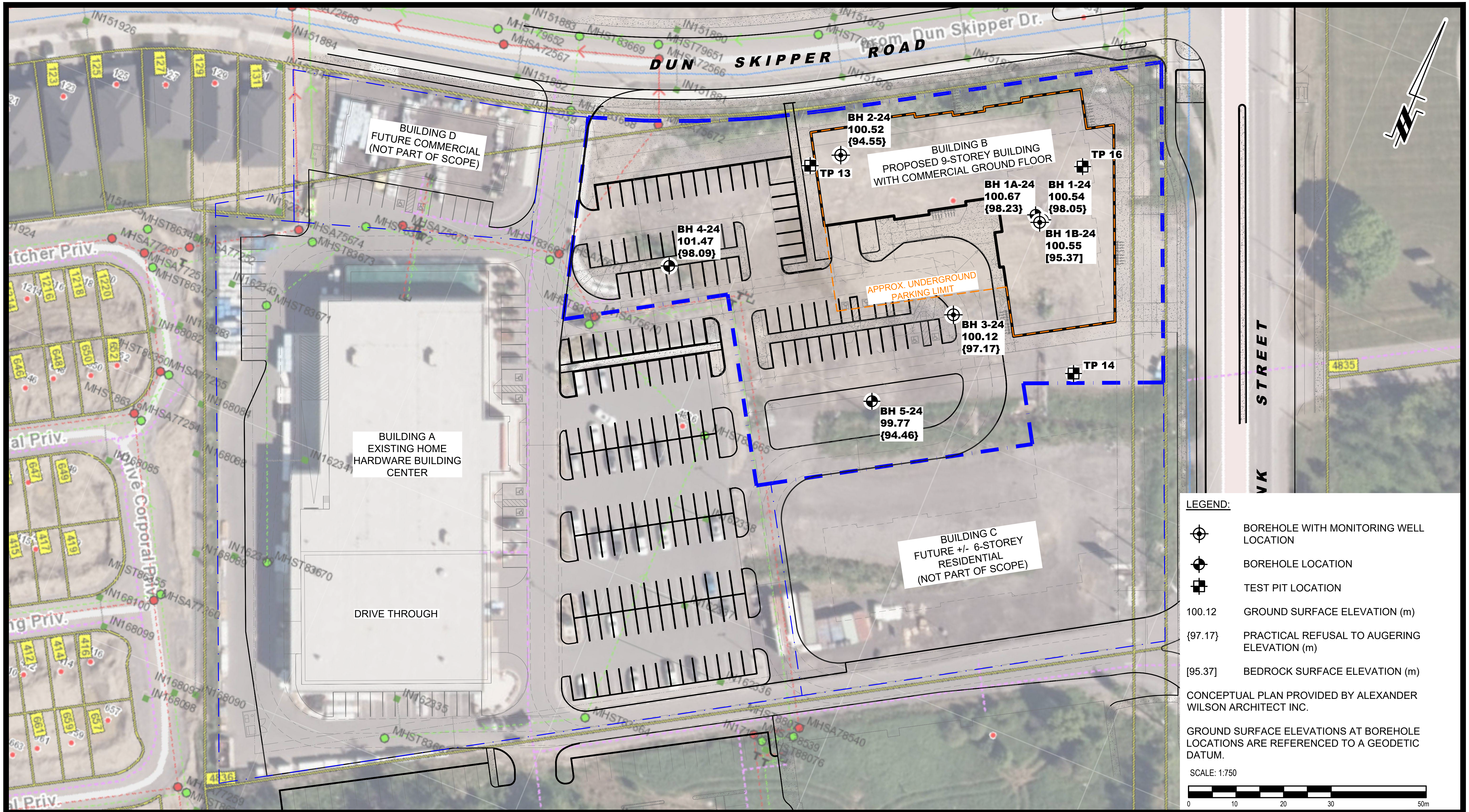


FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

| | |
|---|---|
| ⌖ | BOREHOLE WITH MONITORING WELL LOCATION |
| ⊕ | BOREHOLE LOCATION |
| ⊞ | TEST PIT LOCATION |
| 100.12 | GROUND SURFACE ELEVATION (m) |
| {97.17} | PRACTICAL REFUSAL TO AUGERING ELEVATION (m) |
| [95.37] | BEDROCK SURFACE ELEVATION (m) |

CONCEPTUAL PLAN PROVIDED BY ALEXANDER WILSON ARCHITECT INC.

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

SCALE: 1:750

0 10 20 30 50m

**PATERSON GROUP**

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

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 4 | ADDED 2024 BOREHOLES | 03/10/2024 | SD |
| 3 | UPDATED TO NEW CONCEPTUAL PLAN | 15/08/2024 | SD |
| 2 | UPDATED BASE PLAN AND SITE BOUNDARY | 31/07/2024 | PB |
| 1 | BASE PLAN UPDATED AND NEW TES PITS ADDED | 15/11/2019 | RG |

LEITRIM HOME HARDWARE
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT
4836 BANK STREET

OTTAWA, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

| | |
|---|---|
| Scale: 1:750 | Date: 11/2019 |
| Drawn by: YA | Report No.: PG2934-1 |
| Checked by: PB | Dwg. No.: PG2934-1 |
| Approved by: SD | Revision No.: 4 |

11x17

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