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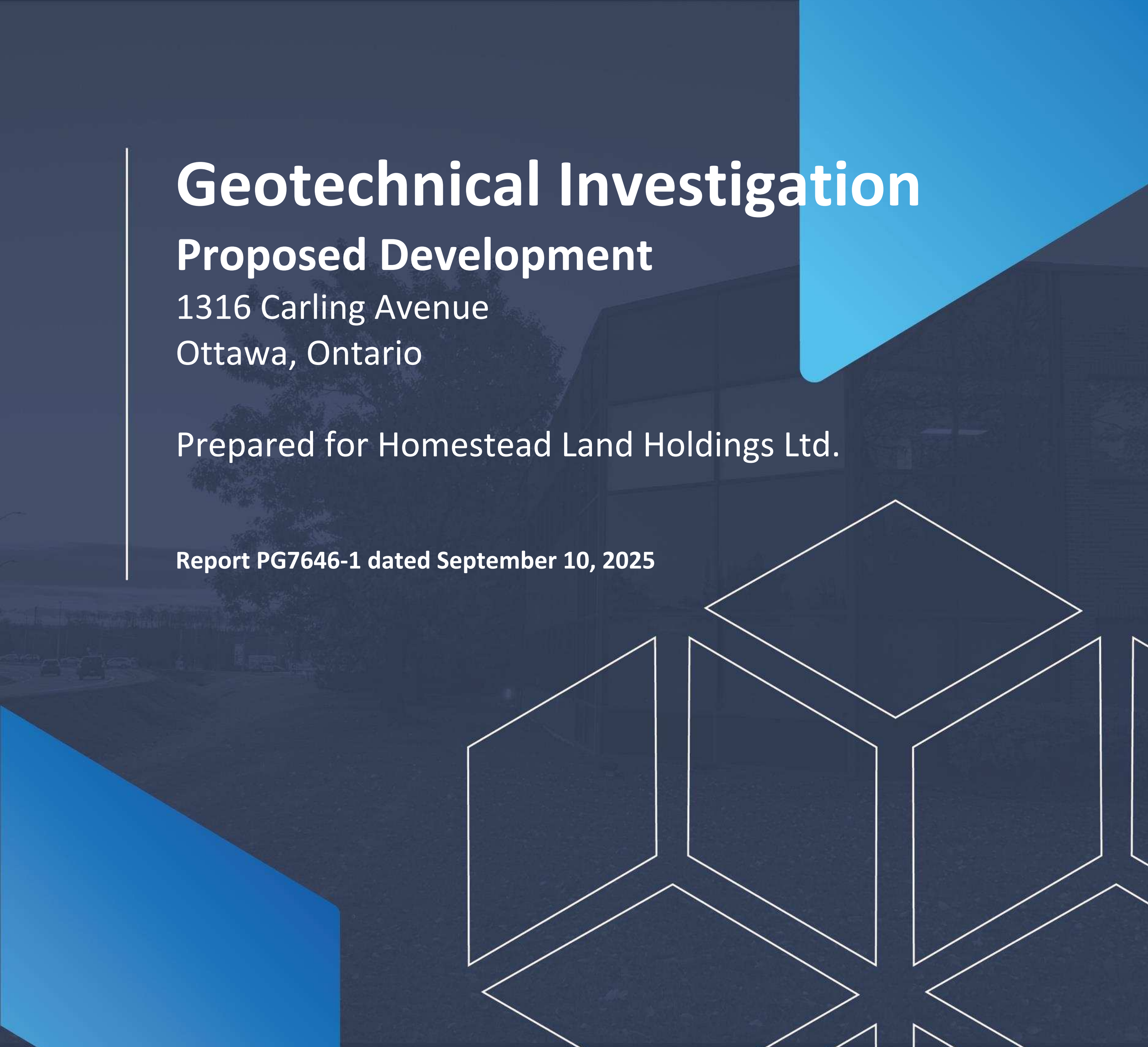

Geotechnical Investigation

Proposed Development

1316 Carling Avenue
Ottawa, Ontario

Prepared for Homestead Land Holdings Ltd.

Report PG7646-1 dated September 10, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Homestead Land Holdings Ltd. to conduct a geotechnical investigation for the proposed development to be located at 1316 Carling Avenue 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, and

☐ Provide geotechnical recommendations pertaining to the 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 information, it is understood that the proposed development will consist of a high-rise residential building with 2 below grade parking levels. At finished grades, it is understood the proposed residential building will be surrounded by asphalt-paved access lanes, parking areas, and walkways with landscaped margins.

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

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The geotechnical investigation was carried out from August 7 to 11, 2025, and consisted of 3 boreholes (BH 1-25 through BH 3-25) advanced to a maximum depth of 19.5 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration existing borehole coverage, underground services, and available access.

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

The approximate locations of the boreholes are shown on Drawing PG7646-1 - Test Hole Location Plan included in Appendix 2.

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

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

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

The subsurface conditions observed in the 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

A groundwater monitoring well was installed in boreholes BH 1-25 and flexible polyethylene standpipes were installed in boreholes BH 2-25 and BH 3-25 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 are presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG7646-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples from the current investigation will be stored in the laboratory for 1 month after completing this report, they will then be discarded 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 during the historic investigation. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing high-rise residential building on the northern portion of the site, fronting onto Carling Avenue, and asphalt-paved access lanes and parking areas across the remainder of the site. Based on available drawings, the existing building has 1 below-grade level is supported on deep foundations.

The subject site is bordered by Carling Avenue to the north, Thames Street to the south, Merivale Road, commercial properties to the northeast and northwest, and residential properties to the southeast and southwest.

The existing ground surface across the subject site is relatively level at an approximate geodetic elevation of 74 m, which is approximately at-grade with Carling Avenue.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of an asphalt pavement structure underlain by fill extending to a depth of about 1.5 to 1.7 m below the existing ground surface. The fill was observed to vary from a compact, brown silty sand with gravel to a very stiff, brown silty clay with trace organics and sand.

Underlying the fill at boreholes BH 2-25 and BH 3-25, a 0.8 to 1.1 m thickness of stiff, brown silty clay was encountered.

A glacial till deposit was encountered underlying the fill and/or silty clay at approximate depths of 1.7 to 2.6 m. The upper part of the glacial till deposit, extending to an approximate depth of 6 to 7 m, was observed to consist of a firm to stiff, grey silty clay with sand and gravel, while the lower part of the glacial till was observed to consist of loose to very dense, brown to grey silty sand to sandy silt with gravel, cobbles and boulders. Coring through numerous boreholes was required to advance the boreholes below an approximate depth of 10 m.

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

Bedrock

Bedrock was encountered underlying the glacial till at depths ranging from 11.7 to 17.3 m below the existing ground surface, generally increasing in depth from south to north across the site.

Bedrock was cored within all boreholes, which was observed to consist of interbedded limestone and dolomite. Based on the RQDs of the recovered rock core, bedrock is generally fair to excellent in quality, generally improving in quality with depth.

4.3 Groundwater

Groundwater levels were measured on September 9, 2025 within the monitoring well and piezometers installed at the borehole locations. The groundwater levels observed are summarized in Table 1 below.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-25	74.31	5.95	68.36	September 9, 2025
BH 2-25	74.35	5.61	68.74	September 9, 2025
BH 3-25	74.58	5.09	69.49	September 9, 2025

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

Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately **4 to 5 m** below the 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. It is recommended that foundation support for the proposed building consist of the following:

- ❒ Within the high-rise footprint, a raft foundation bearing on the undisturbed, compact to very dense glacial till; and
- ❒ For the underground parking levels beyond the high-rise footprint, conventional spread footings bearing on the undisturbed, compact to very dense glacial till.

Numerous boulders were encountered within the glacial till deposit, particularly below a depth of about 10 m. Therefore, the contractor should be prepared for boulders removal, and the presence of boulders should be accounted for in the design of the temporary shoring system.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. This material should be tested and approved prior to delivery. The fill should be placed in a maximum of 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the proposed 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, can be used as general landscaping fill where the settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Protection of Subgrade

Since the footing and raft foundation subgrade material will consist of glacial till, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed, compact to very dense glacial till subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

5.3 Foundation Design

Conventional Spread Footings for Parking Levels beyond High-Rise Building Footprint

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

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

Footings bearing on the undisturbed, compact to very dense glacial till and designed using the bearing resistance values at SLS, given above, will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Raft Foundation for High-Rise Building Footprint

Where the proposed building has two basement levels, the underside of raft foundation is anticipated to be located at a depth of about 7.5 m. The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A maximum bearing resistance value at SLS (contact pressure) of **500 kPa** will be considered acceptable. The loading conditions for the contact pressure are based on sustained loads, generally considered to be 100% Dead Load and 50% Live Load. The factored bearing resistance at ULS can be designed for **750 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **20 MPa/m** for a contact pressure of **500 kPa**. The raft foundation design considers the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

A raft foundation bearing on the undisturbed, compact to very dense glacial till and designed using the parameters provided above will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing- and raft-supported structures are required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the in-situ bearing medium soils when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in-situ soil of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2.0 m** is recommended for grading at this site. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations.

If higher-than-permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post-construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class X_D**. A higher seismic site class, such as Class X_C, may be achievable for this site. However, a site-specific shear wave velocity test is required to accurately determine the applicable seismic site classification for the foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024.

5.5 Basement Slab

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the undisturbed, compact to very dense glacial till is considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

It is understood that the below-grade levels for 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 underslab fill is recommended to consist of 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered during the geotechnical investigation, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the granular fill layer under the lowest level floor slab.

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.

The total earth pressure (P_{AE}) includes the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at the 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.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

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

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

The peak ground acceleration, (a_{max}), for the area of the subject site is 0.366 g for a Site Class X_D according to the OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Design

Rigid Pavement Structure

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 on the next page.

Table 2 – Recommended Rigid Pavement Structure
Lowest 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 lower 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 hours 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 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 *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 *If specified by others, not required from a geotechnical perspective	

Pavement Structure on Overburden Soils

The flexible pavement structure presented in Table 5 should be used for access lanes and heavy loading areas.

Table 5 – Recommended Pavement Structure
Access Lanes and Heavy Loading Area

Thickness (mm)	Material Description
40	**Wear Course –** Superpave 12.5 Asphaltic Concrete
50	**Binder Course –**Superpave 19.0 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – OPSS Granular B Type I or II material placed over in situ soil or engineered fill.	

Minimum performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD using suitable vibratory 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 development. The system should consist of a 100 mm diameter, perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipes 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 installed against the temporary shoring system, extending to a series of drainage sleeve inlets through the building foundation wall at the footing/foundation wall interface. The drainage sleeves should be at least 100 mm in diameter and be spaced at about 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 underslab drainage system should direct water to sump pit(s) within the underground level.

Any pits, such as elevator pits and/or sump pits, should be waterproofed. Additional details for this can be provided upon request.

Foundation Raft Slab Construction Joints

If applicable, it is expected that the raft slab will be poured in sections. For the construction joint at each pour, a rubber water stop, along with a chemical grout (Xypex or equivalent), should be applied to the entire vertical joint of the raft slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Underslab Drainage

Underslab drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 100 mm diameter perforated pipes be placed at approximately 9 m centres underlying the lowest level floor slab. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation, when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available for conventional backfilling, the backfill against the foundation walls within 1.8 m below finished grade should consist of clean sand or OPSS Granular B Type I granular material. Below the frost depth (approximately 1.8 m), the foundation walls can be backfilled with the site excavated fill, provided that this material is maintained in an unfrozen state and at a suitable moisture content for compaction.

6.2 Protection of Footings Against Frost Action

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

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

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. Based on the anticipated depth of excavation and its proximity to the property lines, it is anticipated that a temporary shoring system will be required.

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 the groundwater level. The subsurface soils are considered to be a Type 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.

Temporary Shoring

As noted above, due to the anticipated proximity of the proposed building to the property boundaries and to the existing building to the north, a temporary shoring system is anticipated to be required to support the overburden soils of the adjacent properties. The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer, who is a licensed professional engineer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and includes dewatering control measures.

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

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

The temporary shoring system may consist of a soldier pile and lagging system. Due to the numerous boulders encountered in the glacial till deposit, it is recommended that the soldier piles be drilled as opposed to driven.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described on the following page.

The earth pressures acting on the shoring system may be calculated using the parameters on the next page:

Table 6 – Soil Parameters for Shoring System Design

Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-rest Earth Pressure Coefficient (K_o)	0.5
Total Unit Weight (γ), kN/m^3	210
Submerged Unit Weight (γ'), kN/m^3	13

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

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

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent material specifications and standard detail drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe).

The bedding and cover materials should be placed in a maximum of 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the site-generated fill materials (moist, not wet) above the cover material if excavation and filling operations are carried out in dry and non-freezing weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being reused.

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 a maximum of 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavation will be moderate and controllable using open sumps, potentially requiring well points below the groundwater level. 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.

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Persons as stipulated under O.Reg. 63/16.

Impacts on Neighbouring Properties

It is understood that two levels of underground parking are planned for the proposed building. Due to the presence of relatively shallow glacial till deposit at, and in the vicinity of, the subject site, no adverse effects to neighbouring properties are expected as a result of dewatering which may be required.

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 the founding level.

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

☐ Review of the finalized Grading Plan, from a geotechnical perspective.

☐ Review of the temporary shoring system design, if not prepared by Paterson.

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

☐ Sampling and testing of the concrete and fill materials.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete, including mix design reviews.

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

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Homestead Land Holdings Ltd., 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
Sept. 10, 2025
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Otillia McLaughlin B.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

☐ Homestead Land Holding Ltd.(email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1316 Carling Avenue, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 364775.84 **NORTHING:** 5027546.33 **ELEVATION:** 74.31

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

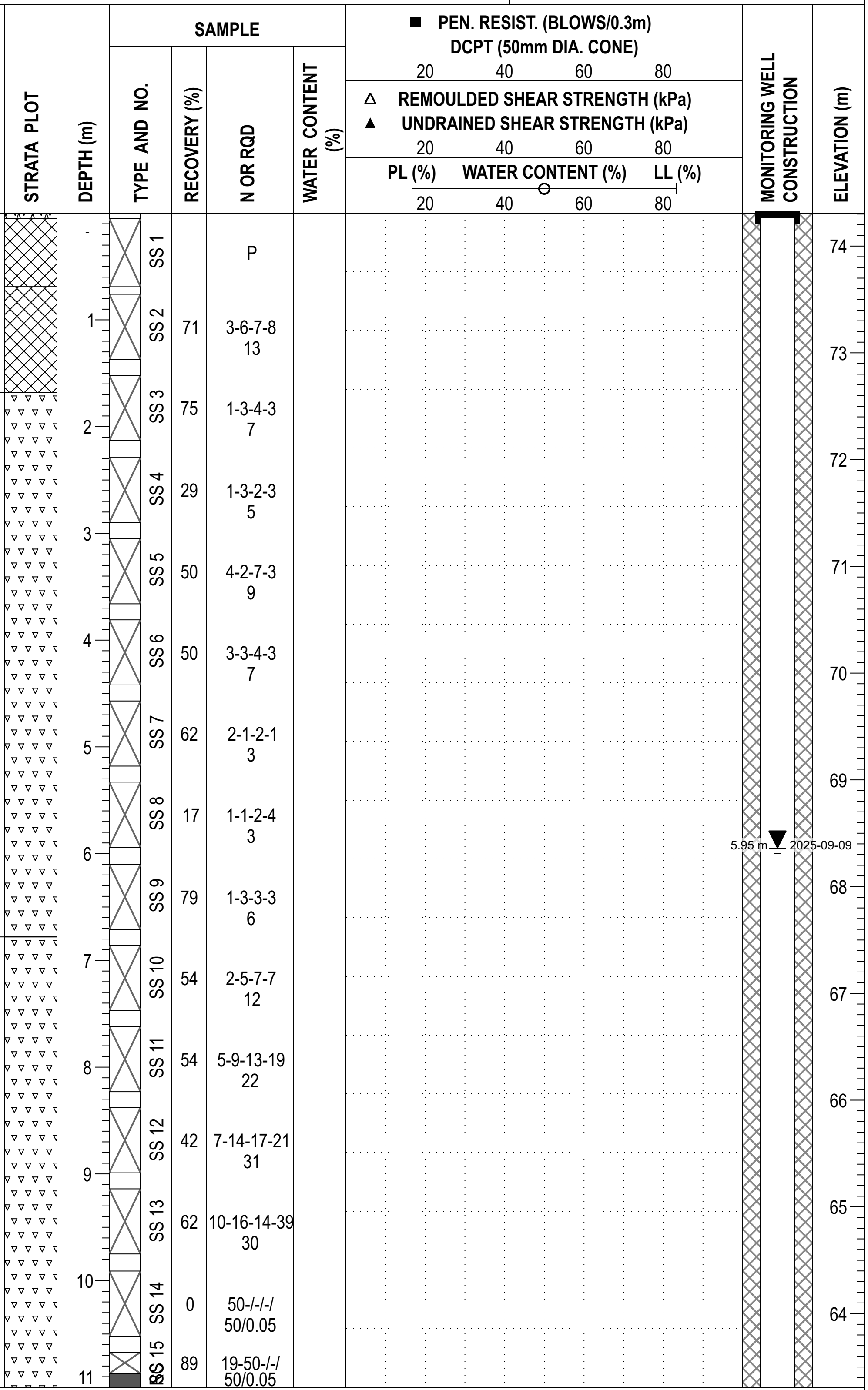
REMARKS: Drilled borehole with augers to 10.7 m, then advanced casing

DATE: August 7, 2025

FILE NO. : PG7646

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE				
ASPHALT 0.05m [74.26m]	SS 1		P	
FILL: Compact, brown silty sand, gravel and crushed stone 0.69m [73.62m]	SS 2	71	3-6-7-8 13	
FILL: Very stiff, brown silty clay, trace organics 1.68m [72.63m]	SS 3	75	1-3-4-3 7	
GLACIAL TILL: Stiff to firm, grey silty clay, crushed stone, sand, trace cobbles and boulders	SS 4	29	1-3-2-3 5	
	SS 5	50	4-2-7-3 9	
	SS 6	50	3-3-4-3 7	
- Soft at 4.5 m depth	SS 7	62	2-1-2-1 3	
- Sand content increasing at 5.3 m depth	SS 8	17	1-1-2-4 3	
6.78m [67.53m]	SS 9	79	1-3-3-3 6	
GLACIAL TILL: Compact to very dense, grey silty sand, to sandy silt, gravel, trace to some clay, cobbles and boulders	SS 10	54	2-5-7-7 12	
	SS 11	54	5-9-13-19 22	
- Clay content decreasing at 8.4 m depth	SS 12	42	7-14-17-21 31	
- Running sand encountered from 9.1 m to 10.7 m depth	SS 13	62	10-16-14-39 30	
	SS 14	0	50-/-/-/ 50/0.05	
	SS 15	89	19-50-/-/ 50/0.05	



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1316 Carling Avenue, Ottawa, ON

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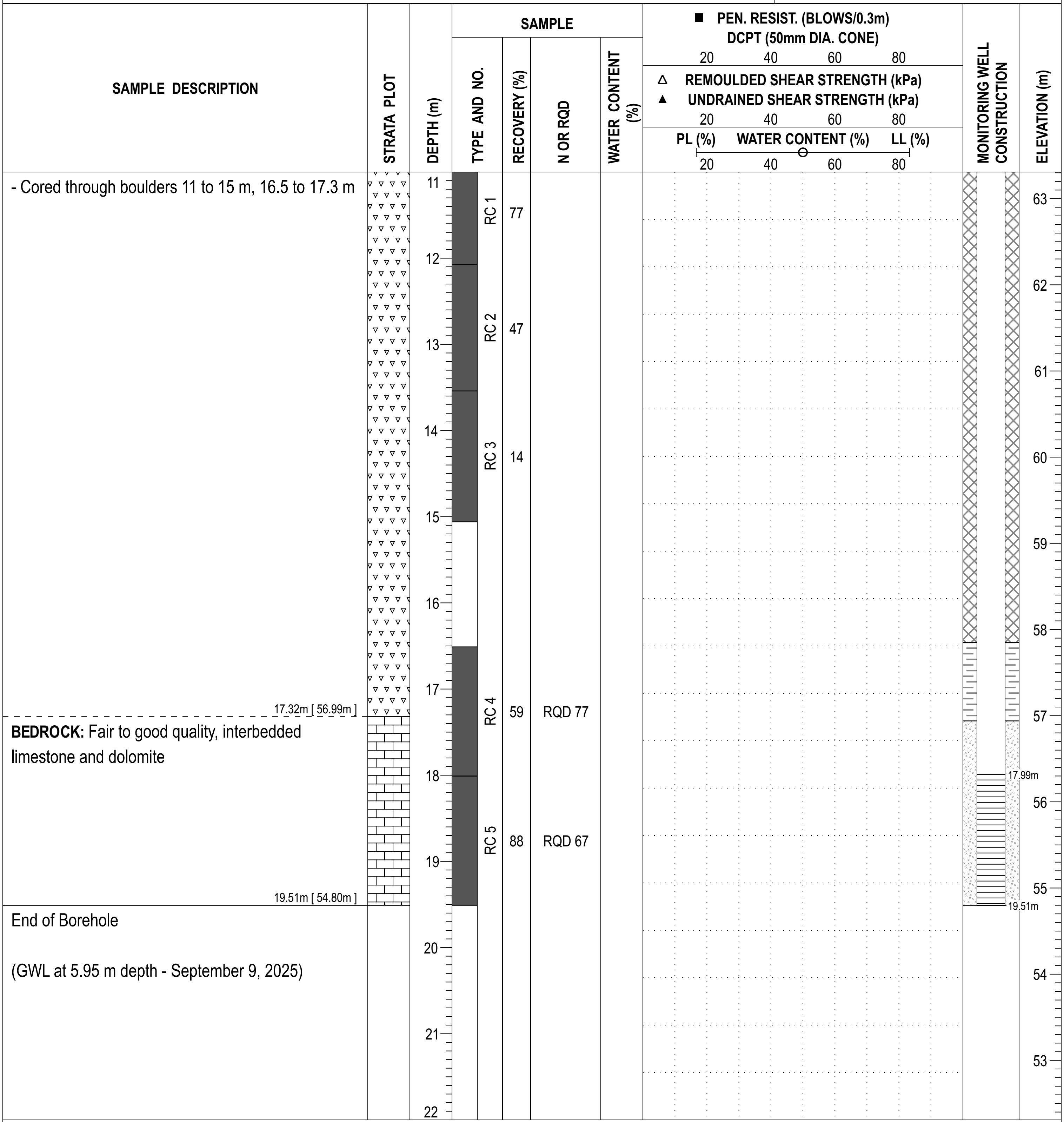


- Cored through boulders 11 to 15 m, 16.5 to 17.3 m

BEDROCK: Fair to good quality, interbedded limestone and dolomite

End of Borehole

(GWL at 5.95 m depth - September 9, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD
RC 1	77	
RC 2	47	
RC 3	14	
RC 4	59	RQD 77
RC 5	88	RQD 67

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1316 Carling Avenue, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 364760.66 **NORTHING:** 5027509.00 **ELEVATION:** 74.35

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

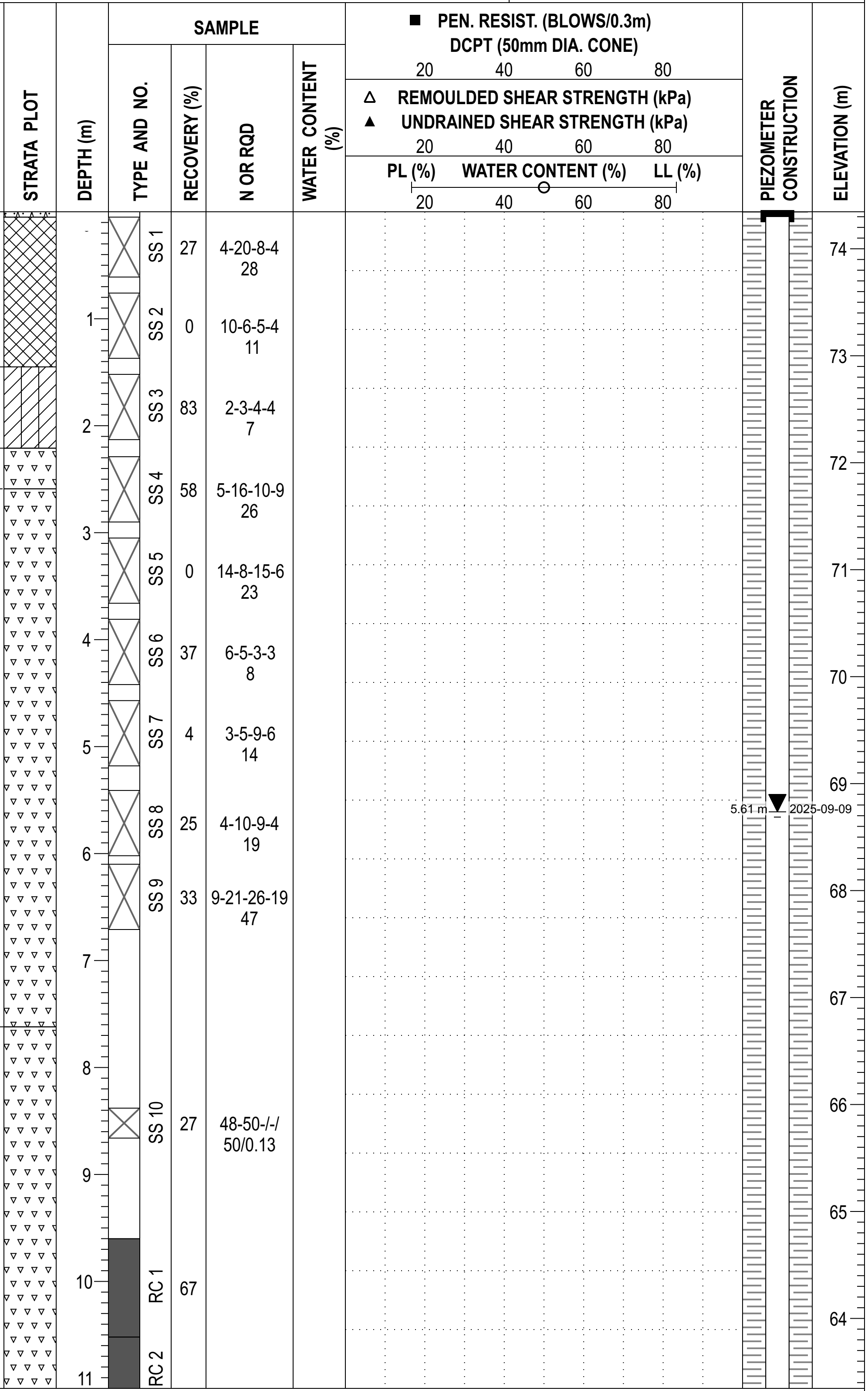
REMARKS: Drilled borehole with casing and wet rotary drilling method. **DATE:** August 11, 2025

FILE NO. : PG7646

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	Base depth [elevation]
GROUND SURFACE	
ASPHALT	0.05m [74.30m]
FILL: Compact, brown silty sand, gravel and crushed stone	1.45m [72.90m]
Stiff, brown **SILTY CLAY**, trace sand	2.21m [72.14m]
GLACIAL TILL: Brown, sand, gravel, trace cobbles and boulders	2.59m [71.76m]
GLACIAL TILL: Loose to very dense, grey sandy silt, gravel, trace cobbles and boulders - Sand content increasing at 4.5 m depth - Rock fragments and clay content decreasing below 5.4 m depth	7.62m [66.73m]
GLACIAL TILL: Compact to very dense, grey silty sand, gravel, trace cobbles and boulders - Cored through boulders 9.5 to 11.8 m	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	27	4-20-8-4 28	
SS 2	0	10-6-5-4 11	
SS 3	83	2-3-4-4 7	
SS 4	58	5-16-10-9 26	
SS 5	0	14-8-15-6 23	
SS 6	37	6-5-3-3 8	
SS 7	4	3-5-9-6 14	
SS 8	25	4-10-9-4 19	
SS 9	33	9-21-26-19 47	
SS 10	27	48-50-/-/ 50/0.13	
RC 1	67		
RC 2			



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1316 Carling Avenue, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 364760.66 **NORTHING:** 5027509.00 **ELEVATION:** 74.35

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS: Drilled borehole with casing and wet rotary drilling method. **DATE:** August 11, 2025

FILE NO. : PG7646

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	ELEVATION (m)
11.81m [62.54m]						
BEDROCK: Fair to good quality, interbedded limestone and dolomite	11.81–16.46	RC 2	100	RQD 63		
		RC 3	100	RQD 94		
		RC 4	93	RQD 75		
		RC 5	97	RQD 77		
16.46m [57.89m]						
End of Borehole (GWL at 5.61 m depth - September 9, 2025)						

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

Elevation scale: 63, 62, 61, 60, 59, 58, 57, 56, 55, 54, 53

■ 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

PIEZOMETER CONSTRUCTION

STRATA PLOT

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1316 Carling Avenue, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 364786.50 **NORTHING:** 5027487.42 **ELEVATION:** 74.58

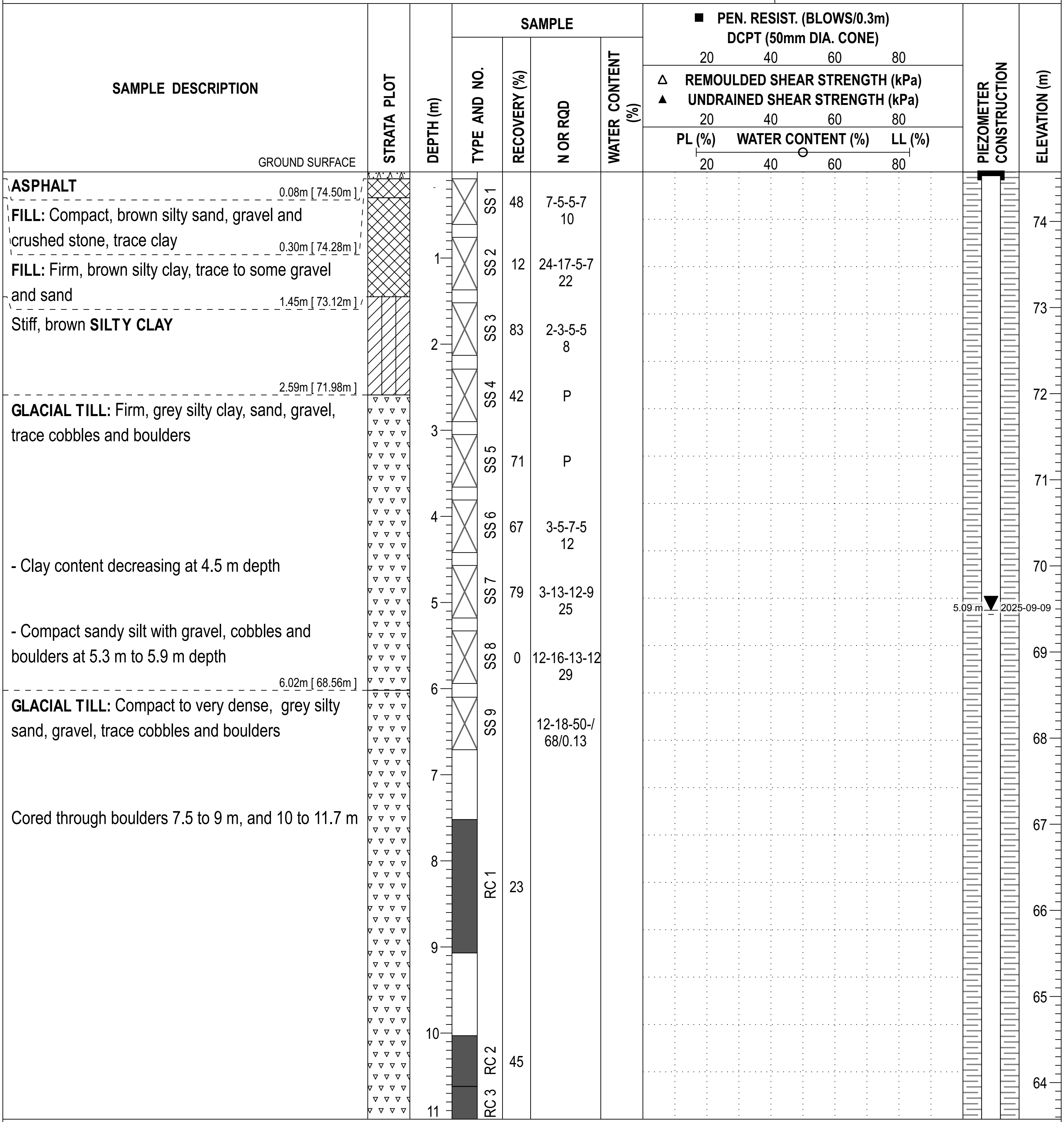
PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS: Drilled borehole with casing and wet rotary drilling method. **DATE:** August 11, 2025

FILE NO. : PG7646

HOLE NO. : BH 3-25



SAMPLE DESCRIPTION	Depth [Elevation]
GROUND SURFACE	
ASPHALT	0.08m [74.50m]
FILL: Compact, brown silty sand, gravel and crushed stone, trace clay	0.30m [74.28m]
FILL: Firm, brown silty clay, trace to some gravel and sand	1.45m [73.12m]
Stiff, brown **SILTY CLAY**	2.59m [71.98m]
GLACIAL TILL: Firm, grey silty clay, sand, gravel, trace cobbles and boulders - Clay content decreasing at 4.5 m depth - Compact sandy silt with gravel, cobbles and boulders at 5.3 m to 5.9 m depth	6.02m [68.56m]
GLACIAL TILL: Compact to very dense, grey silty sand, gravel, trace cobbles and boulders Cored through boulders 7.5 to 9 m, and 10 to 11.7 m	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	48	7-5-5-7 10	
SS 2	12	24-17-5-7 22	
SS 3	83	2-3-5-5 8	
SS 4	42	P	
SS 5	71	P	
SS 6	67	3-5-7-5 12	
SS 7	79	3-13-12-9 25	
SS 8	0	12-16-13-12 29	
SS 9		12-18-50-/ 68/0.13	
RC 1	23		
RC 2	45		
RC 3			

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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1316 Carling Avenue, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 364786.50 **NORTHING:** 5027487.42 **ELEVATION:** 74.58

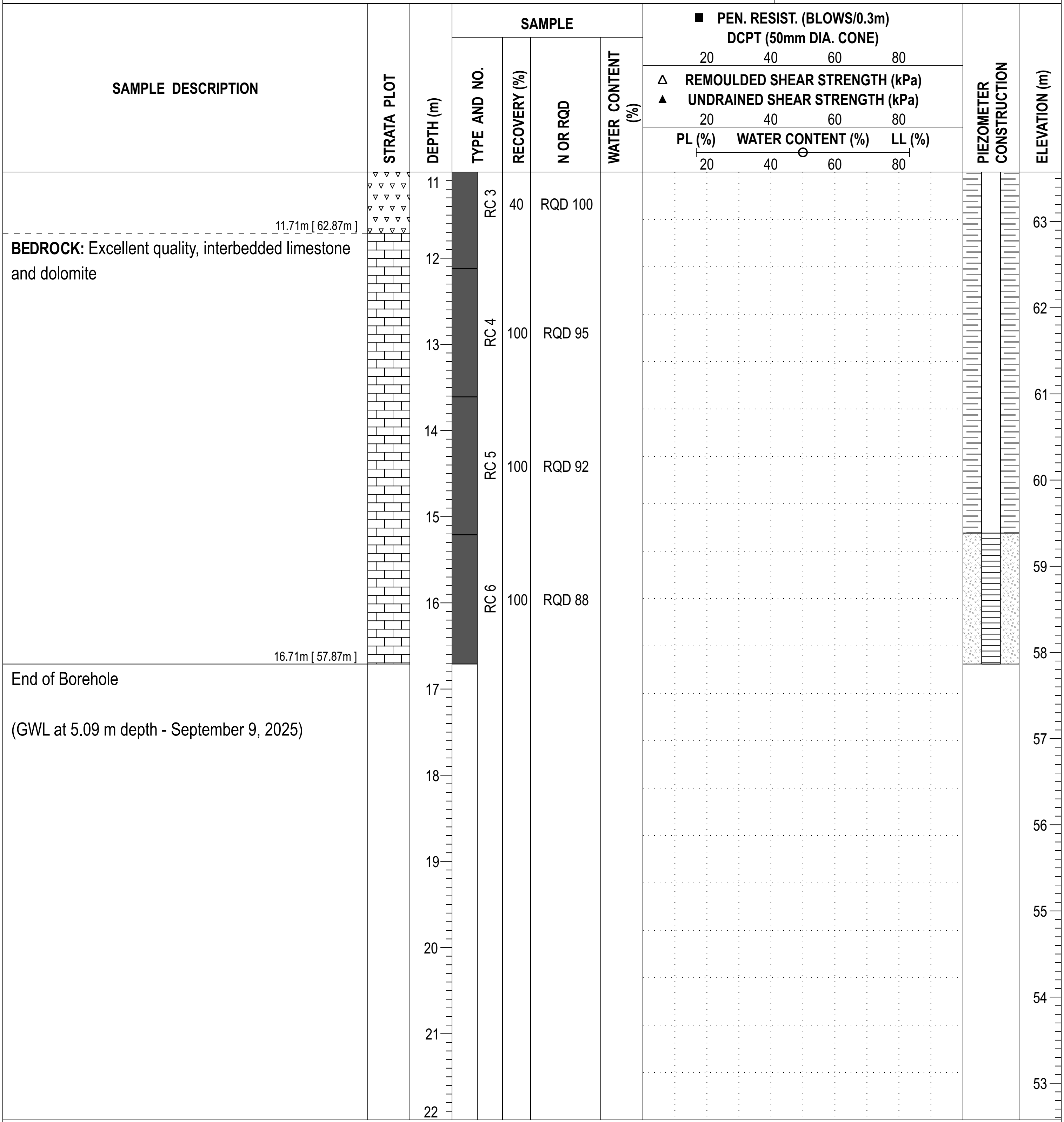
PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS: Drilled borehole with casing and wet rotary drilling method. **DATE:** August 11, 2025

FILE NO. : PG7646

HOLE NO. : BH 3-25



BEDROCK: Excellent quality, interbedded limestone and dolomite

End of Borehole

(GWL at 5.09 m depth - September 9, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD
RC 3	40	RQD 100
RC 4	100	RQD 95
RC 5	100	RQD 92
RC 6	100	RQD 88

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)

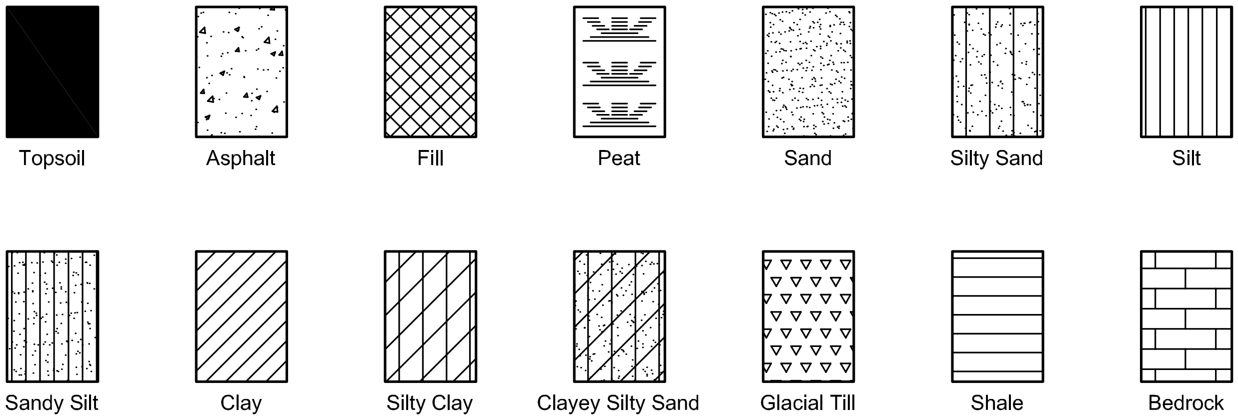
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

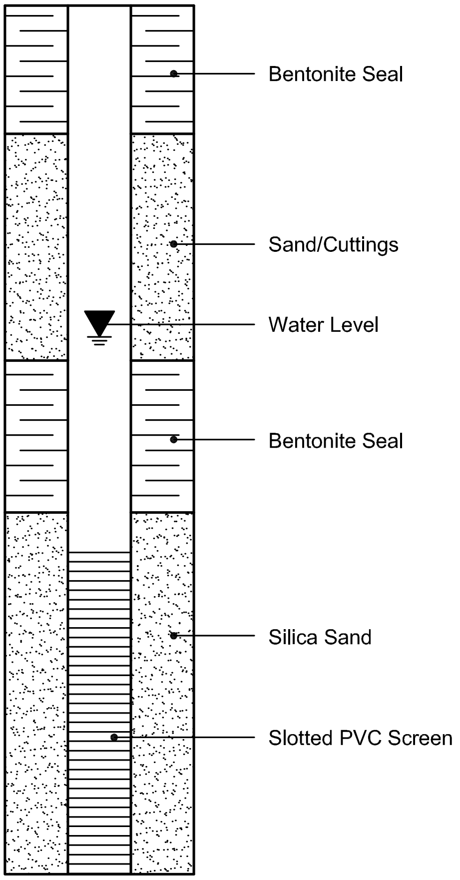
SYMBOLS AND TERMS (continued)

STRATA PLOT

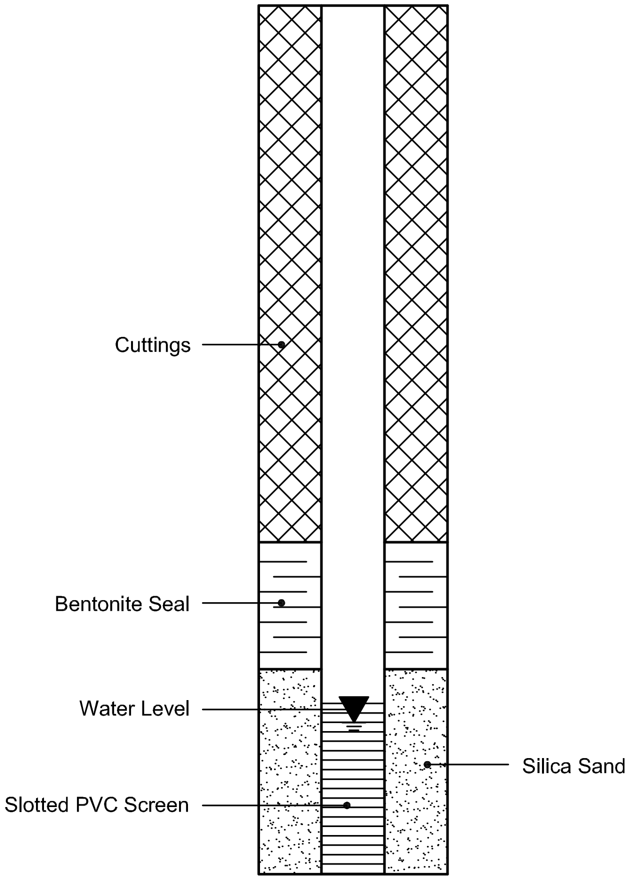


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 20-Aug-2025

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

Order Date: 14-Aug-2025

Client PO:

Project Description: PG7646

	Client ID:	BH3-25-SS5-10'-12'	-	-	-	-	-
	Sample Date:	11-Aug-25 12:00	-	-	-		
	Sample ID:	2533418-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	87.2	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.76	-	-	-	-	-
Resistivity	0.1 Ohm.m	48.8	-	-	-	-	-
Anions							
Chloride	10 ug/g	29	-	-	-	-	-
Sulphate	10 ug/g	34	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG7646-1 – TEST HOLE LOCATION PLAN

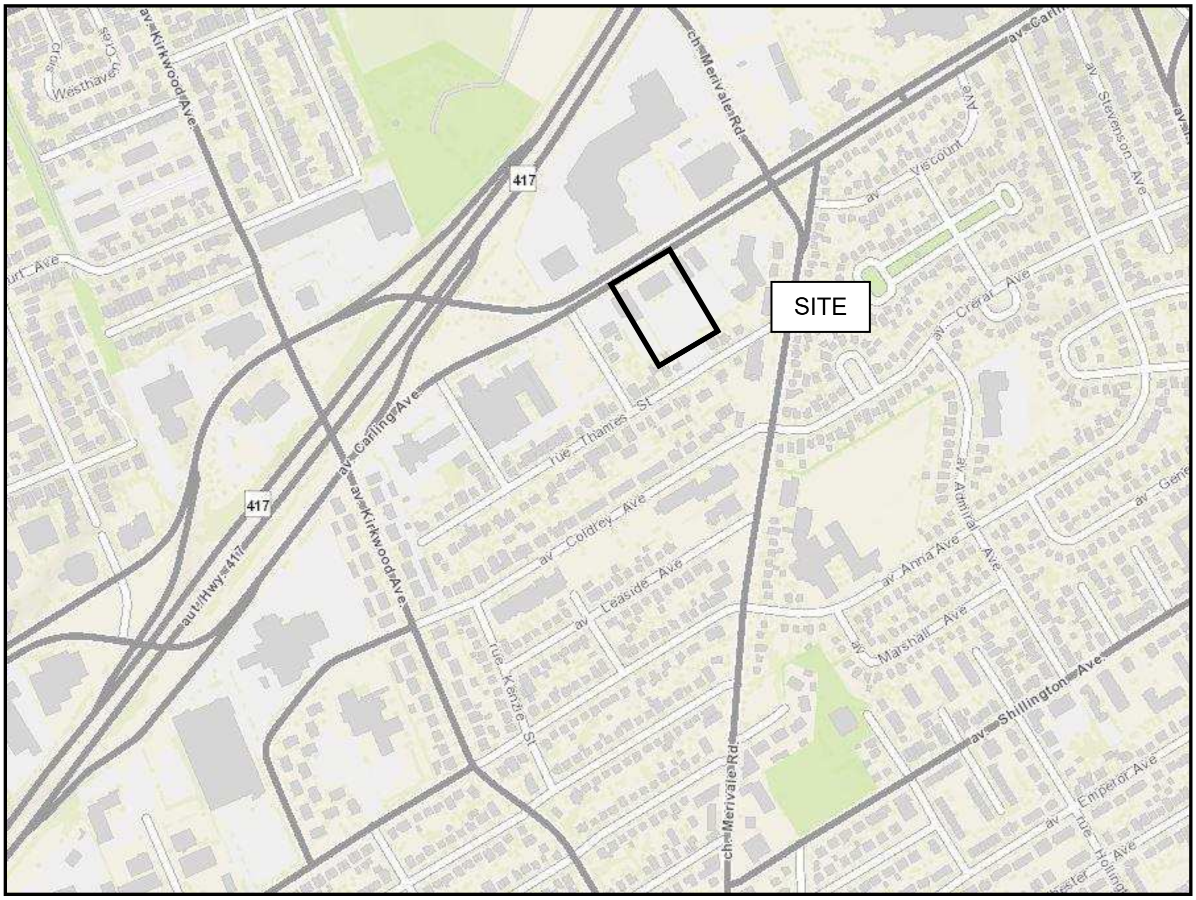
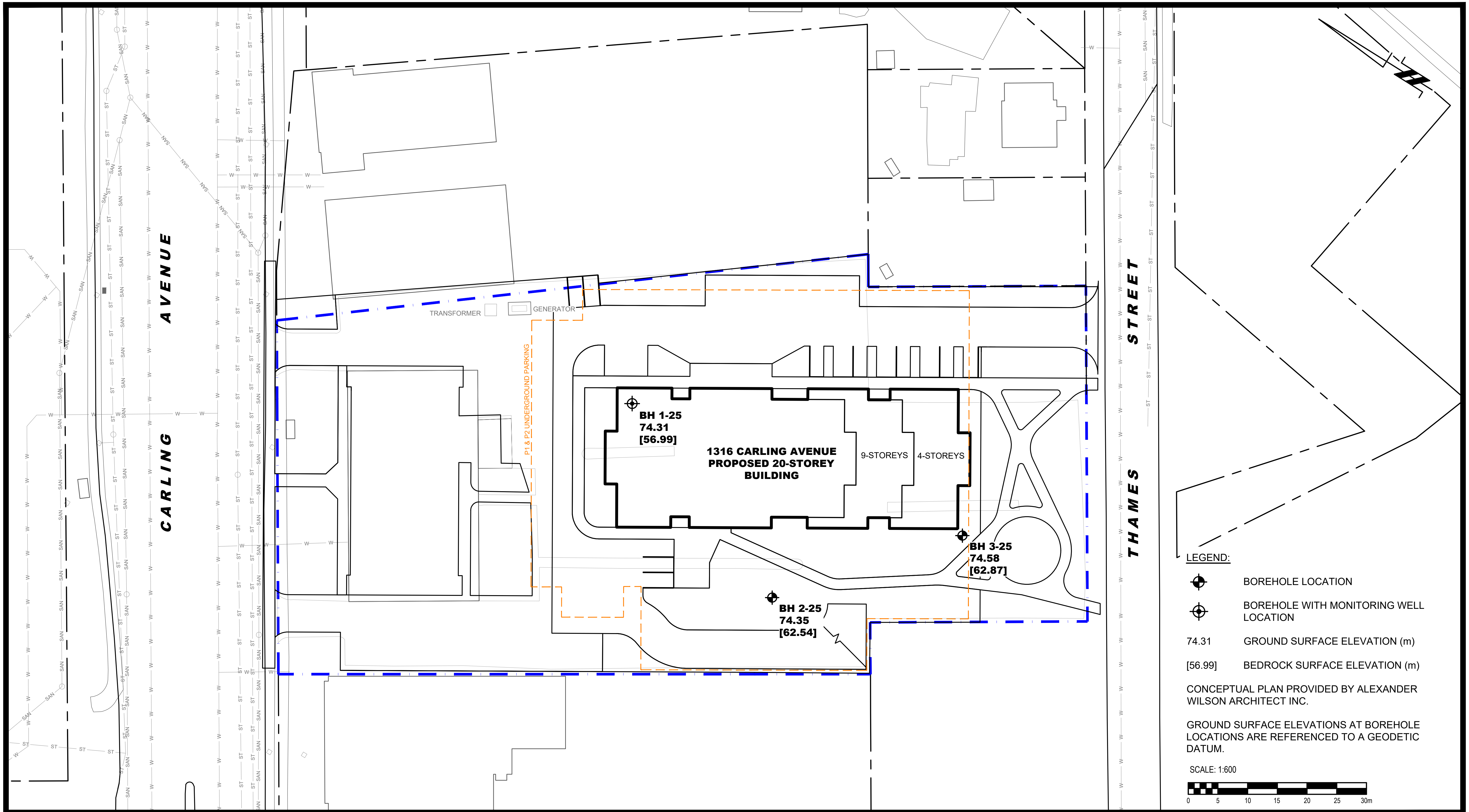


FIGURE 1

KEY PLAN



PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

HOMESTEAD LAND HOLDINGS
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
1316 CARLING AVENUE

OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 08/2025
Drawn by: GK
Report No.: PG7646-1
Checked by: OM
Dwg. No.: PG7646-1
Approved by: SD
Revision No.:

11x17

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