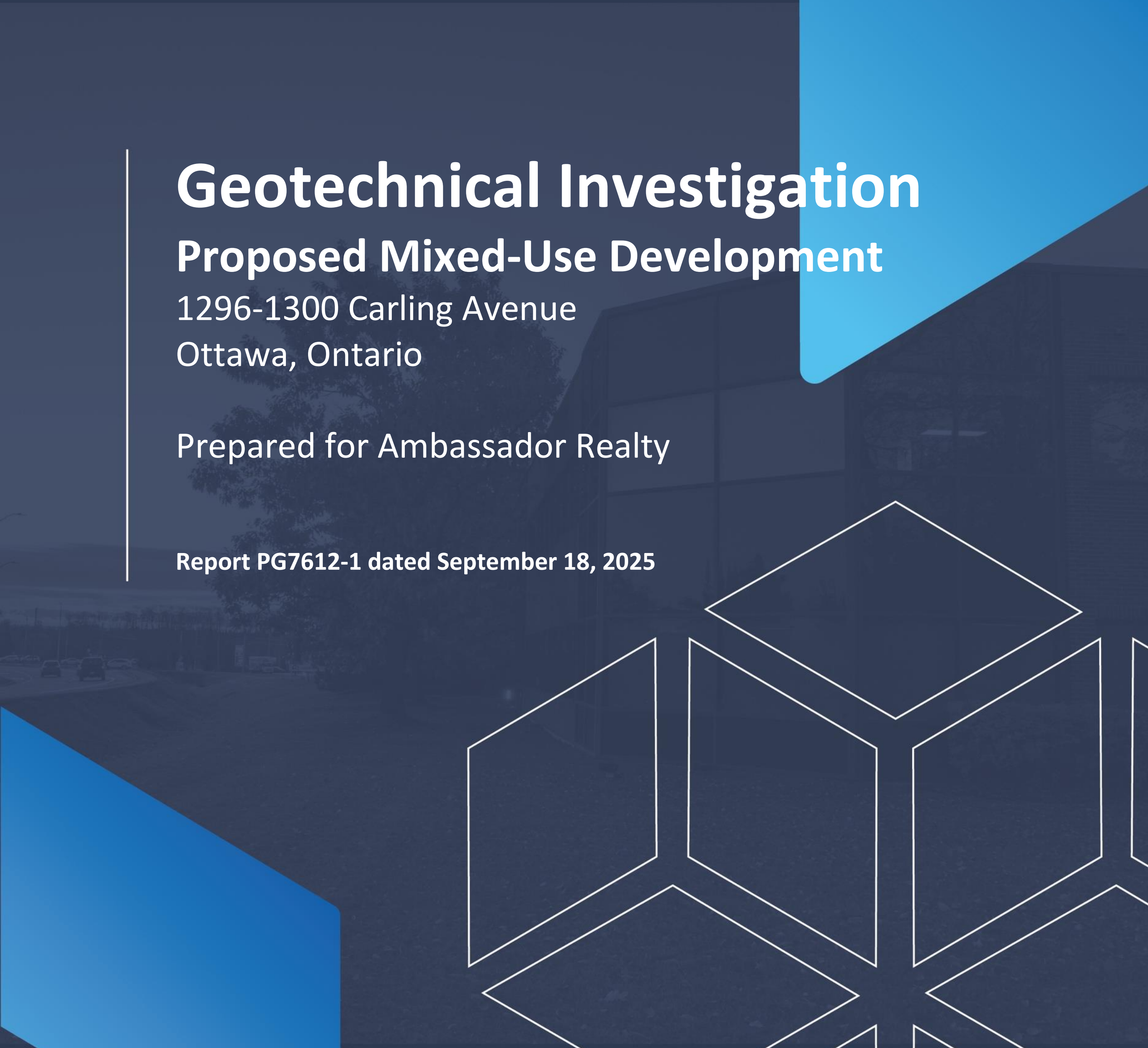

Geotechnical Investigation

Proposed Mixed-Use Development

1296-1300 Carling Avenue
Ottawa, Ontario

Prepared for Ambassador Realty

Report PG7612-1 dated September 18, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ambassador Realty to conduct a geotechnical investigation for the proposed mixed-use development, to be located at 1296-1300 Carling Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

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

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

2.0 Proposed Development

Based on the current concept drawings, it is understood that the proposed development will consist of a high-rise mixed-use building, whose footprint will occupy the majority of the site. It is understood that the building will be provided with two to three, underground levels, however, further details about the project are not available at this time.

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

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out between July 18 to July 22, 2025, and consisted of a total of six (6) boreholes advanced to a maximum depth of 21.1 m below the existing ground surface. The test holes were placed in a manner to provide general coverage of the subject site, taking into consideration site features and underground utilities. The approximate locations of the boreholes are shown on Drawing PG7612-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

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

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

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 overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 2-20. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Groundwater monitoring wells were installed in BH3-25 and BH6-25 and a flexible polyethylene standpipe was installed within BH2-25 & BH4-25, to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

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 PG7612-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

All samples were submitted for moisture content testing. The test results are included on the Soil Profile and Test Data sheets presented in Appendix 1. The results of the moisture contents are presented in the Soil Profile and Test Data Sheets in Appendix 1.

Grain size analyses were conducted on three (3) soil samples recovered during the field investigation from boreholes BH2-25, BH3-25 and BH6-25. Mechanical (i.e. sieve and wash sieve) and/or Hydrometer test methods were used to determine the grain size distribution of each sample. The results of the grain size analyses are presented on the following section and on the Grain Size Distribution sheets in Appendix 1.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is occupied by two (2) three-storey buildings with a basement level along the north side of the site and an asphalt paved parking area over the remaining portion of the site. The grading across the subject site is gently sloped down towards the north side of the site and at grade with surrounding streets.

The site is bordered by Carling Avenue to the north, a four-storey commercial building to the east, a multi-storey residential building to the west and two-storey residential building to the south.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of asphaltic concrete underlain by fill. The fill was generally observed to consist of a brown silty clay with some sand and gravel and /or brown silty sand with some clay and gravel.

A very stiff to firm brown silty clay deposit was generally encountered underlying the fill in boreholes BH3-24, BH4-25 and BH5-25. Some sand and traces of gravel was encountered in the silty clay deposit within BH4-25.

A glacial till deposit comprised of hard to firm brown / grey silty clay, with variable amounts of sand and gravel was encountered at BH1-25 and BH4-25, underlying the fill / silty clay.

A glacial till deposit comprised of stiff brown clayey silt with some sand and trace gravel was encountered underlying the silty clay deposit within BH 3-25.

A glacial till deposit comprised of loose to very dense grey silty sand with variable amount of clay, gravel, cobbles and boulders was generally encountered within all the boreholes underlying the fill / silty clay / glacial till.

Shallow refusal on boulders were encountered on several boreholes within the glacial till deposit. Within BH2- 25 and BH4-25, intermittent refusals were encountered in the glacial till layer, where the boreholes were advanced further by coring through the boulders.

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

Grain Size Distribution and Hydrometer Testing Results

Grain size distribution (sieve and hydrometer analysis) was completed on three (3) representative soil samples within the boreholes. BH2-25 & BH6-25, were sampled within the glacial till layer and BH3-25 was sampled within the silty clay layer. The results of the grain size analysis are summarized in Table 1 and are presented on the Laboratory Testing Results by Others in Appendix 1.

Table 1 – Summary of Grain Size Distribution Analysis

Borehole	Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 2-25	SS11	7.9-8.5	19.3	48.7	26	6
BH 3-25	SS3	1.5-2.1	0	5.9	39.1	55
BH6-25	SS6	3.8-4.4	15.7	40.2	35	9

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone with shaly partings of the Ottawa Formation with an overburden drift thickness ranging from 20 to 26 m.

4.3 Groundwater

The groundwater levels observed are summarized in Table 2 below.

Table 2 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 2-25	74.06	4.93	69.13	July 30, 2025
BH 3-25*	74.03	3.62	70.41	July 30, 2025
BH 4-25	73.68	5.91	67.77	July 30, 2025
BH 6-25*	73.98	4.21	69.77	July 30, 2025

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

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

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. Based on the subsurface conditions encountered in the test holes and the anticipated building depth and loads, it is recommended that the building foundation be comprised of a raft foundation placed over an undisturbed glacial till. As an alternate to raft foundation, drilled piles or micropiles end bearing on bedrock can be considered, however, due to the dense bouldery till underlying the site, pile driving is expected to be complicated. Multiple boulders were encountered during the drilling program. The contractor should be ready to remove any boulders encountered during the excavation and shoring of the site.

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 and other settlement sensitive structures.

Any boulders and loose sand should be sub-excavated from under the bearing surfaces and replaced with clean imported granular fill, as discussed below.

Fill Placement

Fill used for grading beneath the proposed 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. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified site excavated soil can be used as general landscaping fill where 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 these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD.

Site excavated soils are not suitable for use as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

Protection of Subgrade (Raft Foundation)

Where a raft foundation is utilized, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed 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.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay or glacial till to potential disturbance due to drying.

Compact Granular Fill Working Platform for Pile Installations

Should proposed structures be supported on a pile foundation, the use of heavy equipment would be required to install the piles (i.e., pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site freely without sinking and causing significant disturbance to the underlying soil.

A typical working platform could consist of 600 mm of OPSS Granular B, Type II crushed stone placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in maximum 300 mm thick lifts. However, Paterson should review the size and expected load of equipment to be used prior to arrival at the site.

Once the piles have been driven and cut off, the working platform can be re-graded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for basement slab structure. Alternatively, the material can be reused or repurposed to build up the subgrade in the proposed right of way (ROW) or landscaping areas.

5.3 Foundation Design

Foundation Option 1: Raft Foundation

The building's foundation may consist of a raft foundation bearing on undisturbed glacial till.

For 3 levels of underground parking, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between 10 to 11 m. The contact pressure provided considers the stress relief associated with the soil removal required for 3 levels of underground parking.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For 3 levels of underground parking approximately at a depth of 10 to 11 m below ground surface, a bearing resistance value at SLS (contact pressure) of **500 kPa** will be considered acceptable for a raft supported on the undisturbed glacial till. It should be noted that the weight of the raft slab and everything above must be included when designing with this value. The factored bearing resistance (contact pressure) at ULS can be taken as **750 kPa**. For this case, the modulus of subgrade reaction was calculated to be **20.0 MPa/m** for a contact pressure of **500 kPa**.

For 2 levels of underground parking approximately at a depth of 7 to 8 m below ground surface, a bearing resistance value at SLS (contact pressure) of **450 kPa** will be considered acceptable for a raft supported on the undisturbed glacial till. It should be noted that the weight of the raft slab and everything above must be included when designing with this value. The factored bearing resistance (contact pressure) at ULS can be taken as **650 kPa**. For this case, the modulus of subgrade reaction was calculated to be **18.0 MPa/m** for a contact pressure of **450 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Settlement

The total and differential settlement will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 to 20 mm, respectively.

Foundation Option 2: End Bearing Piled Foundation

Pile driving in the dense bouldery till is expected to be complicated. The contractor should be ready to encounter and drill through boulders. The bedrock surface is estimated to be located below 21 m or lower, while the foundation for the development is anticipated at a depth of 7 to 11 m below the existing ground surface.

It is proposed to use concrete filled steel pipe piles driven to refusal on the bedrock surface. A driving shoe is to be used to help with the soil conditions on site. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 3. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended.

This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 3 – Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance	
		SLS (kN)	Factored at ULS (kN)
245	10	975	1,460
245	12	1,100	1,650
245	13	1,175	1,760

As a minimum, the pipe piles should be equipped with a base plate having a thickness of at least 20 mm to reduce potential damage to the pile tip during driving. The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter.

The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Due to the presence of boulder pile driving could present as challenging, the installation method is the responsibility of the contractor to ensure all specification of the project are met.

Settlement

Footings placed on piles extending to the bedrock are subject to negligible post construction total and differential settlements.

Foundation Option 3: Micro Piles

Micropiles are small diameter, deep foundation element composed of a high strength steel casing and a high yield threaded steel bar core. The piles are advanced by rotary drills and normally develop their strength within the underlaying rock layer. The steel elements are grouted in place using non shrink grout in a similar fashion to rock anchors. Pressure grouting can be used to achieve greater bond strengths. The geotechnical resistance presented in Table 4 below can be used for preliminary purposes, based on the available geotechnical information.

A structural engineer should review the buckling potential, lateral capacity and other structural elements of the micropile design.

Table 4 – Micropile Foundation Design Data

Casing Size (mm)	Threaded Bar Size (mm)	Cased Length (m)	Bond Length (m)	Compression Capacity (kN)	Tension Capacity (kN)
150	43	3	3	700	550
175	43	3	3	800	600
250	57	3	4	1,500	1,000

Since micropiles are considered permanent for a life span of 100 years, double corrosion and/or sacrificial steel corrosion allowance are to be considered for the threaded bar and steel casing. It is recommended that the micropile pile casing have a minimum wall thickness of 12 mm with a minimum of 3 mm of sacrificial steel. The minimum centre-to-centre pile spacing is 2.5 times the pile diameter.

Settlement

Footings bearing on an acceptable bedrock surface or pile foundation will be subjected to negligible post construction total and differential settlements.

Conventional Spread Footings

Conventional spread footing could be required for accessory structures or other standalone columns.

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 **275 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **400 kPa**. A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance value at ULS.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over stiff to very stiff silty clay layer can be designed using a bearing resistance value at serviceability limit states (SLS) of **125 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 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.

Settlement

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to a silty clay or glacial till bearing medium 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 or engineered fill of the same or higher capacity as the soil.

5.4 Design for Earthquakes

The site designation for seismic site response can be taken as **Class X_D** for the foundations anticipated at the subject site.

If a higher seismic site class or a specific site velocity is required (Class X_{VS}), a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024.

5.5 Basement Slab

Multi-Storey Buildings

Where raft foundations are utilized, a sub-slab granular layer of OPSS Granular A crushed stone will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

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 under-slab fill is recommended to consist of 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered during the geotechnical investigation, an under-slab 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.

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.

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

a_c = $(1.45 - a_{max}/g)a_{max}$
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 forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

5.7 Pavement Design

Rigid Pavement Structures

For design purposes, it is recommended that the rigid pavement structure for the lower underground parking level of the proposed building 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 5 below.

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 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 5 – Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
125	**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 concrete raft slab or in situ soil.	

Flexible Pavement Structure

The flexible pavement structure presented in Table 6 & 7 should be used for access lanes and heavy loading areas.

Table 6 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 7 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Consideration should also be given to installing subdrains during the pavement construction as per City of Ottawa standards. These drains should extend in four orthogonal directions or longitudinally when placed along a curb.

The clear crushed stone surrounding the drainage lines, or the pipe should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines. Discharge of the subdrains should be directed by gravity to storm sewers or deeper drainage ditches.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

For the proposed underground parking levels of the high-rise buildings, it is expected that the building's 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 board and waterproofing membrane fastened to the temporary shoring system.

Also, an interior perimeter foundation drainage system will be required as a secondary system to account for any groundwater, which breaches the primary ground infiltration control system.

The foundation drainage system should extend from the underside of footing elevation to the proposed finished grade and the following is suggested for preliminary design purposes:

- ☐ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, against the temporary shoring surface. The composite drainage layer should extend from finished grade to underside of footing level.
- ☐ Place a suitable waterproofing membrane, such as a Colphene BSW or NaturaSeal SpraySeal NS F300 or Hygrothane SwiftProof M 200 or equivalent over the composite drainage board surface. The membrane liner should extend from finished grade to underside of footing level.
- ☐ Pour foundation wall against the waterproofing membrane.

Reference should be made to Figure 2 – Foundation Drainage System, presented in Appendix 2.

It is recommended that 150 mm diameter sleeves at 3-6 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

It is important to note that the building's sump pit and elevator pit be fully waterproofed. A detail can be provided by Paterson once the design drawings are available for the elevator and sump pits.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the lowest level floor slabs. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximately 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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

6.2 Protection of Footings Against Frost Action

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

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

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages.

Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given the proximity of the underground parking levels to the property lines, it is expected that a temporary shoring will be required to support the excavation for this proposed development.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

Temporary Shoring

Temporary shoring will 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 these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

The geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system.

The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that precipitation will not negatively impact the shoring system, or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary shoring system is recommended to consist of soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Generally, it is expected that the shoring systems will be provided with tie-back anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated with the parameters provided on Table 8 below.

Table 8 – Soils Parameter 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
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level. The hydrostatic groundwater pressure should be included 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 to 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, should be placed from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

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

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

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

Groundwater Control for Building Construction

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

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

6.6 Winter Construction

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

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

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

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

- Review of the as built grading plan, from a geotechnical perspective.
- Review of the contractor’s design of the temporary shoring system.
- Review and inspection of the foundation waterproofing system and all foundation drainage 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.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by Paterson.

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Ambassador Realty, 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
September 18, 2025
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

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

Report Distribution:

- ☐ Ambassador Realty (email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE ANALYSIS AND TESTING RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364816.33 **NORTHING:** 5027533.84 **ELEVATION:** 74.72

PROJECT: Proposed Mixed-Use Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: July 18, 2025

FILE NO. : PG7612

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.03m [74.69m]

FILL: Brown silty clay, with sand, crushed stone and gravel 1.07m [73.65m]

Hard to very stiff, brown **SILTY CLAY**, some sand, trace gravel 1.52m [73.20m]

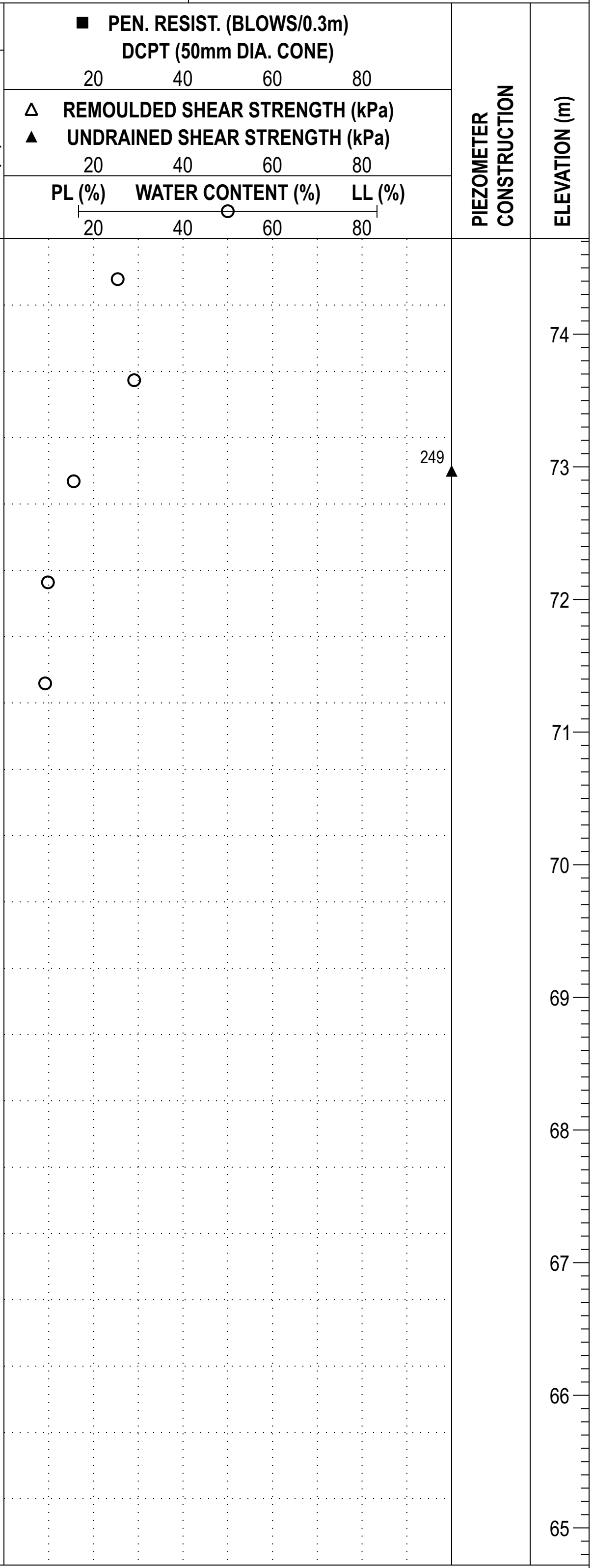
GLACIAL TILL: Loose to compact, brown silty sand, some gravel, trace cobbles and boulders 3.78m [70.94m]

End of Borehole

Practical refusal to augering at 3.78 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			25
SS 2	67	3-4-6-5 10	29
SS 3	58	2-6-12-9 18	16
SS 4		1-2-3-3 5	10
SS 5		3-3-3-50 6	9

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364818.04 **NORTHING:** 5027559.93 **ELEVATION:** 74.06

PROJECT: Proposed Mixed-Use Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: July 18, 2025

FILE NO. : PG7612

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION

GROUND SURFACE

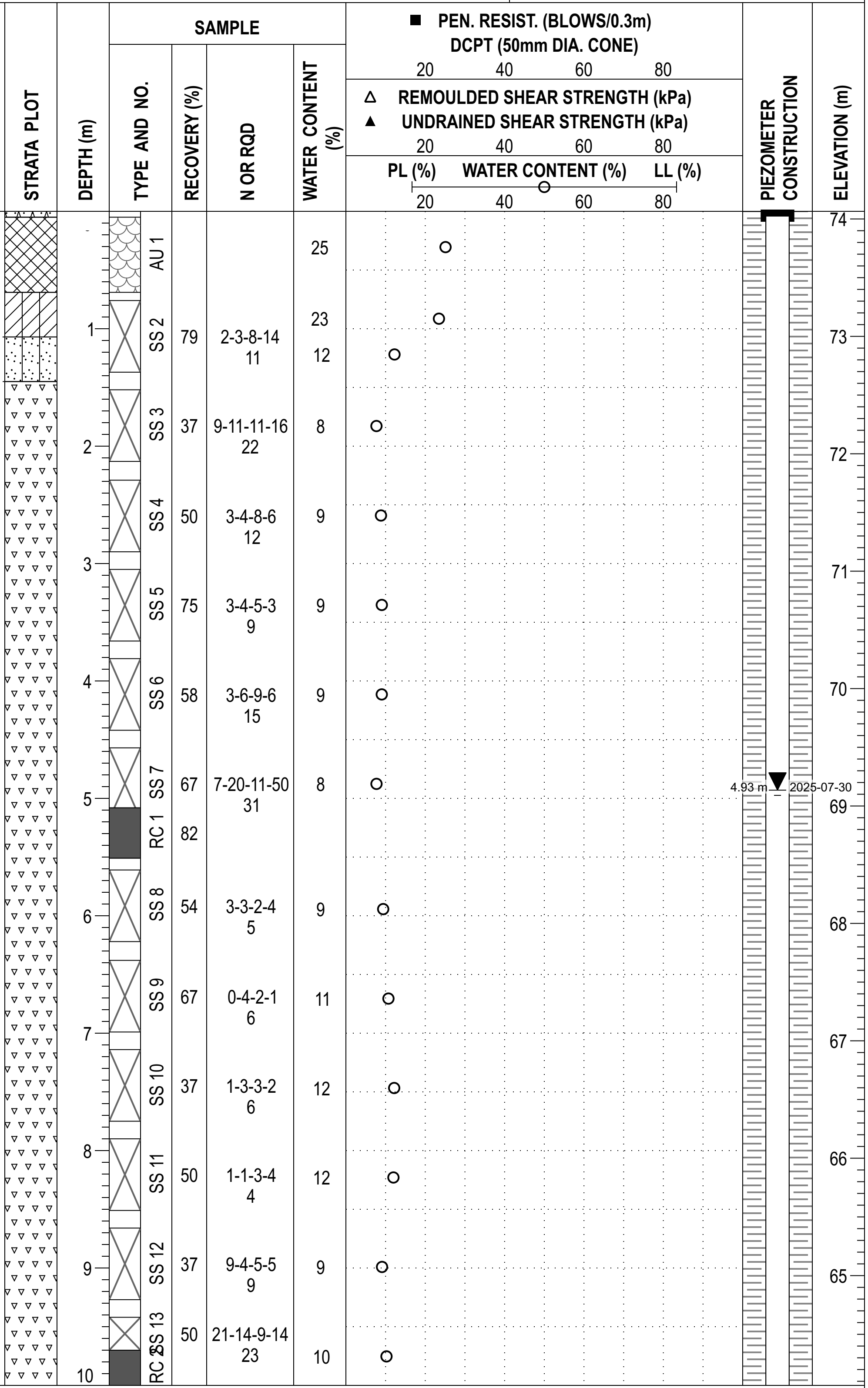
ASPHALT 0.05m [74.02m]

FILL: Brown silty clay, with sand, crushed stone and trace garvel

Brown **SILTY CLAY**, trace gravel 1.07m [73.00m]

Compact, brown **SILTY SAND**, some gravel 1.45m [72.61m]

GLACIAL TILL: Loose to very dense, grey silty sand, some gravel and clay, trace cobbles and boulders



SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			25
			23
SS 2	79	2-3-8-14 11	12
SS 3	37	9-11-11-16 22	8
SS 4	50	3-4-8-6 12	9
SS 5	75	3-4-5-3 9	9
SS 6	58	3-6-9-6 15	9
SS 7	67	7-20-11-50 31	8
RC 1	82		
SS 8	54	3-3-2-4 5	9
SS 9	67	0-4-2-1 6	11
SS 10	37	1-3-3-2 6	12
SS 11	50	1-1-3-4 4	12
SS 12	37	9-4-5-5 9	9
SS 13	50	21-14-9-14 23	
RC 2			10

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364818.04 **NORTHING:** 5027559.93 **ELEVATION:** 74.06

PROJECT: Proposed Mixed-Use Development

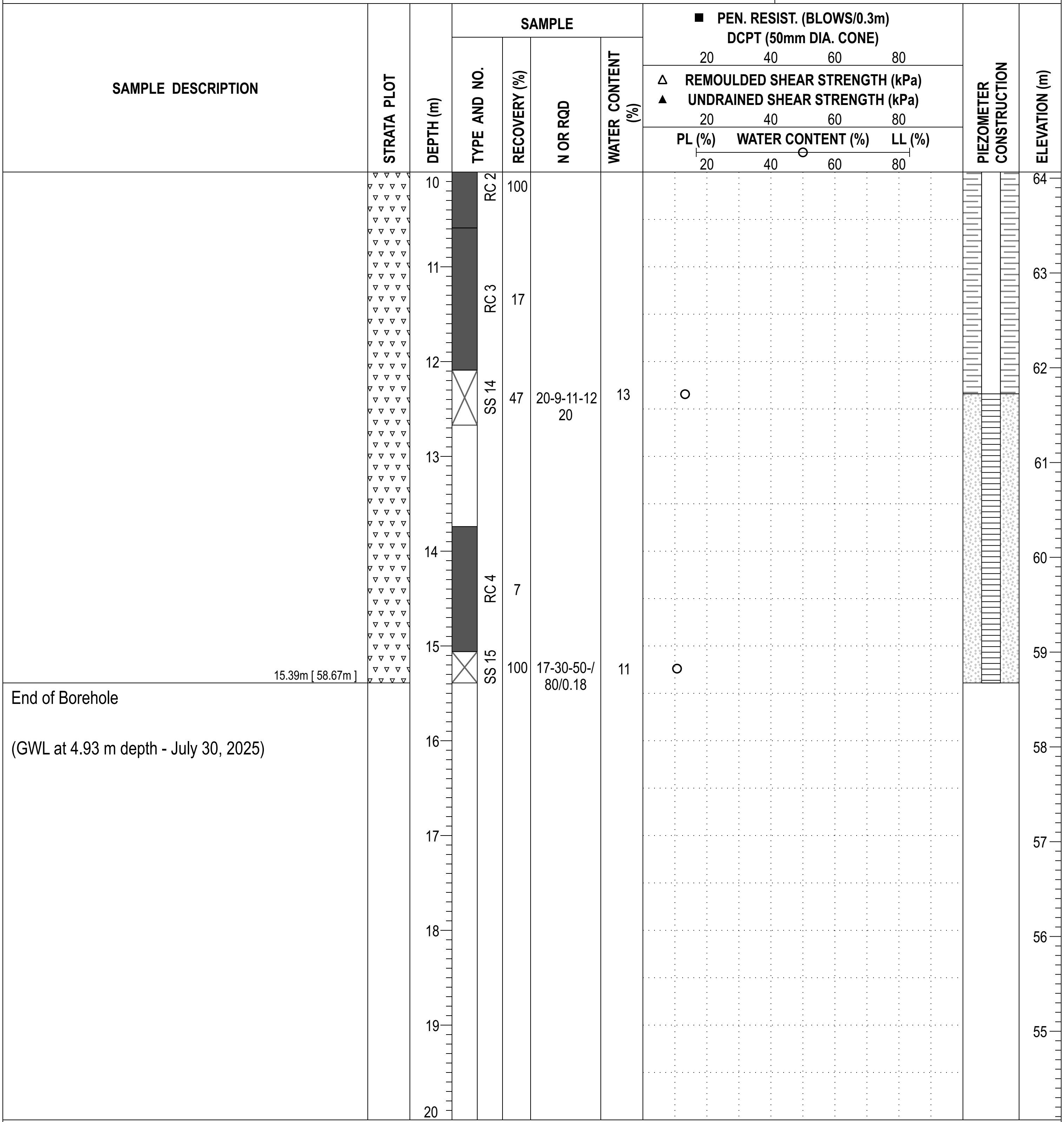
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: July 18, 2025

FILE NO. : PG7612

HOLE NO. : BH 2-25



SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
	RC 2	100		
	RC 3	17		
	SS 14	47	20-9-11-12 20	13
	RC 4	7		
15.39m [58.67m]	SS 15	100	17-30-50-/ 80/0.18	11
End of Borehole (GWL at 4.93 m depth - July 30, 2025)				

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364814.84 **NORTHING:** 5027587.32 **ELEVATION:** 74.03

PROJECT: Proposed Mixed-Use Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

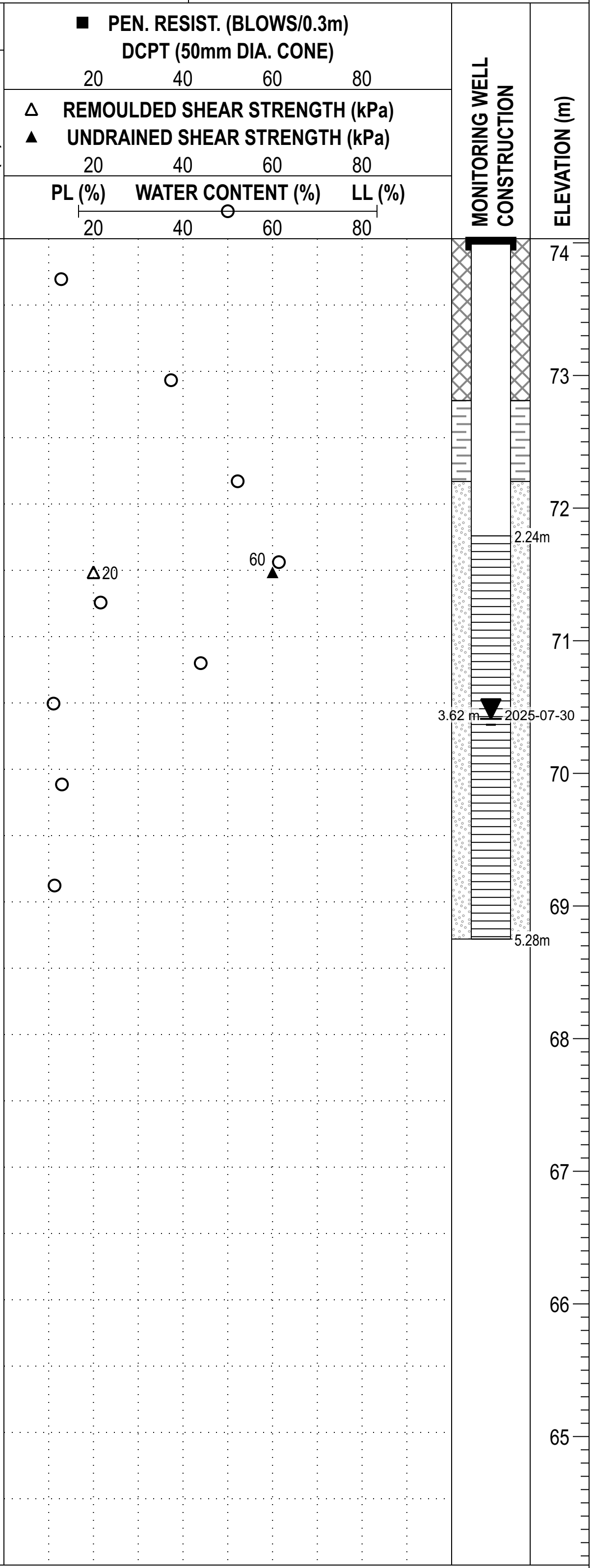
DATE: July 18, 2025

FILE NO. : PG7612

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION	
GROUND SURFACE	
ASPHALT	0.05m [73.98m]
FILL: Brown silty sand	0.91m [73.12m]
Stiff, brown **SILTY CLAY**	2.21m [71.82m]
Stiff, brown **CLAYEY SILT**, some sand, trace gravel	3.12m [70.91m]
GLACIAL TILL: Loose to very dense, grey silty sand, some clay and gravel, trace cobbles and boulders	5.28m [68.75m]
End of Borehole (GWL at 3.62 m depth - July 30, 2025)	

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1		3-5-5-6 10	13
SS 2	71	3-5-5-6 10	37
SS 3	83	5-3-3-3 6	52
SS 4	92	P	61 22
SS 5	79	1-2-2-8 4	44 11
SS 6	25	0-2-3-2 5	13
SS 7	4	18-20-22-21 42	11



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

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364789.31 **NORTHING:** 5027601.71 **ELEVATION:** 73.68

PROJECT: Proposed Mixed-Use Development

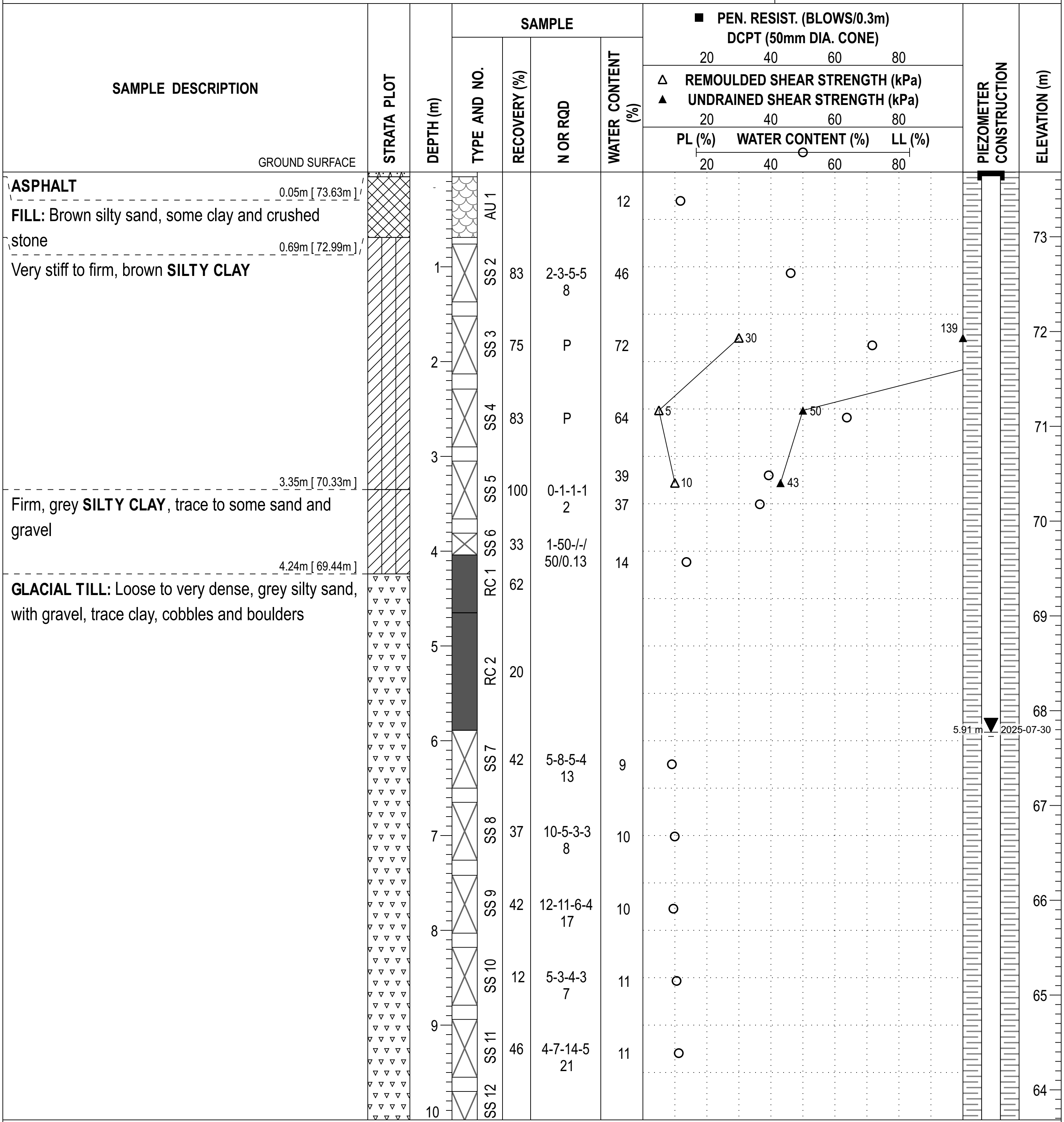
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: July 18, 2025

FILE NO. : PG7612

HOLE NO. : BH 4-24



ASPHALT 0.05m [73.63m]

FILL: Brown silty sand, some clay and crushed stone 0.69m [72.99m]

Very stiff to firm, brown **SILTY CLAY** 3.35m [70.33m]

Firm, grey **SILTY CLAY**, trace to some sand and gravel 4.24m [69.44m]

GLACIAL TILL: Loose to very dense, grey silty sand, with gravel, trace clay, cobbles and boulders

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			12
SS 2	83	2-3-5-5 8	46
SS 3	75	P	72
SS 4	83	P	64
SS 5	100	0-1-1-1 2	39 37
SS 6	33	1-50-/-/ 50/0.13	14
RC 1	62		
RC 2	20		
SS 7	42	5-8-5-4 13	9
SS 8	37	10-5-3-3 8	10
SS 9	42	12-11-6-4 17	10
SS 10	12	5-3-4-3 7	11
SS 11	46	4-7-14-5 21	11
SS 12			

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364789.31 **NORTHING:** 5027601.71 **ELEVATION:** 73.68

PROJECT: Proposed Mixed-Use Development

ADVANCED BY: CME-55 Low Clearance Drill

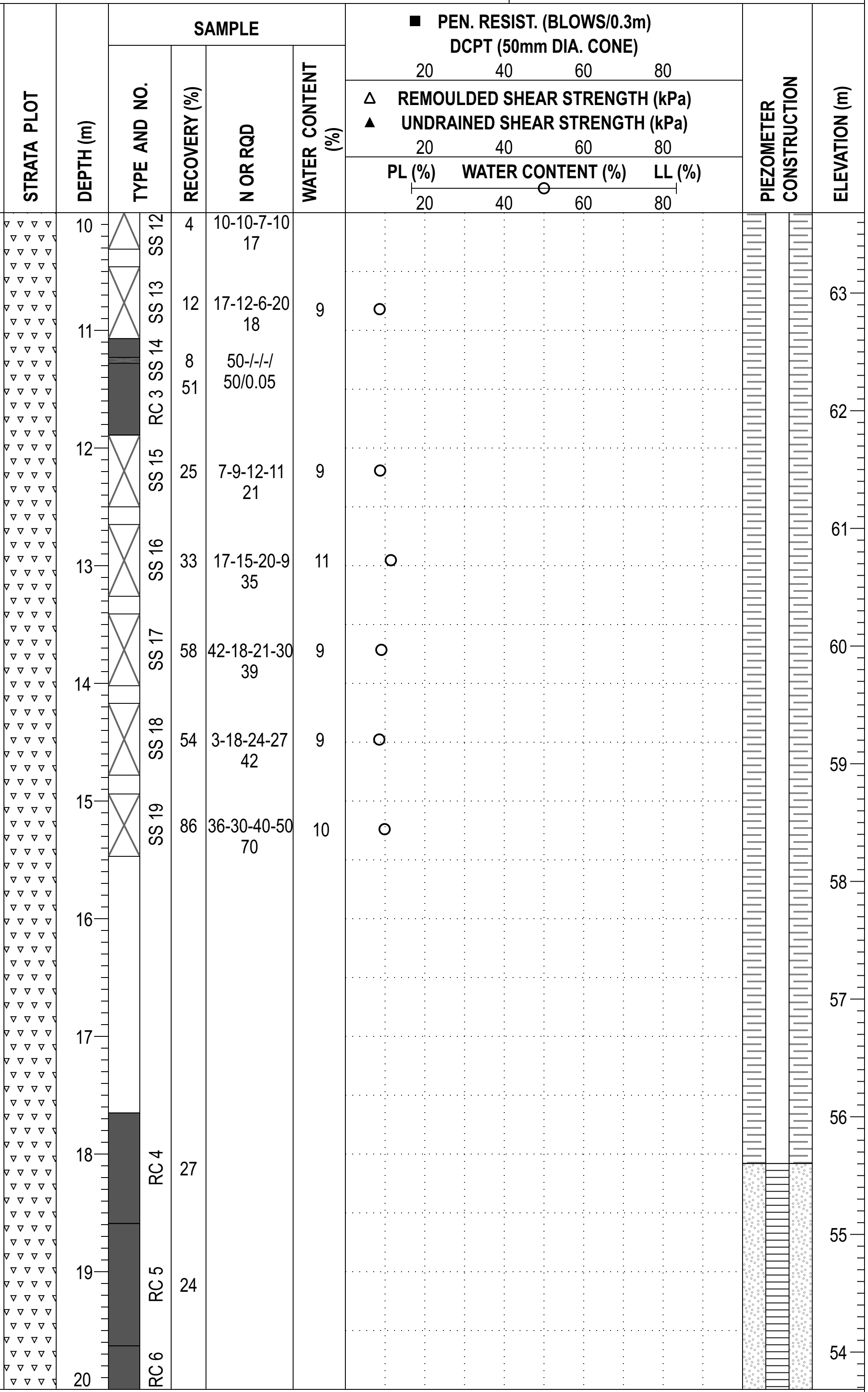
REMARKS:

DATE: July 18, 2025

FILE NO. : PG7612

HOLE NO. : BH 4-24

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 12	4	10-10-7-10 17	
SS 13	12	17-12-6-20 18	9
SS 14	8	50-/-/-/ 50/0.05	
RC 3	51		
SS 15	25	7-9-12-11 21	9
SS 16	33	17-15-20-9 35	11
SS 17	58	42-18-21-30 39	9
SS 18	54	3-18-24-27 42	9
SS 19	86	36-30-40-50 70	10
RC 4	27		
RC 5	24		
RC 6			



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364789.31 **NORTHING:** 5027601.71 **ELEVATION:** 73.68

PROJECT: Proposed Mixed-Use Development

ADVANCED BY: CME-55 Low Clearance Drill

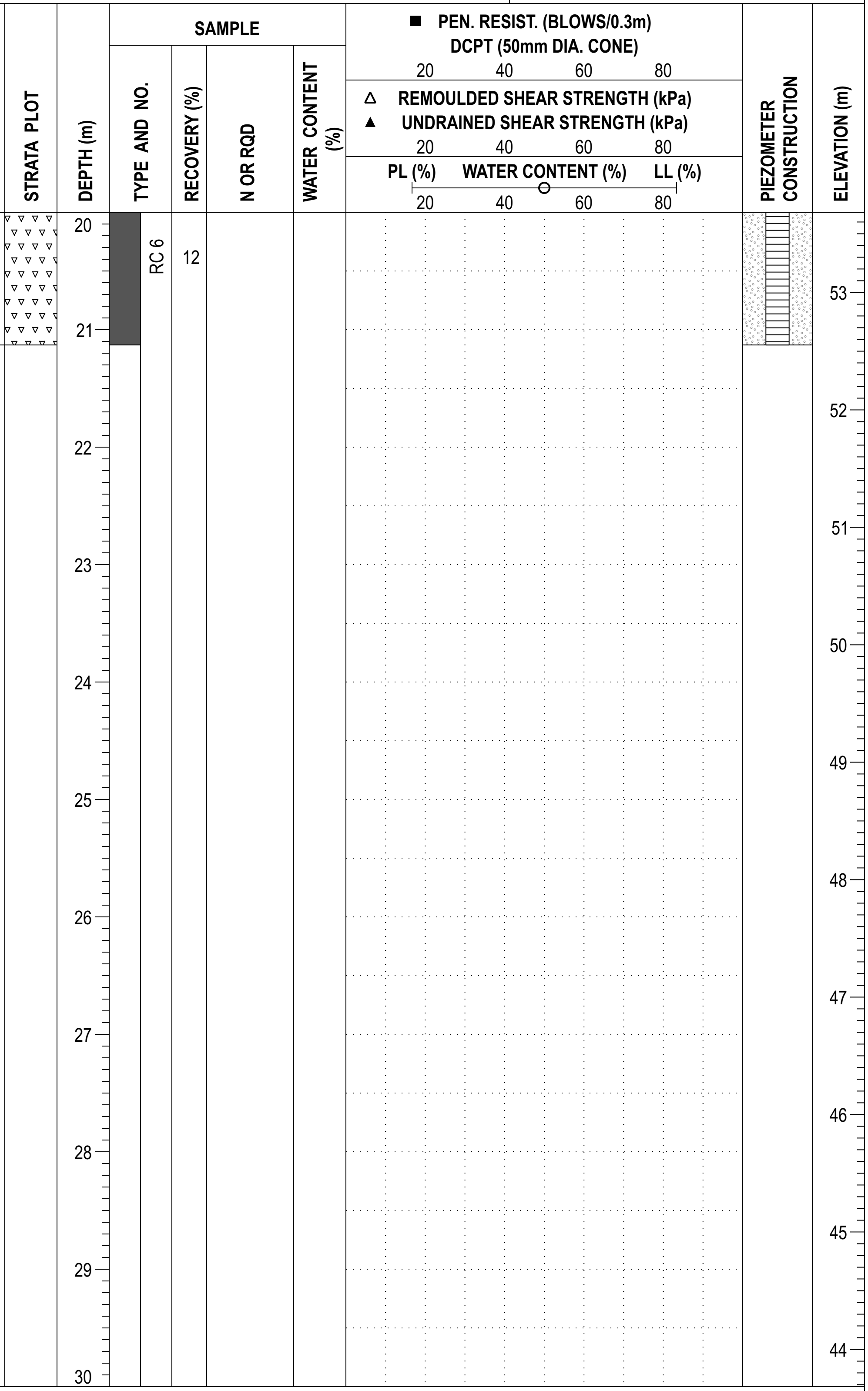
REMARKS:

DATE: July 18, 2025

FILE NO. : PG7612

HOLE NO. : BH 4-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
21.13m [52.55m]		20	RC 6	12				
End of Borehole (GWL at 5.91 m depth - July 30, 2025)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364845.87 **NORTHING:** 5027547.94 **ELEVATION:** 74.85

PROJECT: Proposed Mixed-Use Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: July 22, 2025

FILE NO. : PG7612

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.05m [74.80m]

FILL: Brown silty clay, some sand and crushed stone 0.81m [74.04m]

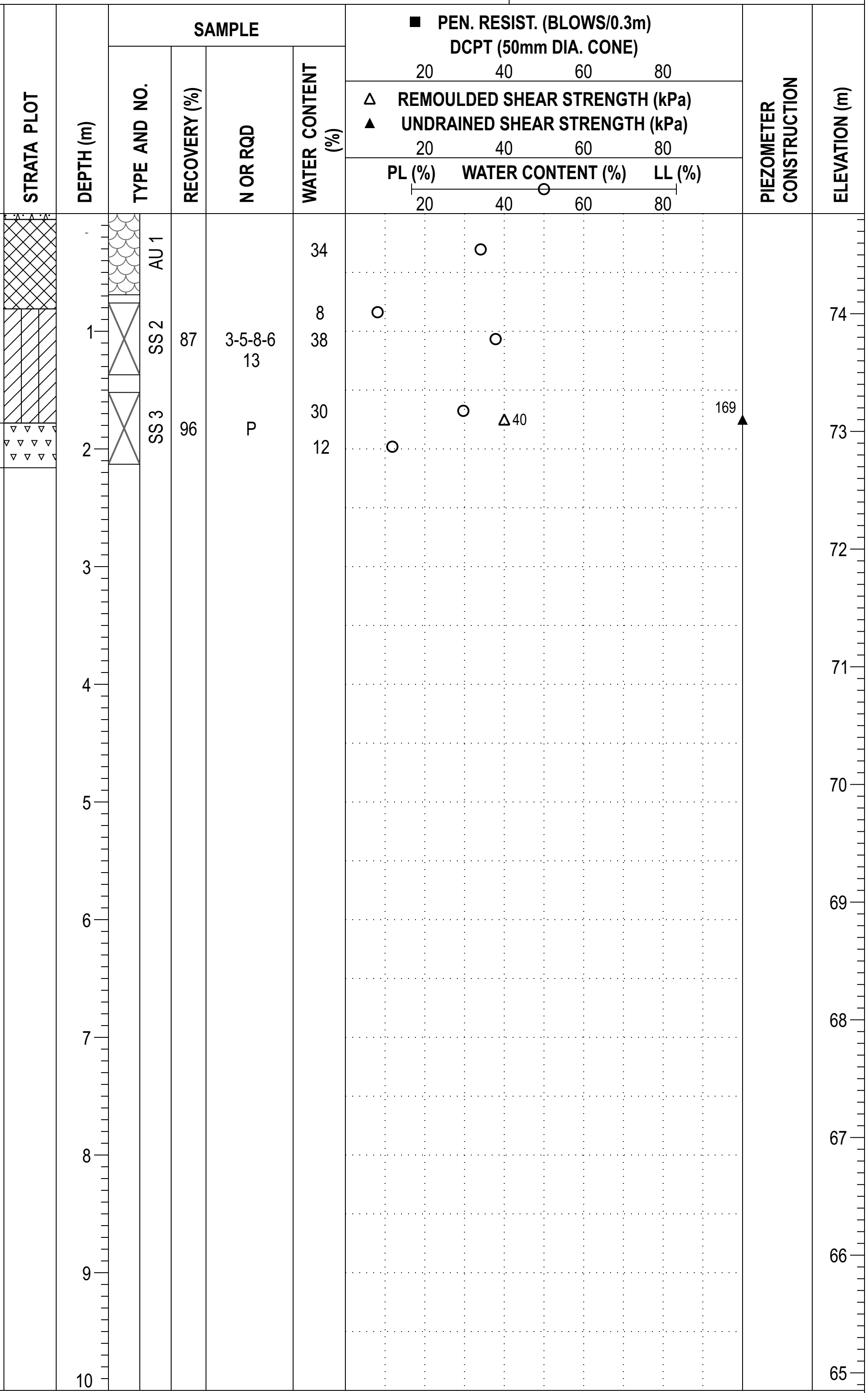
Very stiff, brown **SILTY CLAY** 1.78m [73.07m]

GLACIAL TILL: Loose, brown silty sand, with gravel, some clay, trace cobbles and boulders 2.16m [72.69m]

End of Borehole

Practical refusal to augering at 2.16 m depth

SAMPLE			
TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			34
SS 2	87	3-5-8-6 13	8 38
SS 3	96	P	30 12



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1296 & 1300 Carling Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364786.53 **NORTHING:** 5027571.42 **ELEVATION:** 73.98

PROJECT: Proposed Mixed-Use Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: July 22, 2025

FILE NO. : **PG7612**

HOLE NO. : **BH 6-25**

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.05m [73.93m]

FILL: Brown silty clay, with sand, some gravel 0.69m [73.29m]

FILL: Brown silty sand, some clay and crushed stone 2.21m [71.77m]

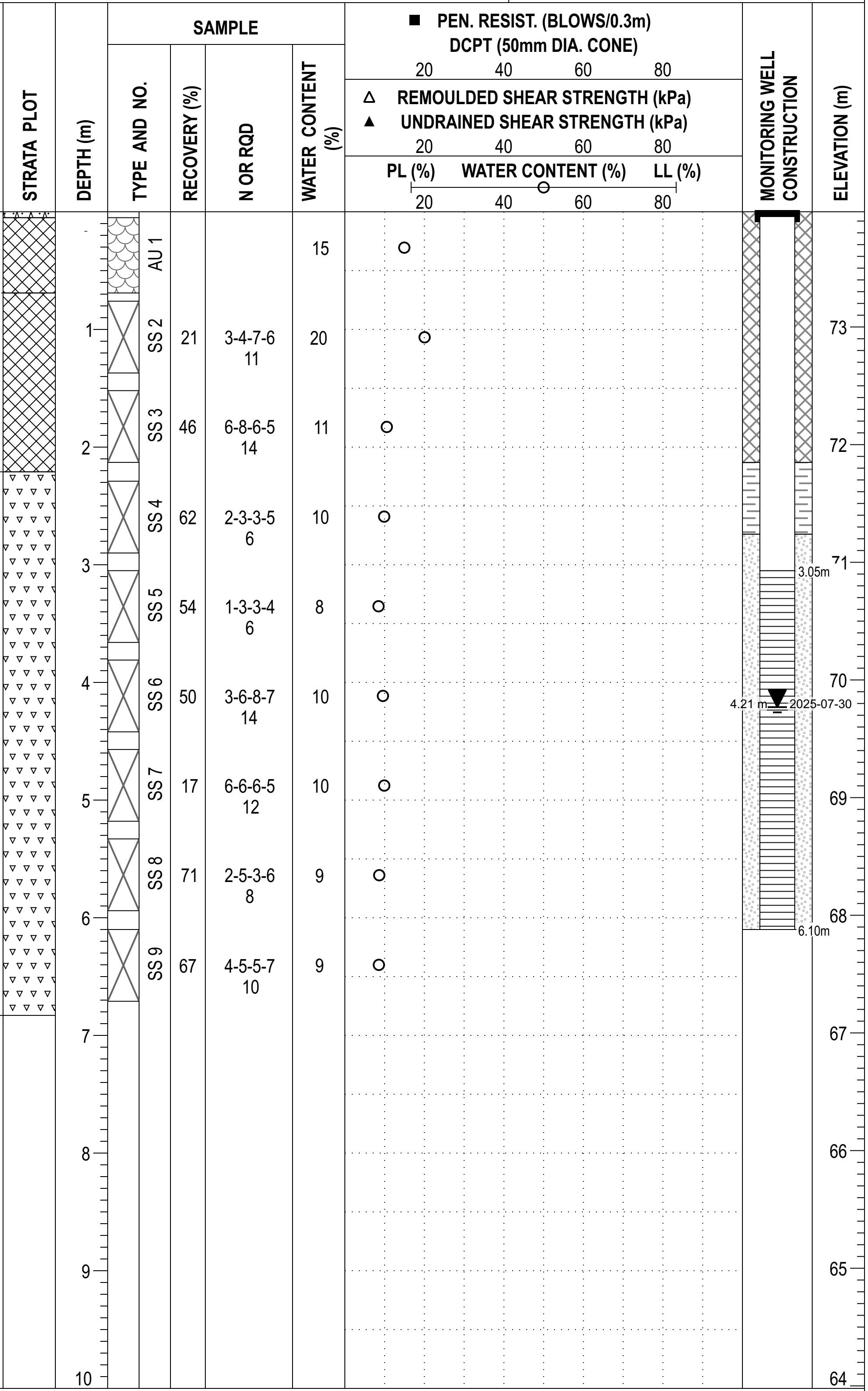
GLACIAL TILL: Loose to compact, grey silty sand, some gravel, trace clay, cobbles, and boulders 6.83m [67.15m]

End of Borehole

Practical refusal to augering at 6.83 m depth

(GWL at 4.21 m depth - July 30, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			15
SS 2	21	3-4-7-6 11	20
SS 3	46	6-8-6-5 14	11
SS 4	62	2-3-3-5 6	10
SS 5	54	1-3-3-4 6	8
SS 6	50	3-6-8-7 14	10
SS 7	17	6-6-6-5 12	10
SS 8	71	2-5-3-6 8	9
SS 9	67	4-5-5-7 10	9



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)

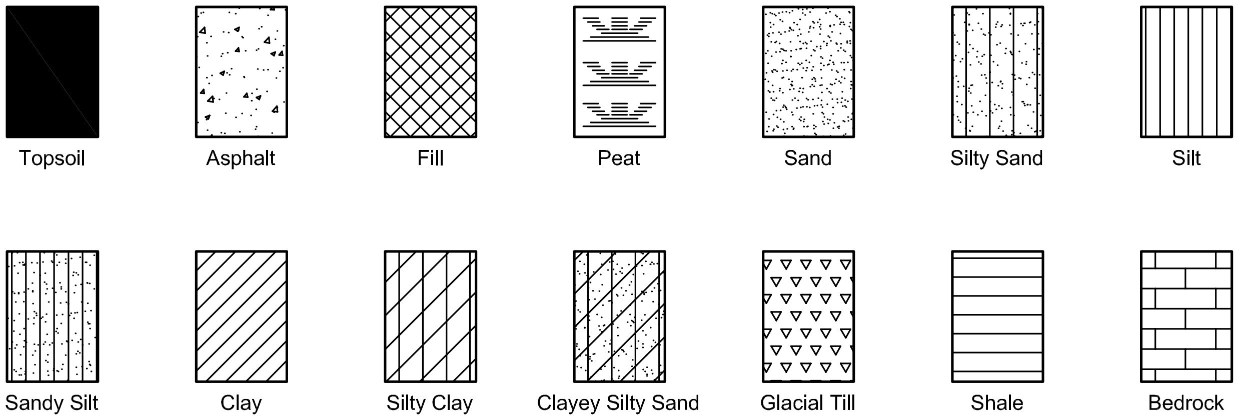
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

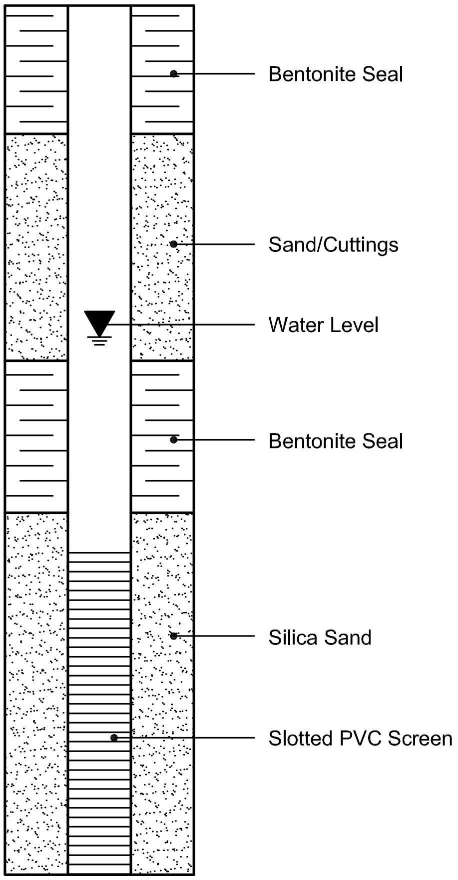
SYMBOLS AND TERMS (continued)

STRATA PLOT

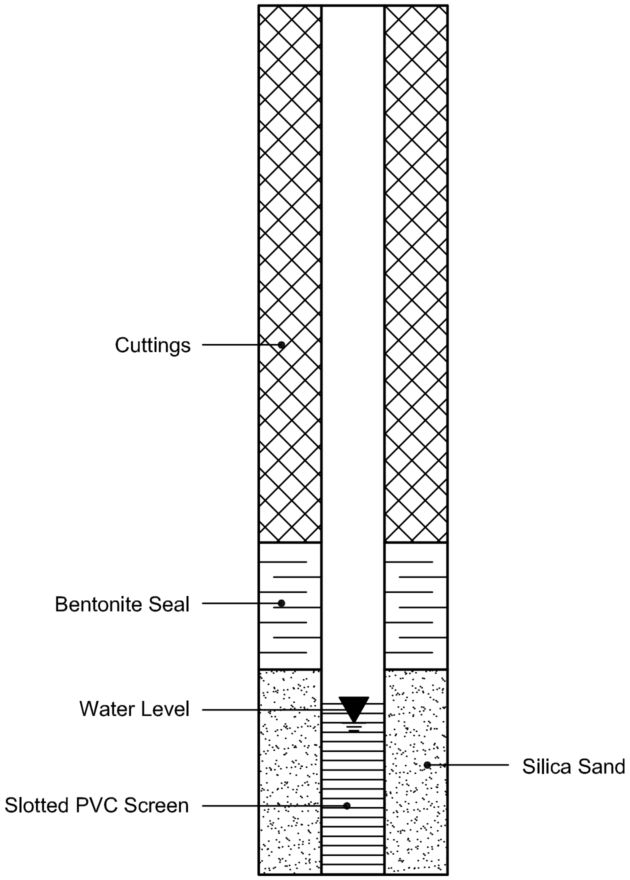


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



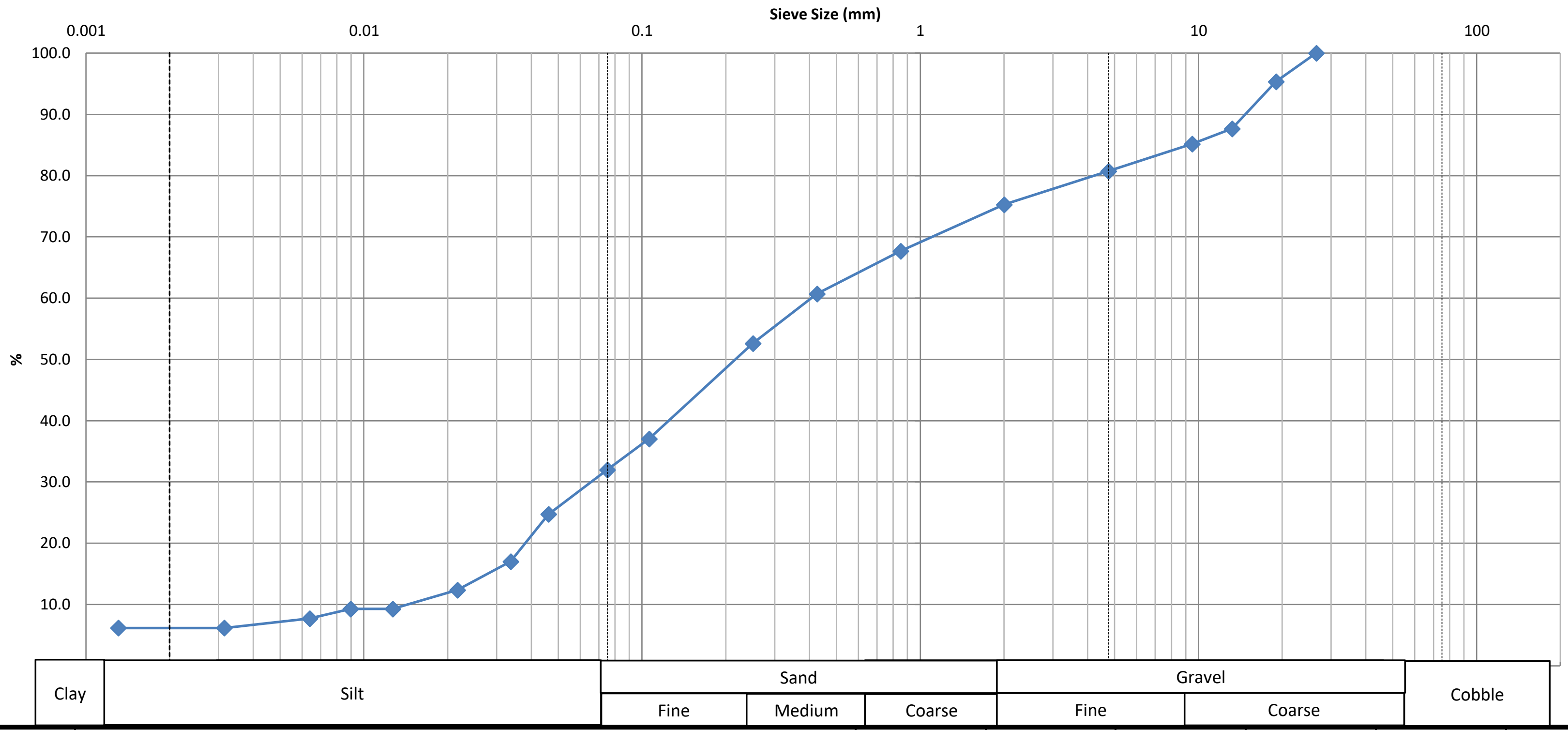
PIEZOMETER CONSTRUCTION



PATERSON GROUP

SIEVE ANALYSIS **ASTM C136**

CLIENT:	Ambassador Realty Inc.	DEPTH:	26' - 28'	FILE NO:	PG7612
CONTRACT NO.:		BH OR TP No.:	BH2-25 - SS11	LAB NO:	61399
PROJECT:	Lab Testing			DATE RECEIVED:	24-Jul-25
				DATE TESTED:	24-Jul-25
DATE SAMPLED:	18-Jul-25			DATE REPORTED:	6-Aug-25
SAMPLED BY:	Client			TESTED BY:	DJ



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		9.2					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				19.3	48.7	26.0	6.0

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Ambassador Realty Inc.	DEPTH:	26' - 28'	FILE NO.:	PG7612
PROJECT:	Lab Testing	BH OR TP No.:	BH2-25 - SS11	DATE SAMPLED	18-Jul-25
LAB No. :	61399	TESTED BY:	DJ	DATE RECEIVE	24-Jul-25
SAMPLED BY:	Client	DATE REPT'D:	06-Aug-25	DATE TESTED:	24-Jul-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
647.4	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	48.15	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	29.48	AIR DRY	507.00	507.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	488.20	488.20
		CORRECTED	0.963	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5	**0.0**	0.0	**100.0**
19	**30.2**	4.7	**95.3**
13.2	**80.0**	12.4	**87.6**
9.5	**95.8**	14.8	**85.2**
4.75	**124.7**	19.3	**80.7**
2.0	**160.1**	24.7	**75.3**
Pan	**487.3**		
0.850	**5.03**	32.3	**67.7**
0.425	**9.67**	39.3	**60.7**
0.250	**15.03**	47.4	**52.6**
0.106	**25.40**	63.0	**37.0**
0.075	**28.75**	68.0	**32.0**
Pan	**29.48**		

SIEVE CHECK	0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:57	22.0	6.0	23.0	0.0461	32.9	**24.7**
2	9:58	17.0	6.0	23.0	0.0336	22.6	**17.0**
5	10:01	14.0	6.0	23.0	0.0217	16.4	**12.4**
15	10:11	12.0	6.0	23.0	0.0127	12.3	**9.3**
30	10:26	12.0	6.0	23.0	0.0090	12.3	**9.3**
60	10:56	11.0	6.0	23.0	0.0064	10.3	**7.7**
250	15:06	10.0	6.0	23.0	0.0031	8.2	**6.2**
1440	9:56	10.0	6.0	23.0	0.0013	8.2	**6.2**

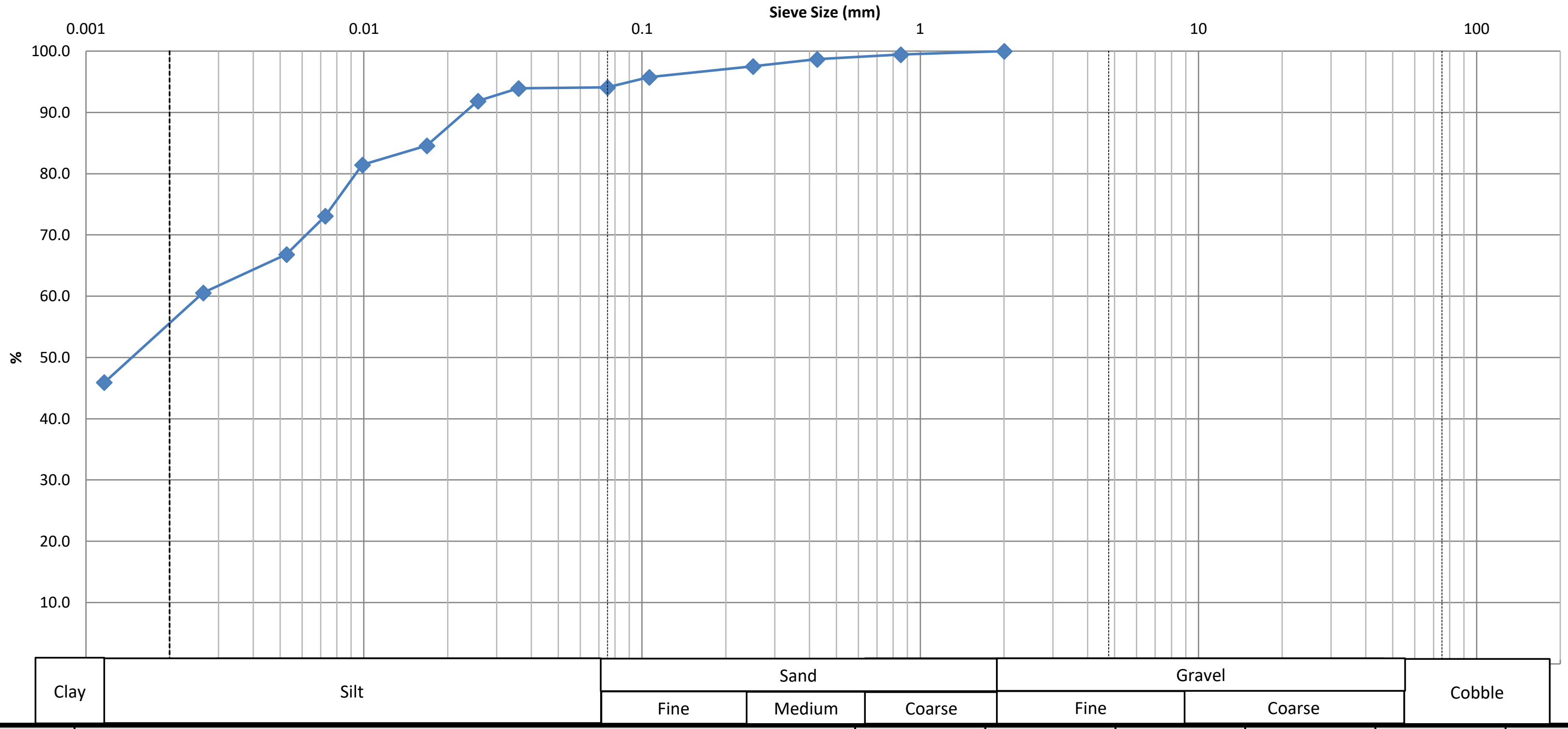
Moisture = 9.2

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS **ASTM C136**

CLIENT:	Ambassador Realty Inc.	DEPTH:	5' - 7'	FILE NO:	PG7612
CONTRACT NO.:		BH OR TP No.:	BH3-25 - SS3	LAB NO:	61400
PROJECT:	Lab Testing			DATE RECEIVED:	24-Jul-25
				DATE TESTED:	24-Jul-25
DATE SAMPLED:	18-Jul-25			DATE REPORTED:	6-Aug-25
SAMPLED BY:	Client			TESTED BY:	DJ



Identification	**Soil Classification**				**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
					45.1					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
					100.0	-94.1	39.1		55.0	

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Ambassador Realty Inc.	DEPTH:	5' - 7'	FILE NO.:	PG7612
PROJECT:	Lab Testing	BH OR TP No.:	BH3-25 - SS3	DATE SAMPLE	18-Jul-25
LAB No. :	61400	TESTED BY:	DJ	DATE RECEIVE	24-Jul-25
SAMPLED BY:	Client	DATE REPT'D:	06-Aug-25	DATE TESTED:	24-Jul-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
103.4	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	40.23	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEV	3.08	AIR DRY	128.50	128.50
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	103.40	103.40
		CORRECTED	0.805	

p

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75			
2.0	**0.0**	0.0	**100.0**
Pan	**103.4**		
0.850	**0.28**	0.6	**99.4**
0.425	**0.65**	1.3	**98.7**
0.250	**1.24**	2.5	**97.5**
0.106	**2.13**	4.3	**95.7**
0.075	**2.97**	5.9	**94.1**
Pan	**3.08**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:12	51.0	6.0	23.0	0.0359	94.0	**94.0**
2	10:13	50.0	6.0	23.0	0.0256	91.9	**91.9**
5	10:16	46.5	6.0	23.0	0.0168	84.6	**84.6**
15	10:26	45.0	6.0	23.0	0.0099	81.4	**81.4**
30	10:41	41.0	6.0	23.0	0.0072	73.1	**73.1**
60	11:11	38.0	6.0	23.0	0.0053	66.8	**66.8**
250	15:21	35.0	6.0	23.0	0.0026	60.5	**60.5**
1440	10:11	28.0	6.0	23.0	0.0012	45.9	**45.9**

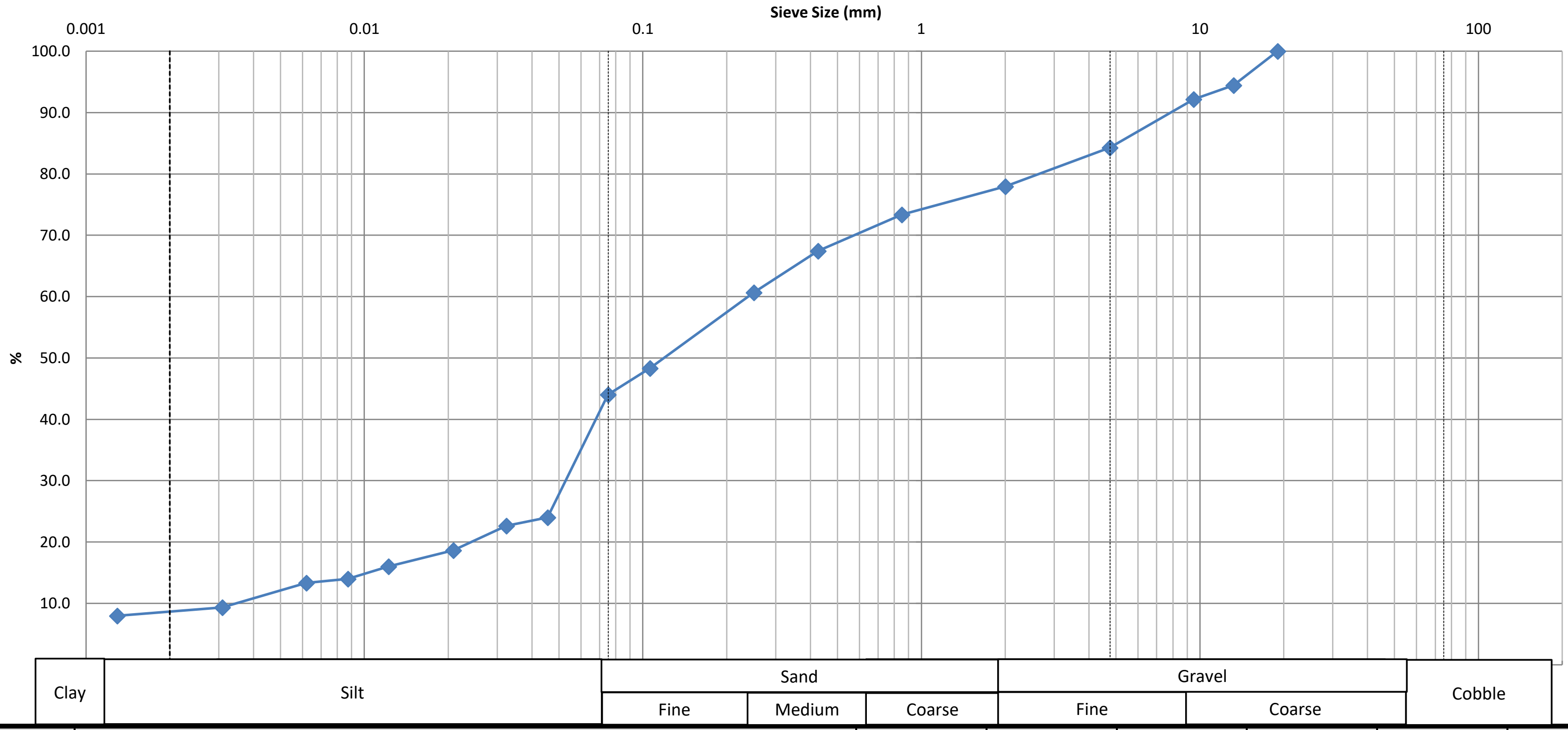
Moisture = 45.1

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS **ASTM C136**

CLIENT:	Ambassador Realty Inc.	DEPTH:	12'6" - 14'6"	FILE NO:	PG7612
CONTRACT NO.:		BH OR TP No.:	BH6-25 - SS6	LAB NO:	61401
PROJECT:	Lab Testing			DATE RECEIVED:	24-Jul-25
				DATE TESTED:	24-Jul-25
DATE SAMPLED:	18-Jul-25			DATE REPORTED:	6-Aug-25
SAMPLED BY:	Client			TESTED BY:	DJ



Identification	**Soil Classification**				**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
					8.5					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					15.7	40.2	35.0	9.0		

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Ambassador Realty Inc.	DEPTH:	12'6" - 14'6"	FILE NO.:	PG7612
PROJECT:	Lab Testing	BH OR TP No.:	BH6-25 - SS6	DATE SAMPLEI	18-Jul-25
LAB No. :	61401	TESTED BY:	DJ	DATE RECEIVE	24-Jul-25
SAMPLED BY:	Client	DATE REPT'D:	06-Aug-25	DATE TESTED:	24-Jul-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
608.6	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	49.12	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	22.02	AIR DRY	482.90	482.90
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	474.40	474.40
		CORRECTED	0.982	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19	**0.0**	0.0	**100.0**
13.2	**33.9**	5.6	**94.4**
9.5	**47.8**	7.9	**92.1**
4.75	**95.7**	15.7	**84.3**
2.0	**134.3**	22.1	**77.9**
Pan	**474.3**		
0.850	**2.93**	26.6	**73.4**
0.425	**6.74**	32.6	**67.4**
0.250	**11.08**	39.3	**60.7**
0.106	**19.00**	51.7	**48.3**
0.075	**21.74**	56.0	**44.0**
Pan	**22.02**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:17	24.0	6.0	23.0	0.0454	30.8	**24.0**
2	10:18	23.0	6.0	23.0	0.0323	29.1	**22.7**
5	10:21	20.0	6.0	23.0	0.0209	23.9	**18.7**
15	10:31	18.0	6.0	23.0	0.0122	20.5	**16.0**
30	10:46	16.5	6.0	23.0	0.0087	18.0	**14.0**
60	11:16	16.0	6.0	23.0	0.0062	17.1	**13.3**
250	15:26	13.0	6.0	23.0	0.0031	12.0	**9.3**
1440	10:16	12.0	6.0	23.0	0.0013	10.3	**8.0**

Moisture = 8.5

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

Certificate of Analysis

Report Date: 30-Jul-2025

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

Order Date: 24-Jul-2025

Client PO: 63648

Project Description: PG7612

Client ID:		BH2-25-SS4	-	-	-		
Sample Date:		18-Jul-25 09:00	-	-	-	-	-
Sample ID:		2530399-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	92.0	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.97	-	-	-	-	-
Resistivity	0.1 Ohm.m	6.0	-	-	-	-	-
Anions							
Chloride	10 ug/g	865	-	-	-	-	-
Sulphate	10 ug/g	237	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 – FOUNDATION DRAINAGE SYSTEM

DRAWING PG7612-1 – TEST HOLE LOCATION PLAN

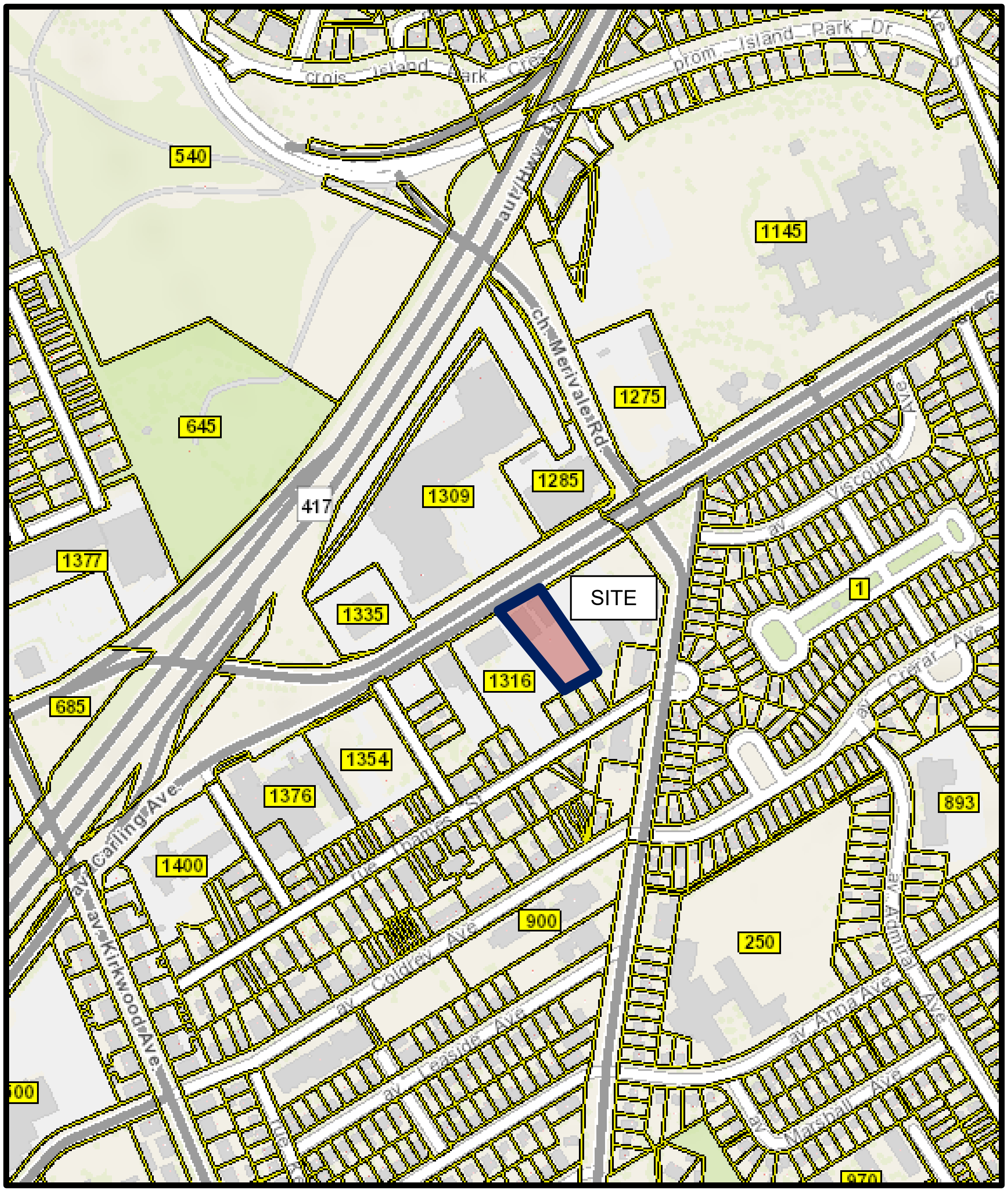
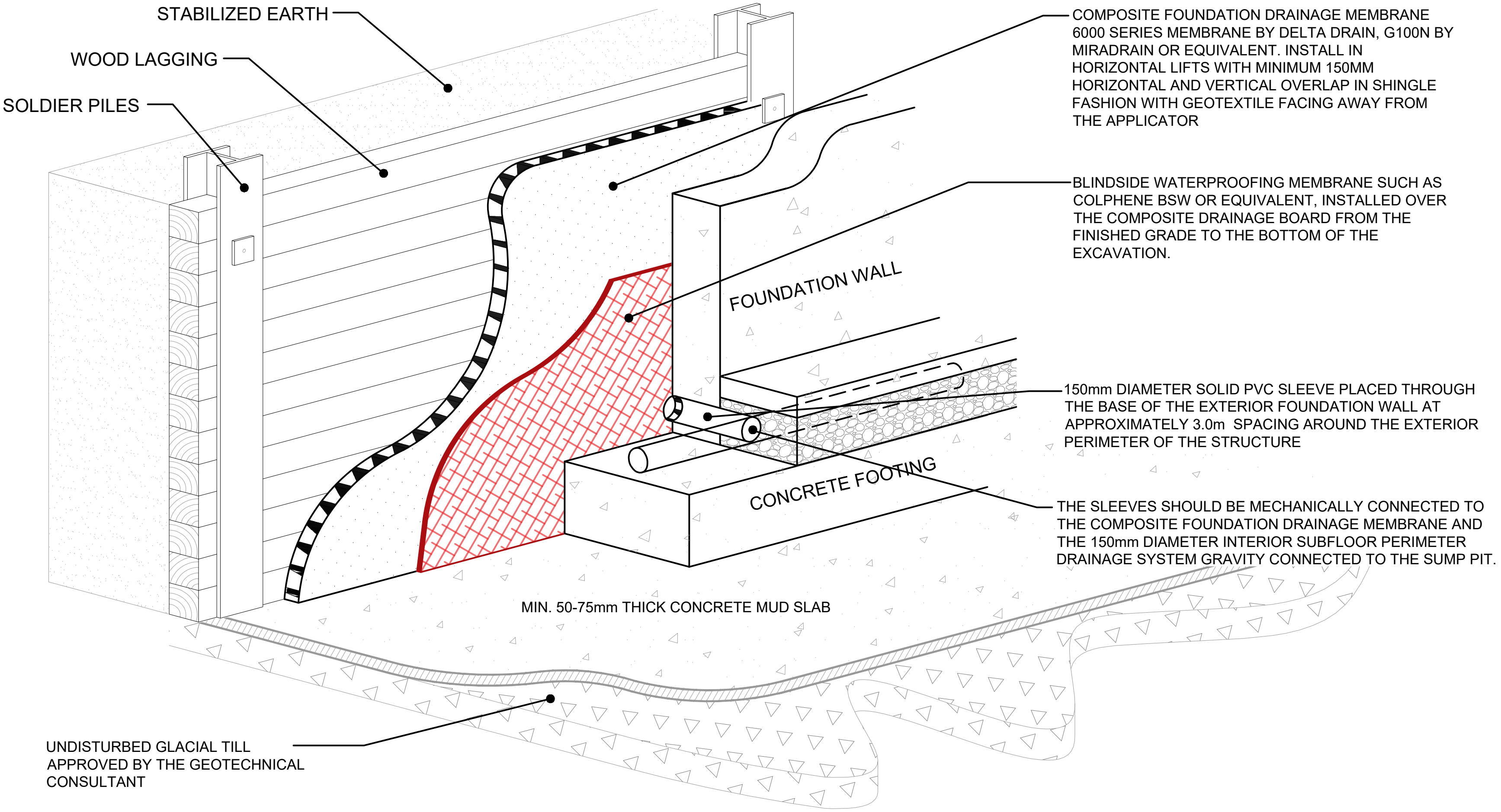


FIGURE 1

KEY PLAN

PATERSON
GROUP



COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150MM HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR

BLINDSIDE WATERPROOFING MEMBRANE SUCH AS COLPHENE BSW OR EQUIVALENT, INSTALLED OVER THE COMPOSITE DRAINAGE BOARD FROM THE FINISHED GRADE TO THE BOTTOM OF THE EXCAVATION.

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

UNDISTURBED GLACIAL TILL APPROVED BY THE GEOTECHNICAL CONSULTANT

PATERSON GROUP

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

AMBASSADOR REALTY INC.

PROPOSED MULTI-STOREY BUILDING
1296-1300 CARLING AVENUE

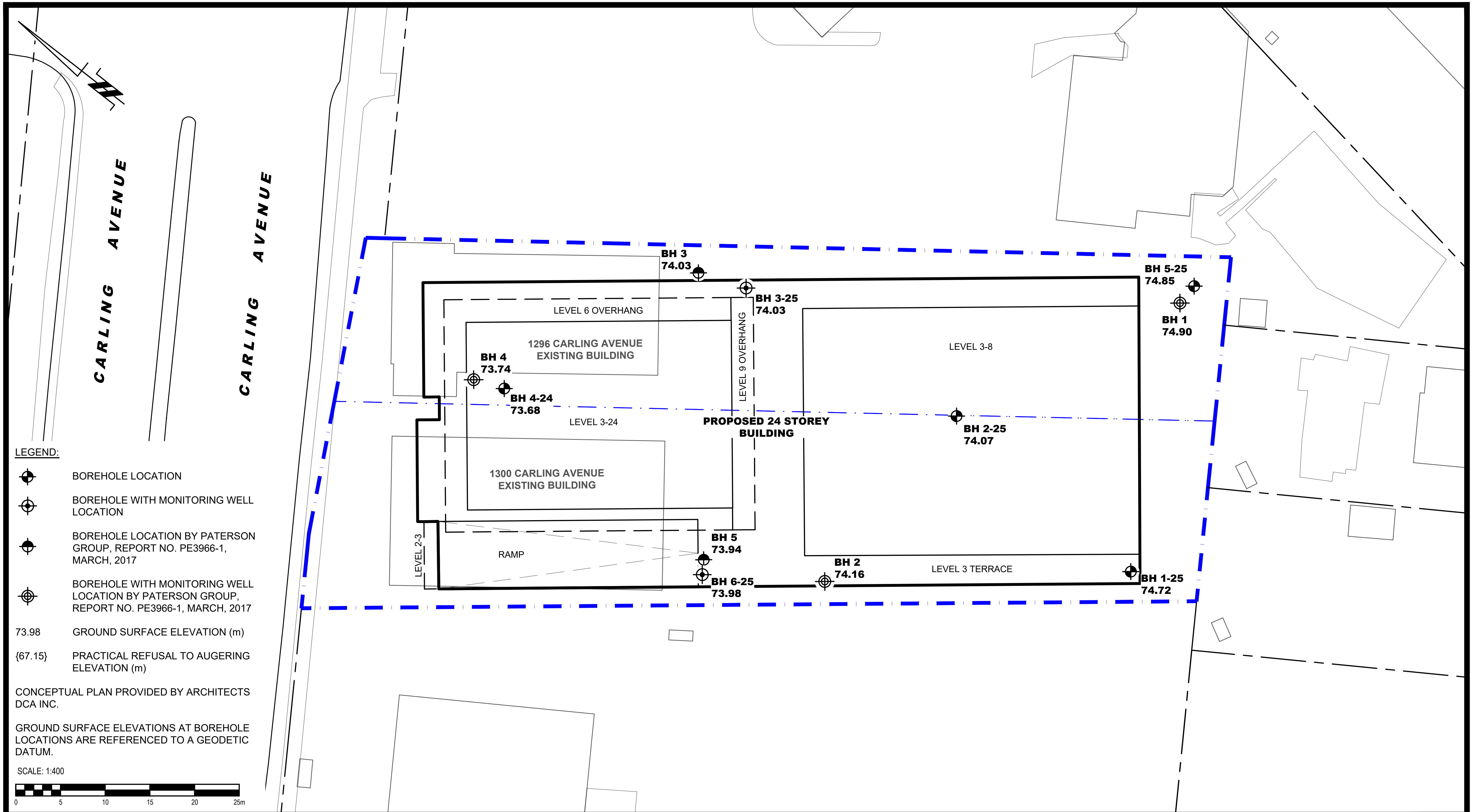
OTTAWA, ONTARIO

Title:

FOUNDATION DRAINAGE SYSTEM

Scale: N.T.S.	**Date:** 08/2025
Drawn by: GK	**Report No.:** PG7612-1
Checked by: PT	**Drawing No.:** FIG-1
Approved by: JV	**Revision No.:**

p:\autocad drawings\geotechnical\pg76xx\pg7612\pg7612-fig 1 foundation drainage system.dwg



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	BOREHOLE LOCATION BY PATERSON GROUP, REPORT NO. PE3966-1, MARCH, 2017
◎	BOREHOLE WITH MONITORING WELL LOCATION BY PATERSON GROUP, REPORT NO. PE3966-1, MARCH, 2017
73.98	GROUND SURFACE ELEVATION (m)
{67.15}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY ARCHITECTS DCA INC.

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

SCALE: 1:400

0 5 10 15 20 25m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

AMBASSADOR REALTY INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
1296 & 1300 CARLING AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 07/2025
Drawn by: ZS	Report No.: PG7612-1
Checked by: PT	Dwg. No.: PG7612-1
Approved by: JV	Revision No.:

11x17

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