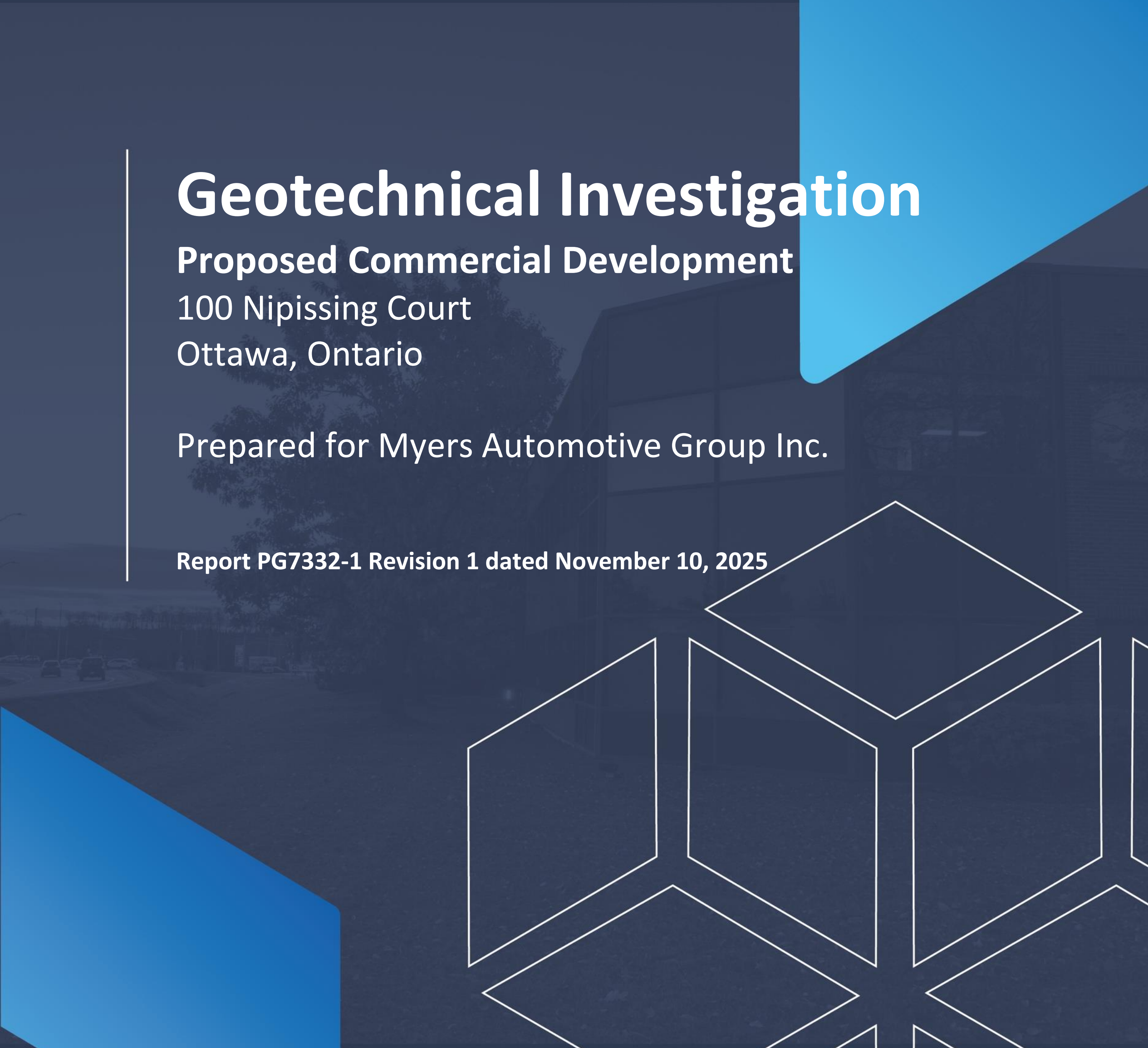

Geotechnical Investigation

Proposed Commercial Development

100 Nipissing Court
Ottawa, Ontario

Prepared for Myers Automotive Group Inc.

Report PG7332-1 Revision 1 dated November 10, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Myers Automotive Group Inc. to conduct a geotechnical investigation for a proposed commercial development located at 100 Nipissing Court in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

- ☐ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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.

Investigating for the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available conceptual plan, it is understood that the proposed development will consist of a slab-on-grade commercial building with associated office areas. The office areas are anticipated to occupy the ground and mezzanine levels.

Associated access lanes, at-grade parking, and landscaped and hardscaped areas are also anticipated as part of the development. The development is anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was conducted between August 11, and August 12, 2025, and consisted of 6 boreholes advanced to a maximum depth of 6.7 m below the existing ground surface. A dynamic cone penetration test (DCPT) was carried out at BH 6-25.

The test hole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG7332-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low clearance track-mounted auger 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 testing procedure consisted of augering to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

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

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.

The thickness of the overburden was evaluated by a dynamic cone penetration test (DCPT) at BH 6-25. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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 logged on the Soil Profile and Test Data Sheets in Appendix 1 of this report.

Groundwater

For the current investigation, a monitoring well was installed in BH 1-25, and the remainder of the boreholes were fitted with a flexible polyethylene standpipe to permit monitoring of the groundwater levels. The groundwater level readings were obtained after a suitable stabilization period subsequent to the completion of the field investigation.

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

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ Up to 1.5 m of slotted 32 or 51 mm diameter PVC screen at base the base of the boreholes.
- ☐ 32 or 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No.3 silica sand backfill within annular space around screen.
- ☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision, handheld GPS and referenced to a geodetic datum. The location of the boreholes is presented on Drawing PG7332-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of three (3) grain size distribution analyses were completed on selected soil samples. Moisture content testing was completed on all recovered soil samples from the current investigation. The results of the testing are presented in Section 4.2 and are provided in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were 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 undeveloped, vacant, and grass and/or gravel-covered with small trees throughout various locations. The site is bordered by Nipissing court to the north, Campeau drive to the west and commercial developments to the east and south. The existing ground surface across the subject site is relatively flat with an approximate geodetic elevation of 105 to 106 m.

Reference should be made to Figure 1 – Key Plan in Appendix 2 of the current report.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of fill underlain by a loose to compact, silty sand to sandy silt layer.

Fill extending to depths 0.7 m below the existing ground surface. The fill was generally observed to consist of brown silty sand with organics and with trace crushed stone. Further, a deposit of loose to dense brown silty sand to sandy silt with trace clay and gravel was encountered below the fill layer extending to depths ranging from 2.9 m to 3.7 m below the existing ground surface.

The fill and silty sand/ sandy silt layers were observed to be underlain by a layer of glacial till deposit extending to approximate depths ranging from 2.97 m to 6.7 m below the ground surface. The glacial till deposit generally consists of very loose to very dense, grey silty sand with trace to some gravel and clay.

Practical refusal to DCPT was reached at depth of 7.82 m at BH 6-25. 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 test hole location.

Bedrock

Based on available geological mapping, the bedrock consists of interbedded limestone and shale of the Verulam formation in the north and interbedded limestone and dolomite of the Gull River formation to the south, with an overburden drift thickness ranging from 10 to 15 m.

Grain Size Distribution and Hydrometer Testing

Grain size distribution analysis was completed on three selected recovered soil samples. The results of the grain size distribution analysis are presented in Table 1 below and on the Grain Size Distribution sheets in Appendix 1.

Table 1 – Grain Size Distribution Results

Sample	Depth (m)	Gravel (%)	Sand (%)	Silt and Clay (%)
BH 3-25 SS4	2.59	0.3	20.4	19.3
BH 4-25 SS6	4.88	5.2	60.3	34.5
BH 5-25 SS6	4.88	8.8	54.6	36.6

4.3 Groundwater

Groundwater levels were recorded at each test hole location and presented in Table 2 below. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 2 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-25*	105.10	2.94	102.16	August 19, 2025
BH 2-25	105.33	2.95	102.38	
BH 3-25	105.24	3.29	101.95	
BH 4-25	105.37	3.2	102.17	
BH 5-25	105.27	5.46	99.81	
BH 6-25	105.50	3.34	102.16	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

* - A monitoring well has been installed in these boreholes.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

The long-term groundwater levels can also be estimated based on the observed colour, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater level is expected to range between approximately **2.0 to 3.0 m** below ground surface. 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 the proposed commercial buildings be founded on conventional spread footings bearing on an undisturbed loose to compact silty sand/sandy silt deposit. Due to the absence of the sensitive silty clay deposit within the subject site, the proposed development will not be subjected to permissible grade raise restrictions.

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 or construction debris, should be stripped from under any paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Fill Placement

Fill placed for grading throughout the building footprint should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in a maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in a maximum of 300 mm thick loose lifts and 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 maximum 300 mm thick loose lifts to at least 98% of the material’s SPMDD.

If site-generated material is to be used to build up the subgrade level for areas to be paved or as backfill around footings throughout the building footprint, it should be compacted in maximum 300 mm thick loose lifts and by several passes of a suitably sized smooth drum roller. The site-generated fill should consist of workable sandy soil fill free of organic debris (topsoil, logs, stumps, etc.), inorganic material and/or stones/cobbles larger than 100 mm in their longest dimension. Fill meeting the aforementioned conditions are considered suitable for re-use throughout the subject site. Wet site-generated fill, such as grey sandy soils, will be saturated and are expected to be difficult to re-use without an extensive drying period. Therefore, those soils are not anticipated to be suitable for this purpose without some improvement.

Soils intended for re-use which become frozen and/or which have excessive moisture contents will not be considered suitable for re-use at the subject site. Placement of this material during winter months increases the risk of placing frozen material which is expected to result in future poor performing areas that will require future repair due to long-term thawing and higher than tolerable amounts of settlement from placing frozen material. This process should be reviewed and approved by Paterson field personnel upon completion of each lift and who are experienced in reviewing the placement of soil fill in this manner.

All imported and site-generated fill material should be tested and approved by Paterson prior to delivery or re-use on site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

5.3 Foundation Design

Bearing Resistance Value (Conventional Shallow Foundation)

Conventional spread footing foundations founded on an undisturbed, compact to dense sandy silt to silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported 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.

Proof Rolling and Subgrade Improvement for Loose Sand Below Footings

Where the sand bearing surface for foundations is considered loose by Paterson at the time of construction, it would be recommended to proof roll the bearing surface prior to forming for footings or sub-excavating in-situ material. Proof-rolling (i.e., re-compacting) is recommended to be undertaken in **dry conditions and above freezing temperatures** by an adequately sized vibratory roller making several passes to achieve optimal compaction levels.

Depending on the looseness and degree of saturation of loose sandy soils at the time of construction, other measures (additional compaction, sub-excavation and reinstatement with crushed stone fill) may be recommended to accommodate site conditions at the time of construction. However, these considerations would be evaluated at the time of design by Paterson on a footing-specific basis.

Settlement

Footings bearing on an undisturbed soil can be designed for the bearing resistance values provided herein 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 an undisturbed silty sand/sandy silt bearing surface 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.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to determine the applicable seismic site designation for the proposed building in accordance with the Ontario Building Code 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figure 2 and Figure 3 in Appendix 2 of the present report.

Field Program

The seismic array was located as presented in Drawing PG7332-1 - Test Hole Location Plan attached to the present report.

Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a laptop computer and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 20.0, 4.5, and 3.0 m away from the first geophone and last geophone, and at the centre of the geophone array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct, reflected, and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile immediately below the proposed buildings foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the average overburden shear wave velocity is **297 m/s**, while the bedrock shear wave velocity is **2,441 m/s**. Further, the testing results indicate the average overburden thickness to be approximately 8 m.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2024.

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{s_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{8\ m}{297\ m/s} + \frac{22\ m}{2{,}441\ m/s}\right)}$$

$$V_{s30=}\ 835\ m/s$$

Based on the results of the seismic testing, the average shear wave velocity Vs_{30}, for the proposed building is **835 m/s**. However, it is anticipated that the bedrock surface will be located more than 3 m of the founding depth. Therefore, a **Site Designation X_{760}** is applicable for the design of the proposed building according to the Ontario Building Code 2024.

The soils underlying the subject site are not susceptible to liquefaction.

5.5 Slab-on-Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the undisturbed, existing fill or undisturbed, in-situ compact and/or re-compacted silty sand/sandy silt are considered to be acceptable subgrades on which to backfill for slab on grade construction.

As noted above in Section 5.2, consideration may be considered to re-using suitably prepared and Paterson-approved site-generated fill to reinstated the sub-slab fill subgrade layer, which should be observed and approved by Paterson. Any poor performing areas in the slab on grade subgrade should be removed and reinstated with an engineered fill such as OPSS Granular A or OPSS Granular B Type II.

It is recommended that the upper 200 mm sub-floor fill consist of OPSS Granular A crushed stone. All backfill materials within the footprint of the proposed buildings should be placed in a maximum of 300 mm thick loose layers and compacted to at least 95% of their SPMDD.

5.6 Pavement Structure

For design purposes, the following pavement structures, presented below, are recommended for the design of, car-only parking, and heavy traffic areas. The proposed pavement structures are shown in Tables 3 and 4.

Table 3 - Recommended Pavement Structure - Car-Only Parking Areas and Fire-Truck Routes

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

Table 4 - Recommended Pavement Structure – Access Lanes and Heavy-Truck Parking Areas

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

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 100% of the material's SPMDD using suitable compaction equipment.

Unpaved Pavement Structures

It is understood that a portion of the subject site will be left unpaved. The following granular structure presented below in Table 5 is recommended for the design of these unpaved areas.

Table 5 - Recommended Granular Structure (Gravel Covered) Access Lanes and Heavy-Duty Truck Parking Areas

Thickness (mm)	Material Description
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
Separation Layer	**Woven Geotextile** - Terrafix 200W or equivalent
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

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

5.7 Preliminary Percolation Rates

Infiltration galleries are anticipated to be located beneath the asphaltic parking areas within the subject site. Paterson completed a detailed hydrogeological investigation of the lands north and west of the subject site as part of previous phases of the Kanata West Business Park (KWBP) in order to establish hydraulic conductivity and percolation time of in-situ materials throughout the KWBP.

It is anticipated that silty sand or sandy silt will be encountered at the base of the infiltration galleries. The percolation rate for silty sand was interpreted from typical material properties and accepted published data. Based on these values, the average percolation rate (T-Time) is estimated to range between 8 to 20 mins/cm, and the hydraulic conductivity ranging between $1x10^{-4}$ and $1x10^{-5}$ m/sec.

The percolation rate for sandy silt was interpreted from hydraulic conductivity which was estimated based on previous investigations within the immediate vicinity of the subject site and consistent subsoils. Based on these values, the average percolation rate (T-Time) was estimated to range between 20 to 50 mins/cm, and the hydraulic conductivity ranges between $1x10^{-7}$ and $1x10^{-8}$ m/sec.

These values may be used for preliminary planning purposes and are recommended to be supplemented by site-specific testing at the design invert depth of the proposed systems to verify in-situ conditions and provide site-specific design parameters. Further, infiltration-based stormwater management systems should be planned considering the groundwater levels identified herein.

6.0 Design and Construction Precautions

6.1 Groundwater Control for Construction

Foundation Drainage

Since the buildings will consist of a slab-on-grade structure, the perimeter foundation wall will be backfilled with either relatively free-draining site-generated sandy fill and/or non-frost susceptible crushed stone fill and groundwater is relatively deeper than the anticipated founding depths, a perimeter foundation drainage system is not considered required for the proposed structure.

Retaining walls, such as for loading docks, are recommended to be provided with a subdrain pipe within the retained backfill layer. Generally, this is recommended to consist of a minimum 100 to 150 mm diameter perforated pipe extending the length of the wall structure and placed at the interface between the rear face and the bedding surface. This pipe should be surrounded with a minimum 150 mm thick layer of 19 mm clear crushed stone and fitted with a geosock. The subdrain system should be provided with a gravity outlet such as connections to a nearby catch basin.

Foundation Backfill

Backfill against the exterior sides of the foundation wall may consist of on-site excavated fill provided it is prepared and compacted as advised in Subsection 5.2 of this report. Imported granular materials, such as OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Consideration should be given to using a minimum 450 mm thick layer of OPSS Granular B Type II crushed stone surfaced with a minimum 150 mm thick layer of OPSS Granular A crushed stone below entrance slabs and other settlement/heave-sensitive surfaces surrounding the structure. The use of rigid insulation to provide additional frost protection may be evaluated and advised upon by Paterson during the detailed design stage once final design concepts are better known.

6.2 Protection Against Frost Action

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

A minimum 1.5 m thick soil cover alone, or a combination of soil cover in conjunction with foundation insulation should be provided in this regard.

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

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. For the proposed development, it is expected that sufficient room will be available for the greater part of the excavation to be undertake by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 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 maintain safe working distance from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

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

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill 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) should match the soils exposed at the trench walls to minimize differential frost heaving.

All trench backfill placed above the pipe surround layers (i.e., bedding, spring-line and cover layers) should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD. It is recommended the trench backfill process is reviewed for conformance with these recommendations during the construction program.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps when undertaken above the groundwater table. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

Should excavations be required to extend below the groundwater table, it is recommended that the excavation contractor retain a construction dewatering specialist to advise on a suitable means to carry out excavations below the groundwater table and in the dry. Additional field testing may be undertaken to assess infiltration rates and volumes once preliminary site servicing concepts are available to verify anticipated excavations depths and potential works below the groundwater table. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

Impacts on Neighboring Properties

Based on the nature of the in-situ subsoils, groundwater levels and dpeht of influence of the proposed development, the proposed development is not anticipated to result in negative impacts to neighboring developments by potential long-term dewatering from a geotechnical perspective.

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 by the use of 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 in 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 samples 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 non-aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

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

➢ Review preliminary and detailed grading, servicing and structural plan(s) from a geotechnical perspective.

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

☐ Review of the final design grading, and servicing details from a geotechnical perspective.

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

☐ Sampling and testing of the concrete and fill materials used.

☐ 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 Paterson personnel.

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 Myers Automotive Group Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
November 10, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Udhaya Ramachandran, E.I.T.

Drew Petahtegoose, P.Eng.

Report Distribution:

☐ Myers Automotive Group. (E-mail 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

100 Nipissing Court, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 348237.23 **NORTHING:** 5017270.36 **ELEVATION:** 105.10

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

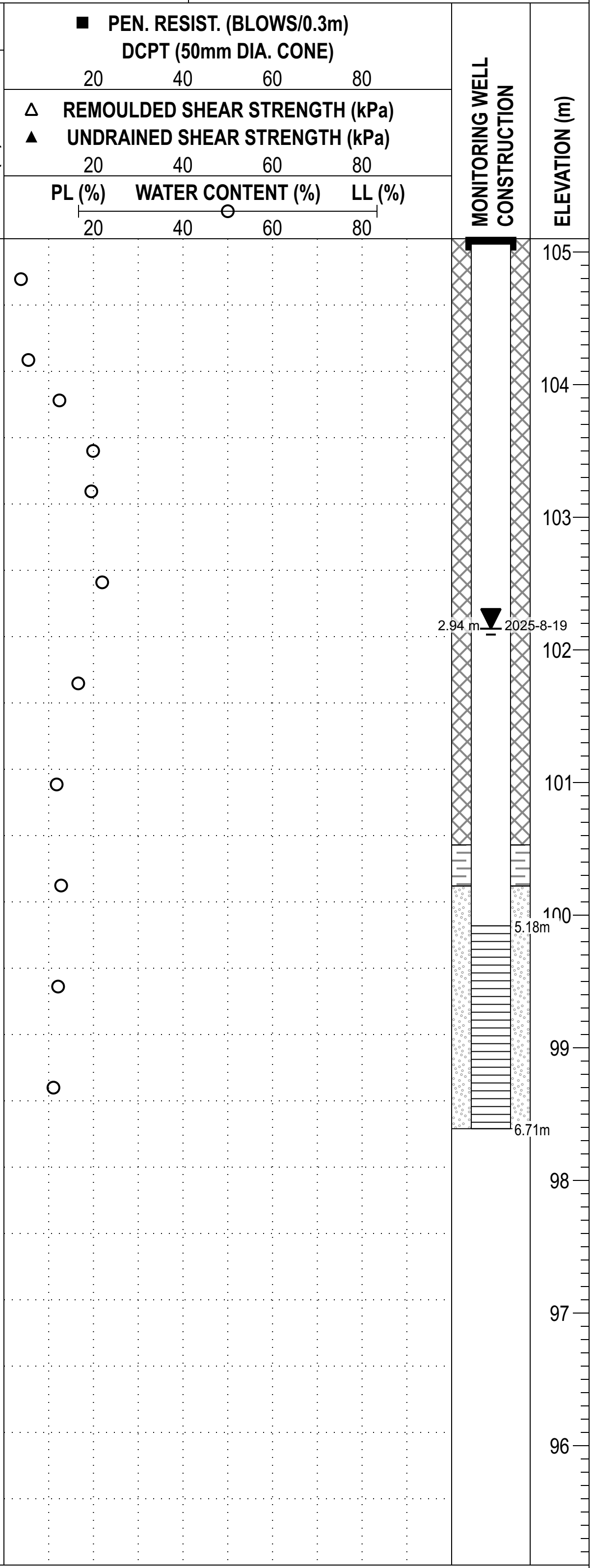
DATE: August 11, 2025

FILE NO. : PG7332

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)
GROUND SURFACE		
FILL: Brown silty sand, some organics, trace crushed stone 0.69m [104.41m]		
Compact, brown **SILTY SAND** 1.83m [103.27m]		
Compact to loose, brown **SANDY SILT** 2.21m [102.89m]		
Compact, grey **SILTY SAND** - Trace clay by 3.1 m depth 3.73m [101.37m]		
GLACIAL TILL: Loose to very dense, grey silty sand, gravel, cobbles and boulders - Some clay by 4.9 m depth - Silt content decreasing with depth 6.71m [98.39m]		
End of Borehole (GWL at 2.94 m depth - August 19, 2025)		

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			4
SS 2	79	7-9-11-11 20	5 12
SS 3	58	6-8-7-8 15	20 20
SS 4	58	4-6-9-7 15	22
SS 5	62	2-2-2-2 4	17
SS 6	100	6-7-3-2 10	12
SS 7	75	3-6-12-12 18	13
SS 8	83	8-12-12-11 24	12
SS 9	79	14-21-40-50 61/0.03	11



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

100 Nipissing Court, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 348212.22 **NORTHING:** 5017270.48 **ELEVATION:** 105.33

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: August 11, 2025

FILE NO. : PG7332

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand, with organics

0.69m [104.64m]

Compact, brown silty sand

0.89m [104.44m]

Compact to loose, brown **SILTY SAND**, trace clay

- Silt content increasing with depth

3.35m [101.98m]

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

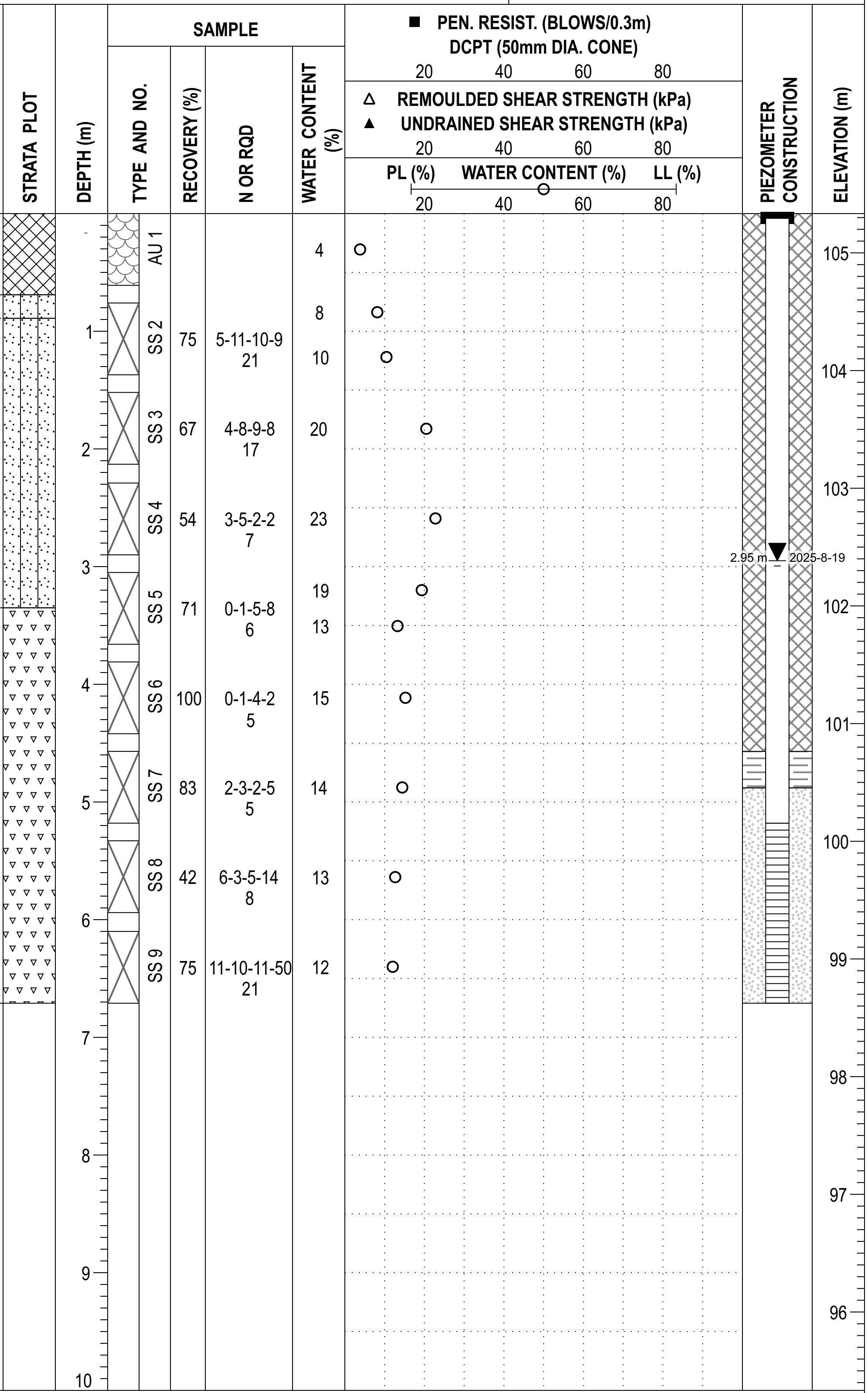
- Some clay, with gravel by 5.5 m depth

6.71m [98.62m]

End of Borehole

(GWL at 2.95 m depth - August 19, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			4
SS 2	75	5-11-10-9 21	8 10
SS 3	67	4-8-9-8 17	20
SS 4	54	3-5-2-2 7	23
SS 5	71	0-1-5-8 6	19 13
SS 6	100	0-1-4-2 5	15
SS 7	83	2-3-2-5 5	14
SS 8	42	6-3-5-14 8	13
SS 9	75	11-10-11-50 21	12



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

100 Nipissing Court, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 348232.45 **NORTHING:** 5017247.98 **ELEVATION:** 105.24

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

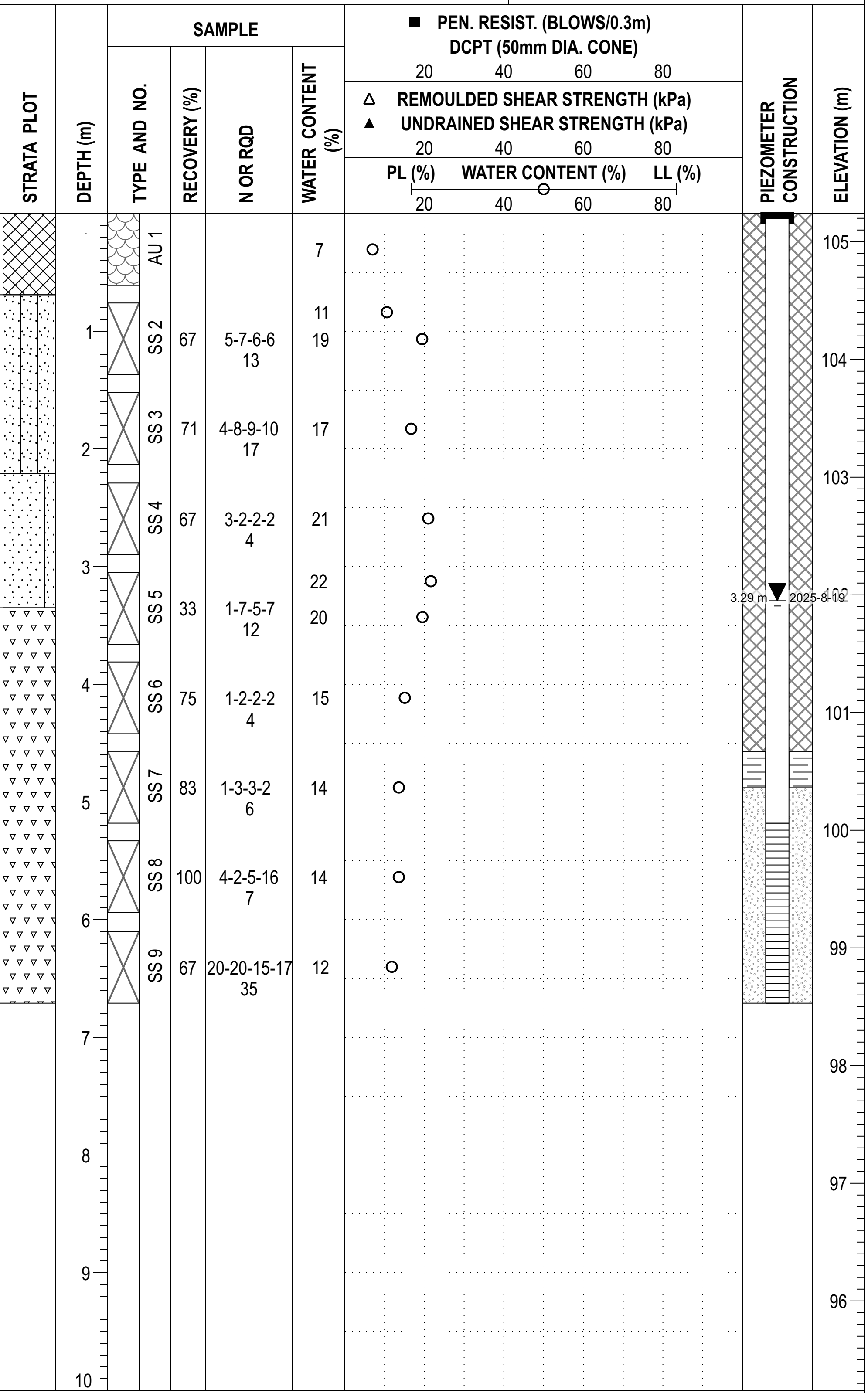
DATE: August 11, 2025

FILE NO. : **PG7332**

HOLE NO. : **BH 3-25**

SAMPLE DESCRIPTION	
GROUND SURFACE	
FILL: Brown silty sand, with organics, trace crushed stone	0.69m [104.55m]
Compact, brown **SILTY SAND**	2.21m [103.03m]
Loose to compact, brown **SANDY SILT**	3.35m [101.89m]
GLACIAL TILL: Very loose to dense, grey silty sand, trace clay and some gravel, cobbles and boulders - Silt content decreasing with depth	6.71m [98.53m]
End of Borehole (GWL at 3.29 m depth - August 19, 2025)	

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			7
SS 2	67	5-7-6-6 13	11 19
SS 3	71	4-8-9-10 17	17
SS 4	67	3-2-2-2 4	21
SS 5	33	1-7-5-7 12	22 20
SS 6	75	1-2-2-2 4	15
SS 7	83	1-3-3-2 6	14
SS 8	100	4-2-5-16 7	14
SS 9	67	20-20-15-17 35	12



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

100 Nipissing Court, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 348228.85 **NORTHING:** 5017228.16 **ELEVATION:** 105.37

PROJECT: Proposed Commercial Development

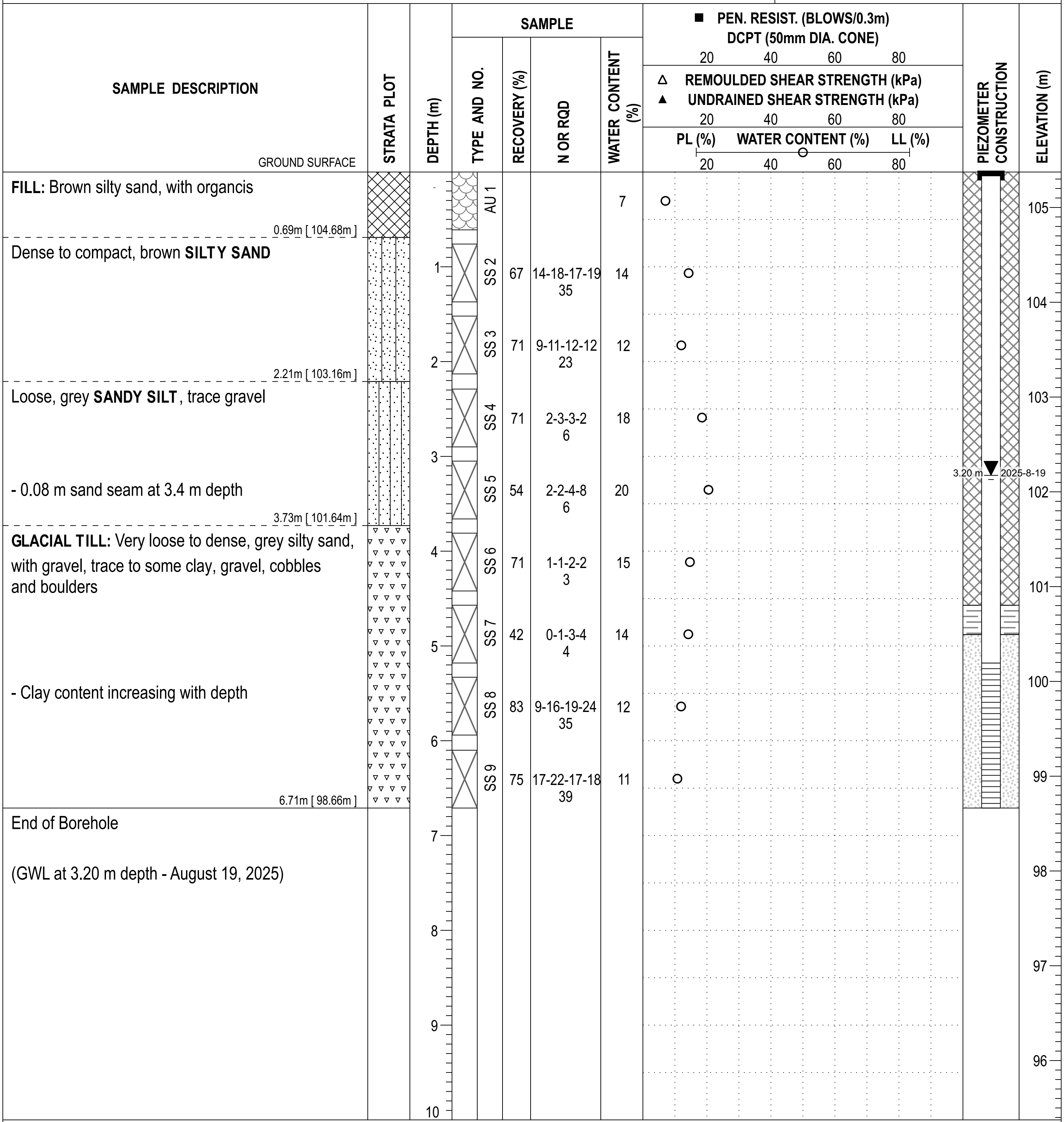
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: August 12, 2025

FILE NO. : PG7332

HOLE NO. : BH 4-25



FILL: Brown silty sand, with organcis

0.69m [104.68m]

Dense to compact, brown **SILTY SAND**

2.21m [103.16m]

Loose, grey **SANDY SILT** , trace gravel

- 0.08 m sand seam at 3.4 m depth

3.73m [101.64m]

GLACIAL TILL: Very loose to dense, grey silty sand, with gravel, trace to some clay, gravel, cobbles and boulders

- Clay content increasing with depth

6.71m [98.66m]

End of Borehole

(GWL at 3.20 m depth - August 19, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			7
SS 2	67	14-18-17-19 35	14
SS 3	71	9-11-12-12 23	12
SS 4	71	2-3-3-2 6	18
SS 5	54	2-2-4-8 6	20
SS 6	71	1-1-2-2 3	15
SS 7	42	0-1-3-4 4	14
SS 8	83	9-16-19-24 35	12
SS 9	75	17-22-17-18 39	11

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

100 Nipissing Court, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 348205.90 **NORTHING:** 5017246.44 **ELEVATION:** 105.27

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: August 12, 2025

FILE NO. : PG7332

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand, with organics

0.69m [104.58m]

Compact, brown **SILTY SAND**

1.45m [103.82m]

Compact, brown **SANDY SILT**

2.21m [103.06m]

Loose, grey **SANDY SILT**

3.51m [101.76m]

GLACIAL TILL: Very loose to compact, grey silty sand, with gravel, trace to some clay, cobbles and boulders

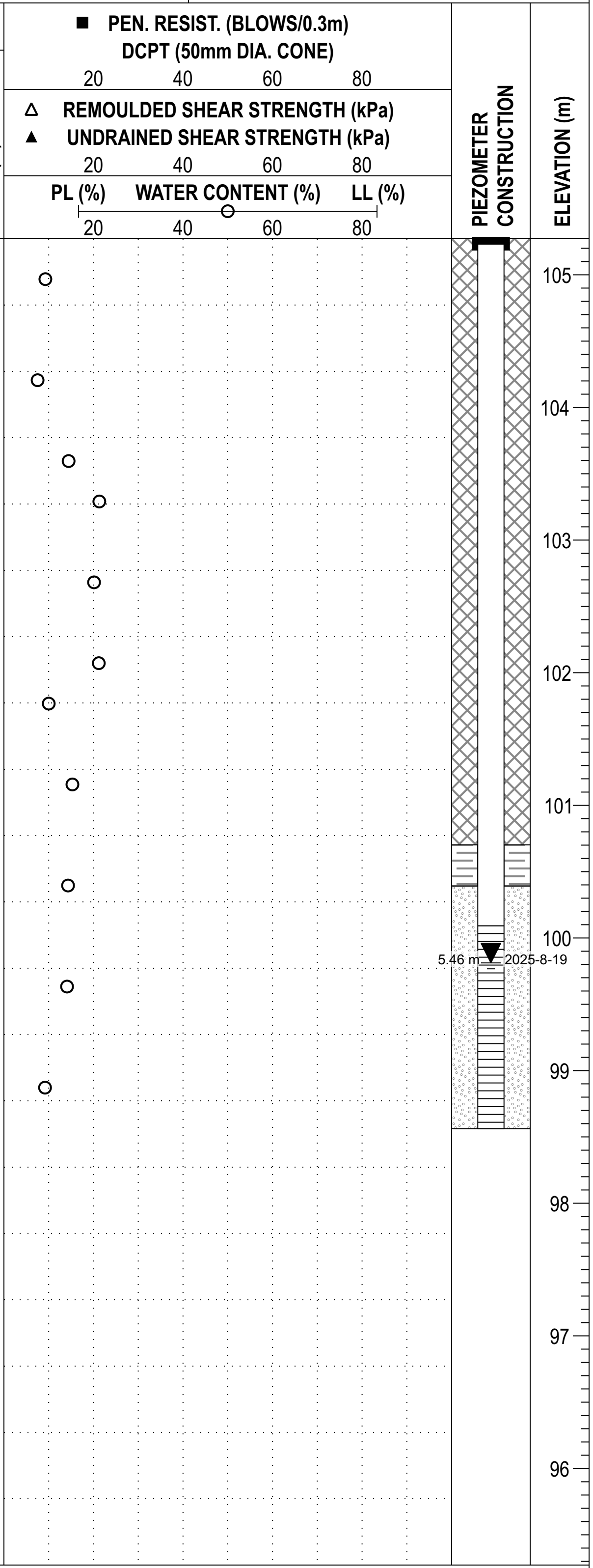
- Clay content increasing with depth

6.71m [98.56m]

End of Borehole

(GWL at 5.46 m depth - August 19, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			9
SS 2	75	9-14-15-15 29	8
SS 3	75	7-9-9-5 18	14 21
SS 4	50	2-2-4-3 6	20
SS 5	79	3-4-2-1 6	21 10
SS 6	67	0-2-3-2 5	15
SS 7	83	0-2-4-5 6	14
SS 8	75	7-3-6-7 9	14
SS 9	67	5-11-16-29 27	9



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

100 Nipissing Court, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 348186.33 **NORTHING:** 5017276.04 **ELEVATION:** 105.50

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: August 12, 2025

FILE NO. : PG7332

HOLE NO. : BH 6-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand, with organcis and crushed stone

0.69m [104.81m]

Compact to very dense, brown **SILTY SAND**

2.21m [103.29m]

Loose, grey **SANDY SILT**

2.97m [102.53m]

GLACIAL TILL: Loose to compact, grey silty sand, trace clay, gravel, cobbles and boulders

- Trace gravel to with gravel, some clay

6.71m [98.79m]

Dynamic Cone Penetration Test commenced at 6.71 m depth

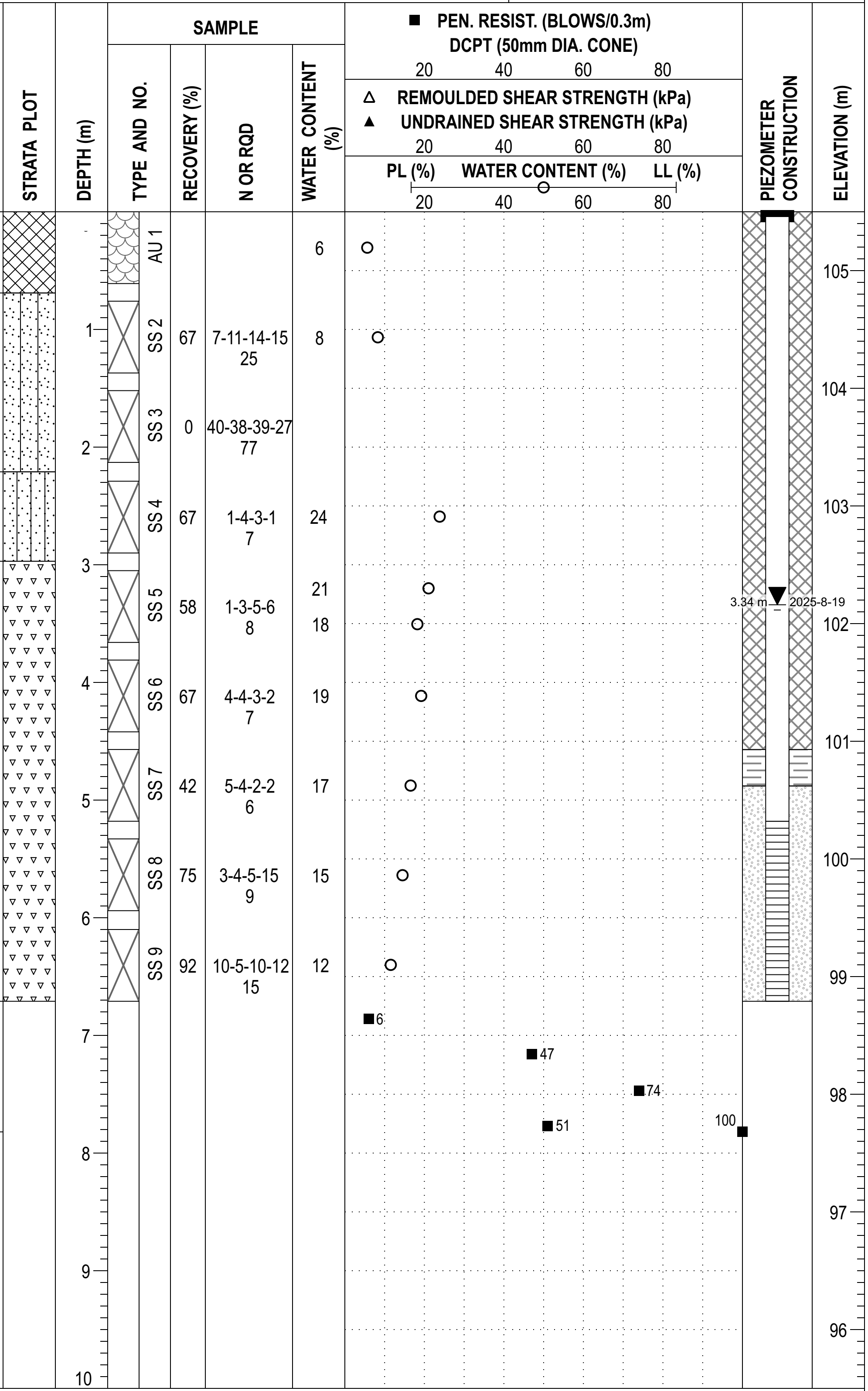
7.82m [97.68m]

End of Borehole

Practical refusal to DCPT at 7.82 m depth

(GWL at 3.34 m depth - August 19, 2025)

SAMPLE			
TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			6
SS 2	67	7-11-14-15 25	8
SS 3	0	40-38-39-27 77	
SS 4	67	1-4-3-1 7	24
SS 5	58	1-3-5-6 8	21 18
SS 6	67	4-4-3-2 7	19
SS 7	42	5-4-2-2 6	17
SS 8	75	3-4-5-15 9	15
SS 9	92	10-5-10-12 15	12



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 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

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

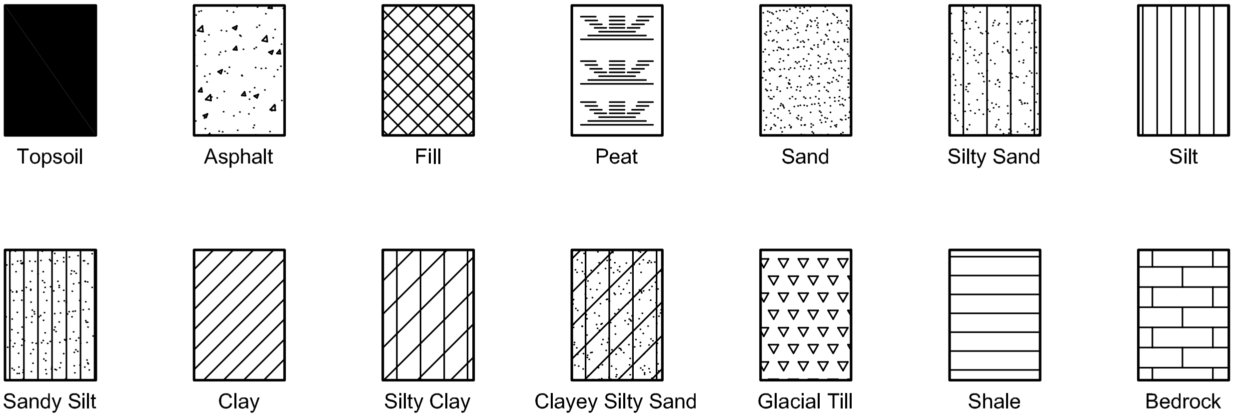
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

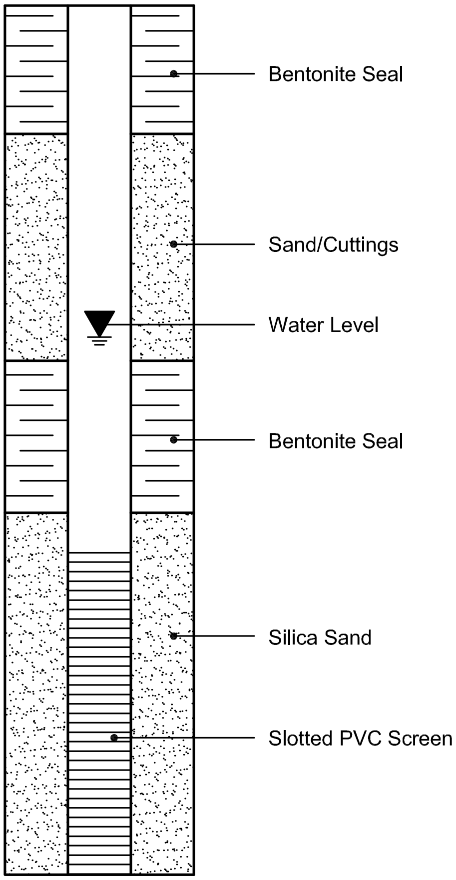
SYMBOLS AND TERMS (continued)

STRATA PLOT

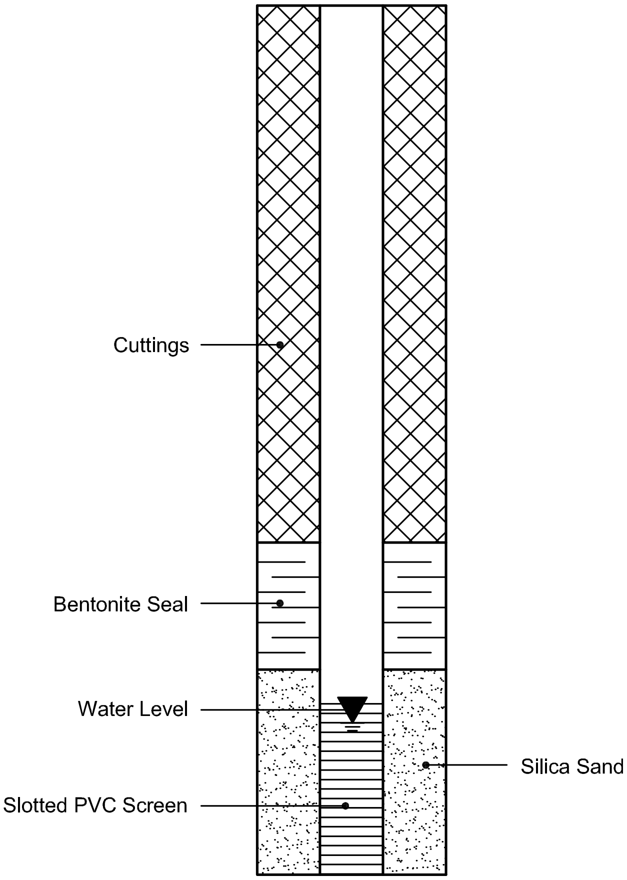


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 21-Aug-2025

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

Order Date: 15-Aug-2025

Client PO: 63832

Project Description: PG7332

	Client ID:	BH2-25 SS3	-	-	-		
	Sample Date:	11-Aug-25 00:00	-	-	-	-	-
	Sample ID:	2534011-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	83.3	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.84	-	-	-	-	-
Resistivity	0.1 Ohm.m	122	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	<10	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG7332-1 – TEST HOLE LOCATION PLAN

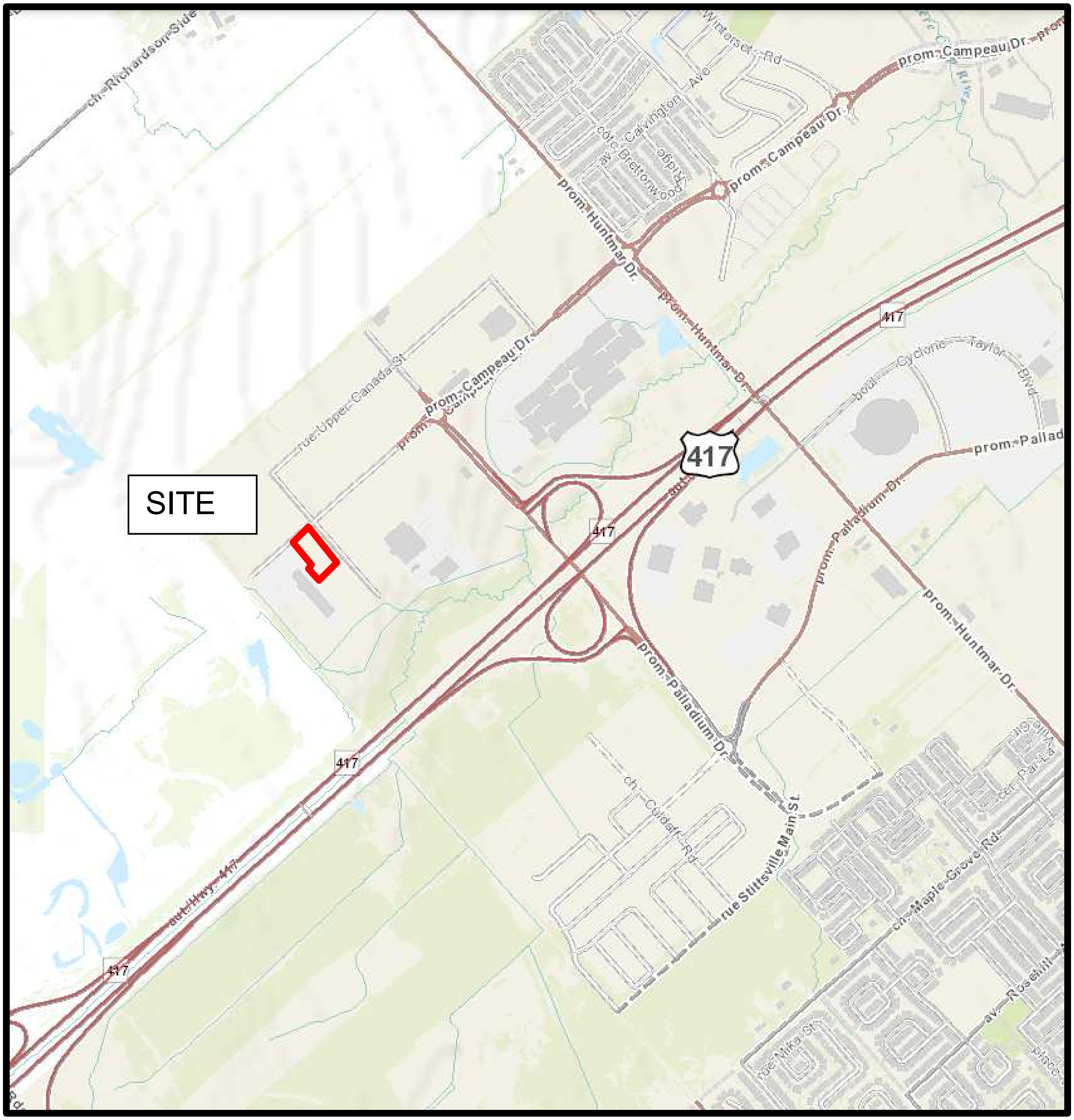


FIGURE 1

KEY PLAN

PATERSON GROUP

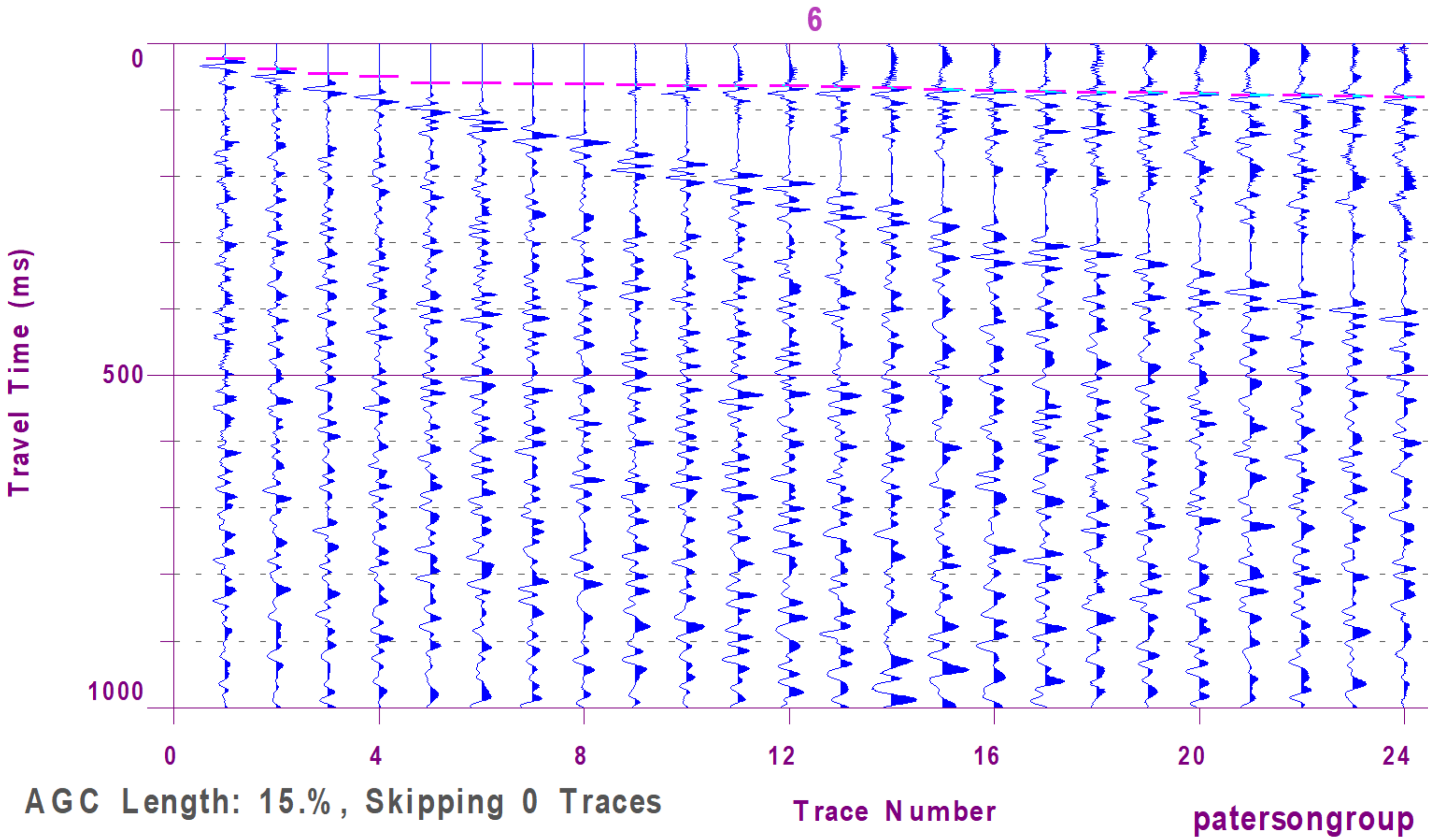


Figure 2 – Shear Wave Velocity Profile at Shot Location -4.5 m

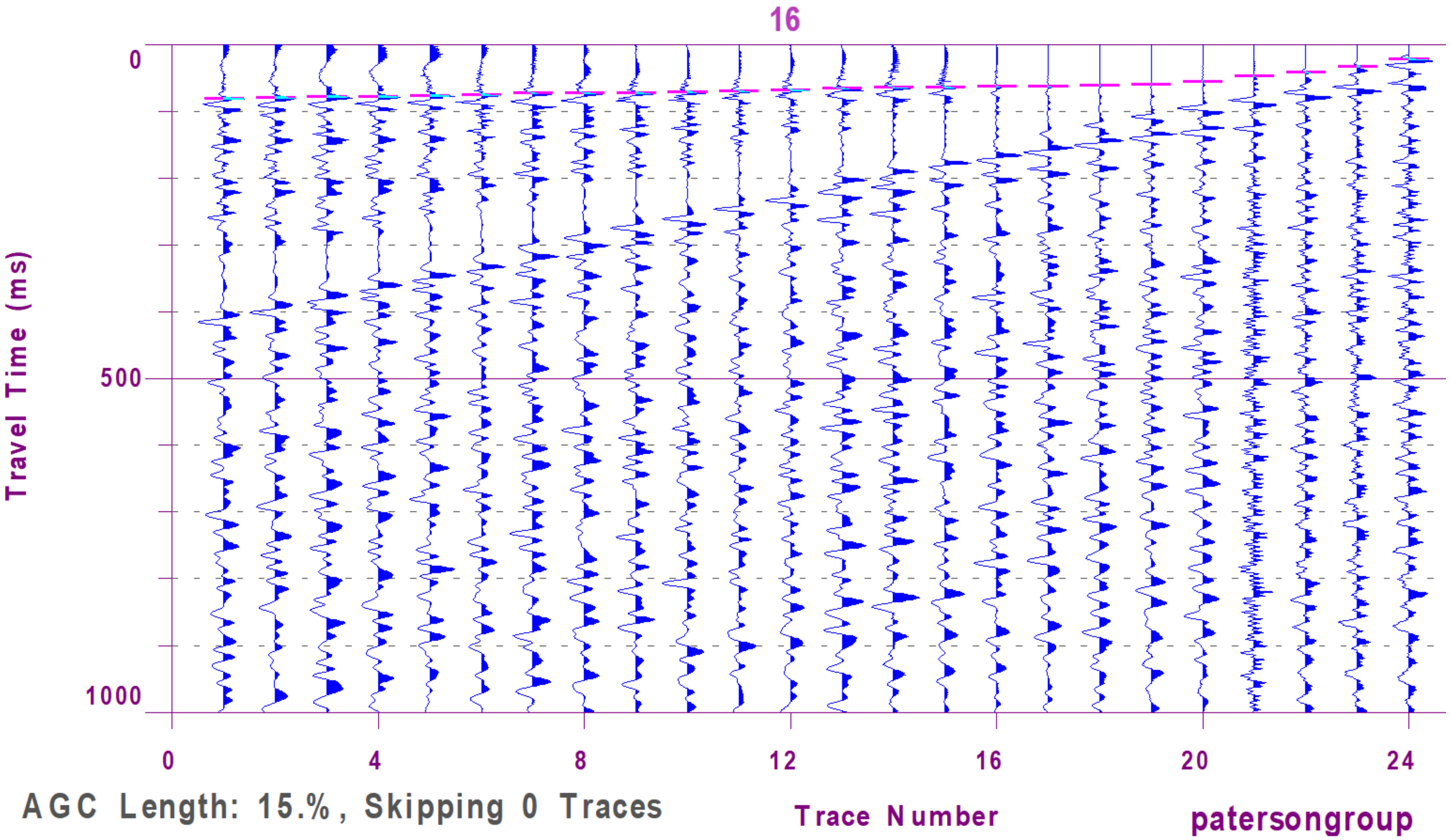
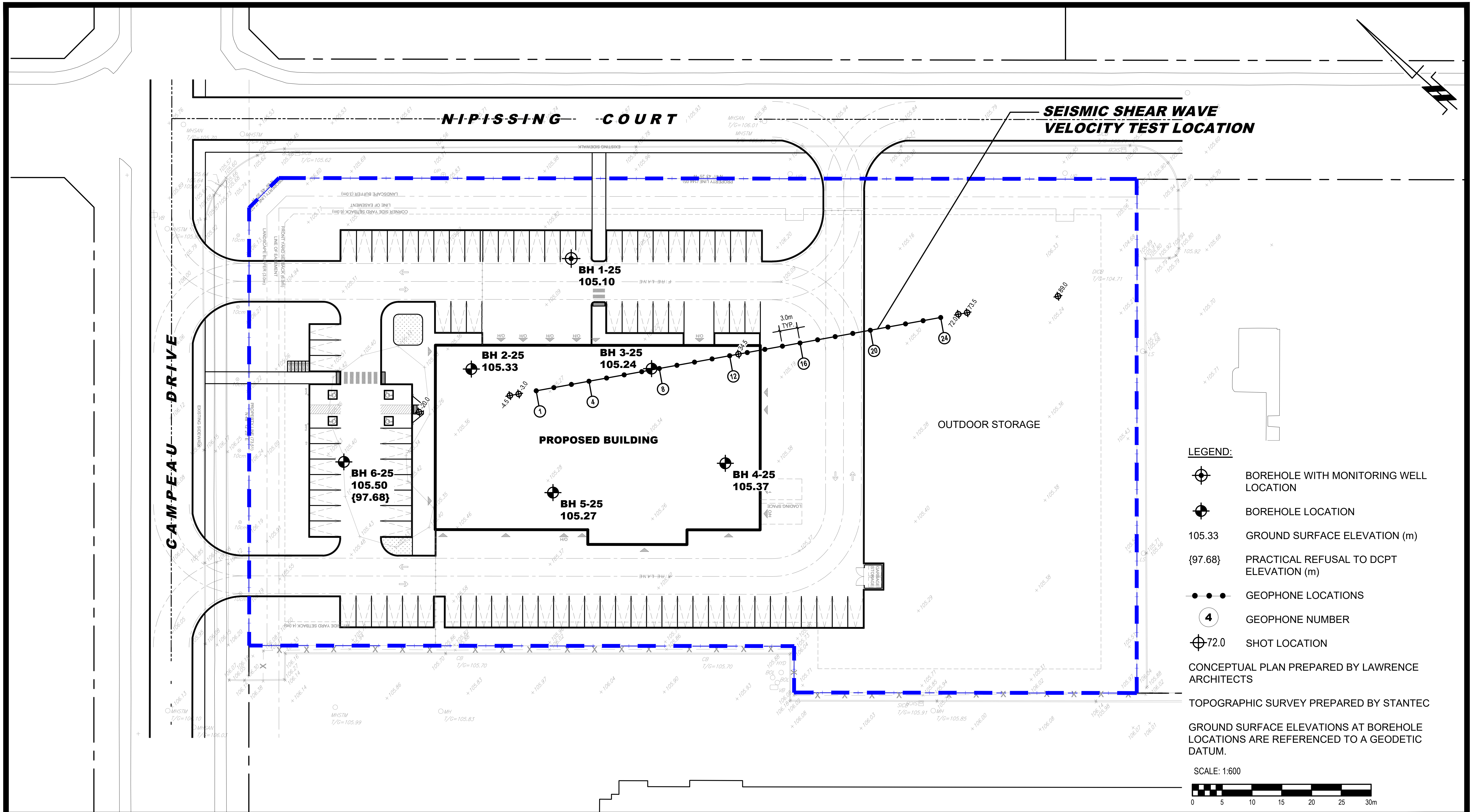


Figure 3 – Shear Wave Velocity Profile at Shot Location 73.5 m



LEGEND:

- ⌖ BOREHOLE WITH MONITORING WELL LOCATION
- ⊕ BOREHOLE LOCATION
- 105.33 GROUND SURFACE ELEVATION (m)
- {97.68} PRACTICAL REFUSAL TO DCPT ELEVATION (m)
- ● ● ● GEOPHONE LOCATIONS
- ④ GEOPHONE NUMBER
- ⌖72.0 SHOT LOCATION

CONCEPTUAL PLAN PREPARED BY LAWRENCE ARCHITECTS

TOPOGRAPHIC SURVEY PREPARED BY STANTEC

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

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

NO.	REVISIONS	DATE	INITIAL

MYERS AUTOMOTIVE GROUP
GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
100 NIPISSING COURT
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 08/2025
Drawn by: ZS
Report No.: PG7332-1
Checked by: UR
Dwg. No.: PG7332-1
Approved by: DP
Revision No.:

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11x17