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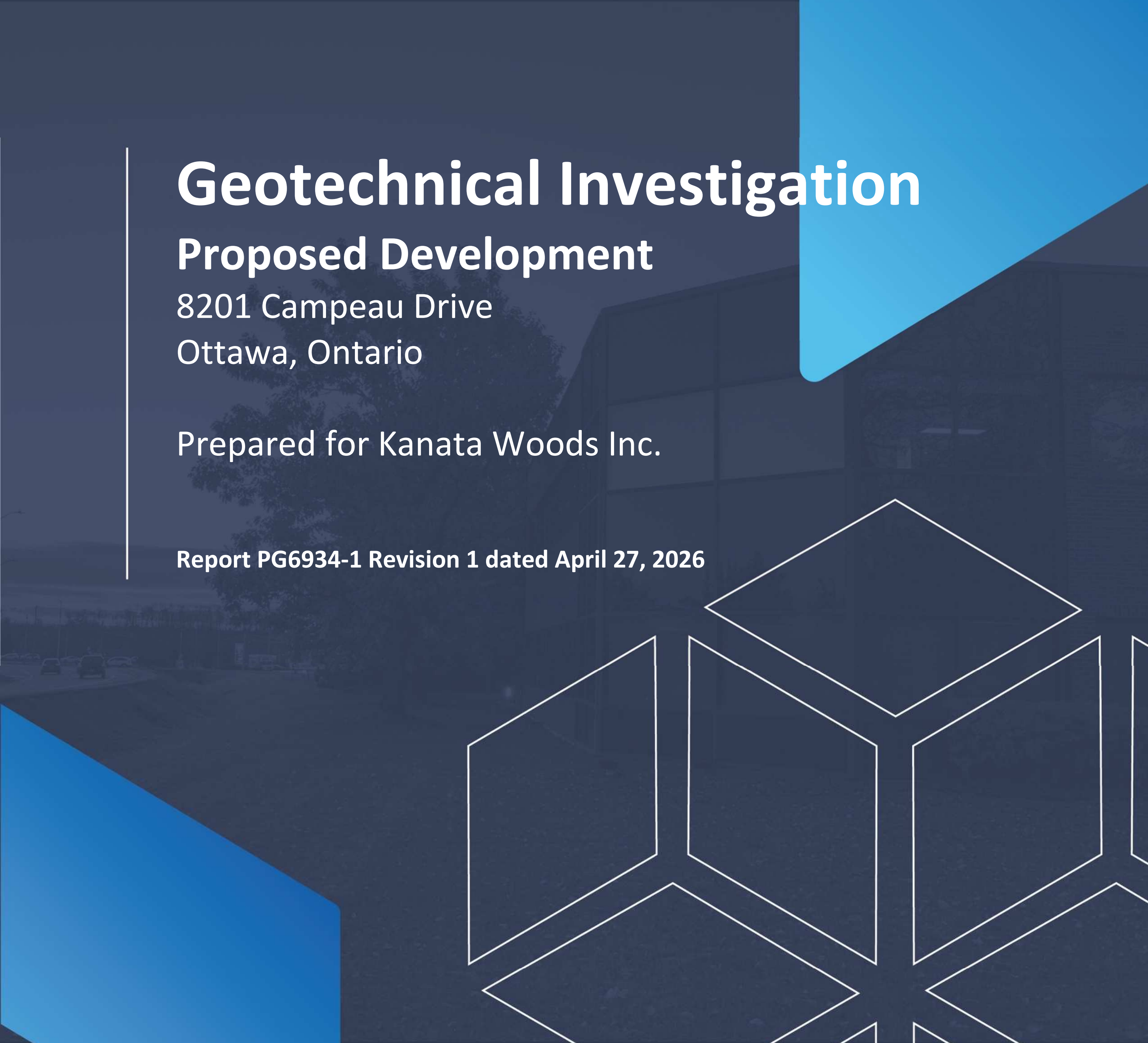

Geotechnical Investigation

Proposed Development

8201 Campeau Drive
Ottawa, Ontario

Prepared for Kanata Woods Inc.

Report PG6934-1 Revision 1 dated April 27, 2026

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Kanata Woods Inc. to conduct a geotechnical investigation for the proposed development to be located at 8201 Campeau Drive 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 existing subsoil and groundwater information at this site by means of test holes, and to

❒ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

2.0 Proposed Development

Based on our review of the available drawings, it is understood that the northwestern portion of the proposed development will consist of 2 mid-rise structures (Buildings A and B) with 3 levels of shared underground parking. Further, the proposed development will include asphalt-paved access lanes, loading areas, outdoor amenities, and landscaped areas.

It is expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The current geotechnical investigation was completed during the period of September 4 through 10, 2024. The investigation consisted of a total of 5 boreholes (BH 1-24 to BH 5-24) advanced to a maximum depth of 24 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access.

A previous geotechnical investigation was also conducted at this site by others in January 2024 which consisted of 2 boreholes (BH 1 & BH 2) and 44 rock probeholes (P1 to P44).

The boreholes were advanced using a track-mounted drill rig operated by a two-person crew, while the rock probeholes were drilled using a pneumatic drill rig. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The borehole drilling procedure consisted of augering to the required depths at the selected borehole locations, and sampling and testing the overburden.

The approximate locations of the test holes are shown on Drawing PG6934-1 - Test Hole Location Plan included in Appendix 2.

Sampling and In Situ Testing

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

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

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils.

The overburden thickness was evaluated by completing dynamic cone penetration testing (DCPT) at boreholes BH 1 & BH 2, by others. The DCPT testing consisted 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.

Rock samples were recovered from borehole BH 5-24 using a core barrel and diamond drilling techniques. The depths at which the rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

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 Soil Profile and Test Data sheet for borehole BH 5-24. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the 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.

Groundwater

Monitoring wells were installed at boreholes BH 2-24 and BH 4-24 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

The installed monitoring well should be decommissioned in accordance with Ontario Regulations O.Reg 903 by a qualified licensed well technician and prior to construction.

3.2 Field Survey

The test hole locations, and the ground surface elevation at each test hole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG6934-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Further, 2 samples were submitted for Atterberg limits testing, 1 sample was submitted for grain size distribution testing, and 1 sample was submitted for shrinkage testing. The results are discussed in Section 4.2.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant, with a grassed surface and scattered trees. The site is bordered by Campeau Drive to the north, Didsbury Road to the east, Roger Neilson Way to the south, and an existing commercial development to the west. The ground surface across the site generally slopes downward from about geodetic elevation 98 at the eastern boundary of the site, to geodetic elevation 94 m at the western boundary.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of about 0.15 to 0.3 m of topsoil underlain by fill, silty sand, silty clay, and glacial till. The fill was generally observed to consist of loose, brown silty sand to sand with varying amounts of clay and organics.

A silty sand deposit was encountered below the fill and/or topsoil at approximate depths of 0.5 to 3 m below the existing ground surface. This deposit was generally observed to consist of a very loose to compact, brown to grey silty sand with trace clay.

A firm to stiff, grey silty clay deposit was encountered underlying the silty sand at approximate depths of 3.7 to 8.5 m below current side grades.

Glacial till was encountered within boreholes BH 4-24 and BH 5-24 at depths of about 10 m and 19.5 m, respectively. The glacial till was observed to consist of grey silty sand to silty clay with varying amounts of gravel, cobbles, and boulders.

Bedrock

Bedrock varies in depth from about 12 m in the southwest corner of the site, to over 30 m in depth at the northeast corner Where bedrock was cored at borehole BH 5-24, it was observed to consist of excellent quality limestone.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil and bedrock profiles encountered at each test hole location.

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was completed on 1 selected soil sample. The results of the grain size analysis are summarized in Table 1 below and are presented in Appendix 1.

Table 1 – Summary of Grain Size Distribution Analysis

Borehole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 4-24	SS7	0	2.4	44.6	53.0

Atterberg Limit Tests

A total of 2 silty clay samples were submitted for Atterberg limits testing. The test results indicate that the silty clay is generally classified as an Inorganic Clay of High Plasticity (CL). The results are summarized in Table 2 below.

Table 2 – Summary of Atterberg Limits Results

Borehole	Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 3-24	SS7	4.5 - 5.2	27	14	13	CL
BH 5-24	SS3	1.5 – 2.1	45	21	24	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; CL: Clay of Low Plasticity

Shrinkage Test

The results of the shrinkage limit test indicate a shrinkage limit of 37.245 and a shrinkage ratio of 1.894.

4.3 Groundwater

Groundwater level readings were measured in the monitoring wells on September 13, 2024, and are summarized in Table 3 below.

Table 3 – Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 2-24	98.55	5.53	93.02	September 13, 2024
BH 4-24	95.12	4.94	90.18	

Note: The ground surface elevation at each borehole location was surveyed by Paterson and was referenced to a geodetic datum.

It should be noted that the long-term groundwater level can also be estimated based on the recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is anticipated at a depth of approximately 4 to 5 m below ground surface.

However, groundwater levels are subject to seasonal fluctuations and 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 mid-rise buildings be supported on raft foundations bearing on the undisturbed, firm to stiff silty clay.

Due to the presence of a silty clay layer, the site is subjected to a permissible grade raise restriction. The permissible grade raise recommendations are discussed in Section 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

Fill used for grading beneath the proposed buildings 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. 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 building and paved areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material’s SPMDD.

Protection of Subgrade

Since the subgrade material will most likely consist of a silty clay deposit, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the concrete mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

Pressure Relief Chamber

The installation of a pressure relief chamber in conjunction with the raft foundation should be considered along with collection pipes within the silty clay deposit. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber. It is suggested that the pressure relief chamber be incorporated into the lowest section of the lowest level of underground parking. Once the pressure relief chamber and associated piping is installed, the proposed raft slab can be constructed. The purpose of the pressure relief chamber will be as follows:

- Manage any water infiltration along the founding surface during the excavation.

- Manage the water infiltration during the pouring of the raft slab to prevent water flow in the fresh concrete.

- Manage water infiltration below the raft slab until sufficient load is applied to resist any potential hydrostatic uplift.

- Regulate the discharge valve to control water infiltration once the raft slab is in place and over the long term to manage the hydrostatic pressure to permit any repairs associated with any water infiltration.

- Once sufficient load is applied to the raft slab, the pressure relief valve will be fully closed to prevent any further dewatering.

With the fully closed valve within the pressure relief chamber and a perfectly watertight foundation, it is expected that a maximum hydrostatic pressure of 25 kPa will be developed over the long-term, and should be incorporated in the design of the raft foundation and the foundation walls.

5.3 Foundation Design

Raft Foundation

As noted above, it is recommended to use a raft foundation for support of the proposed mid-rise buildings. For the 3 underground parking levels, it is anticipated that excavation will extend to depths of about 9 to 10 m below the existing ground surface.

The maximum serviceability limit states (SLS) contact pressure can be taken as **125 kPa** for the raft foundation bearing on the undisturbed, firm to stiff silty clay. It should be noted that the weight of the raft slab and everything above has to be included when designing with the aforementioned SLS values. 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. The factored bearing resistance (contact pressure) at ultimate limit states (ULS) can be taken as **200 kPa.** A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **5 MPa/m** for a contact pressure of 125 kPa. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed buildings can be designed using the above parameters and total and differential settlements of 25 and 20 mm, respectively, for a raft foundation.

Permissible Grade Raise Recommendations

Due to the presence of a silty clay deposit, a permissible grade raise restriction of **0.5 m** is recommended within 3 m of the proposed buildings' footprints, and **1.5 m** is recommended for grading at the remainder of the subject site.

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

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 soil bearing surface above the groundwater table when a plane extending horizontally and vertically from the bottom edge of the footing at a minimum of 1.5H:1V, passing through in situ soil of the same or higher capacity as the bearing medium soil.

5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed at the subject site to accurately determine the applicable seismic site classification for the proposed buildings in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided on Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as shown on Drawing PG6934-1 - 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 spikes attached to the geophone land case. The geophones were spaced at 3 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop 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 4 to 8 times at each shot location to improve signal to noise ratio. The shot locations were 25, 4.5 and 3 m away from the first and last geophones, and at the centre of the seismic array.

Data Processing and Interpretation

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

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. 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 expected to be a conservative estimate of the bedrock velocity. 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 **184 m/s**, while the bedrock shear wave velocity is **2,405 m/s**. Considering that the proposed building will be provided with 2 underground levels and based on the results of the seismic shear wave velocity test, it is assumed that the overburden thickness below underside of foundation will be **19 m**.

Based on this, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{19\ m}{184\ m/s} + \frac{11\ m}{2{,}405\ m/s}\right)}$$

$$V_{s30=}\ 278\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **278 m/s** for the proposed buildings. Therefore, a **Site Class X_{278}** is applicable for design of the proposed buildings as per Table 4.1.8.4.A of the OBC 2024. The proposed building foundations are anticipated to extend below any sand at this site, and therefore will not be impacted by liquefaction.

5.5 Basement Floor Slab

It is understood that the underground levels for the proposed buildings 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 where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction.

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

In consideration of the groundwater conditions encountered at the time of the field investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

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

Lateral Earth Pressures

The static horizontal earth pressure (po) can be calculated using a triangular earth pressure distribution equal to Ko·γ·H where:

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (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 Ottawa area is 0.338g for a Site Class X_{278} 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 Structure

Rigid Pavement Structure

It is recommended that the rigid pavement structure for the lower underground parking level consists 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. The flexible pavement structure presented in Table 6 should be used for at-grade access lanes and heavy loading parking areas.

Table 5 – Recommended Rigid Pavement Structure – Lower Parking Level

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

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Flexible Pavement Structure

The flexible pavement structure presented in Tables 6 and 7 should be used for car-only parking areas, and 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 Asphalt Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE – Either existing 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 Loading Area

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

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

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

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

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

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

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

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

Foundation Raft Slab Construction Joints

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

Underslab Drainage

Underslab drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 100 mm diameter perforated pipes be placed at approximate 6 m centres underlying the lowest level floor slab. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available for conventional backfilling, the backfill material against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials.

Pressure Relief Chamber

The pressure relief chamber will be used to control the groundwater infiltration and hydrostatic pressure created by tanking the lowest level of underground parking. To avoid uplift on the raft foundation slab prior to having sufficient loading to resist uplift, it is recommended that the water infiltration be pumped via the pressure relief chamber during construction.

The valve of the pressure relief chamber can be gradually closed during construction as the loading is applied to resist hydrostatic pressure. Once sufficient load is available to resist the full hydrostatic pressure, the valve of the pressure relief chamber can be adjusted and closed to minimize water infiltration volumes.

6.2 Protection of Footings Against Frost Action

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

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

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

6.3 Excavation Side Slopes

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

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. Deeper excavations may not be feasible to slope due to running sands, which were encountered during the geotechnical investigation. The subsurface soils are considered to be a 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

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

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

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

Due to the running sands encountered in the boreholes, it is recommended that the temporary shoring system consist of steel sheet piles which would be cantilevered, anchored or braced.

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

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

Table 8 – Soil Parameters for Shoring System Design

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

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

The hydrostatic groundwater pressure should be included to 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 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 material should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Service Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

It should generally be possible to re-use materials above the cover material if the operations are carried out in dry weather conditions.

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 controllable using open sumps, provided steel sheet piles are used as a temporary shoring system to create a cofferdam around the perimeter of the site. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

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), provided 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 Neighbouring Properties

As the proposed buildings will be founded below the long-term groundwater level, a groundwater infiltration control system has been recommended to mitigate the effects of groundwater infiltration. Any long-term dewatering of the site will be minimal and should have no adverse effects to the surrounding buildings or structures. Further, use of steel sheet piles as the temporary shoring system should mitigate dewatering beyond the site boundaries during the excavation and foundation construction.

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 indicate 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 to very aggressive corrosive environment.

6.8 Tree Planting Restrictions

Due to the anticipated foundation depths, tree roots will not extend below the proposed building foundations. Accordingly, there are no applicable tree planting restrictions for the proposed development, from a geotechnical perspective.

7.0 Recommendations

A materials testing and observation services program is also a requirement for the foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- Review of the geotechnical aspects of the excavation contractor’s design of the temporary shoring.
- Review of the final Grading Plan, 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 materials.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

All excess soils 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 Kanata Woods 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.

Scott Dennis

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

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
April 27, 2026
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Report Distribution:

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- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

ROCK PROBEHOLE SUMMARY BY OTHERS

ATTERBERG LIMITS TESTING RESULTS

GRAIN SIZE DISTRIBUTION TESTING RESULTS

SHRINKAGE TESTING RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350196.88 **NORTHING:** 5018629.61 **ELEVATION:** 95.42

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 04, 2024

FILE NO. : PG6934

HOLE NO. : BH 1-24

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL Loose, brown, silty sand, with organics

0.15m [95.27m]

Loose, brown, **SILTY SAND**, trace clay

- Trace sea shells at 1.83 m depth

- Grey by 5.03 m depth

5.26m [90.16m]

Firm, grey **SILTY CLAY**, with some sand and gravel

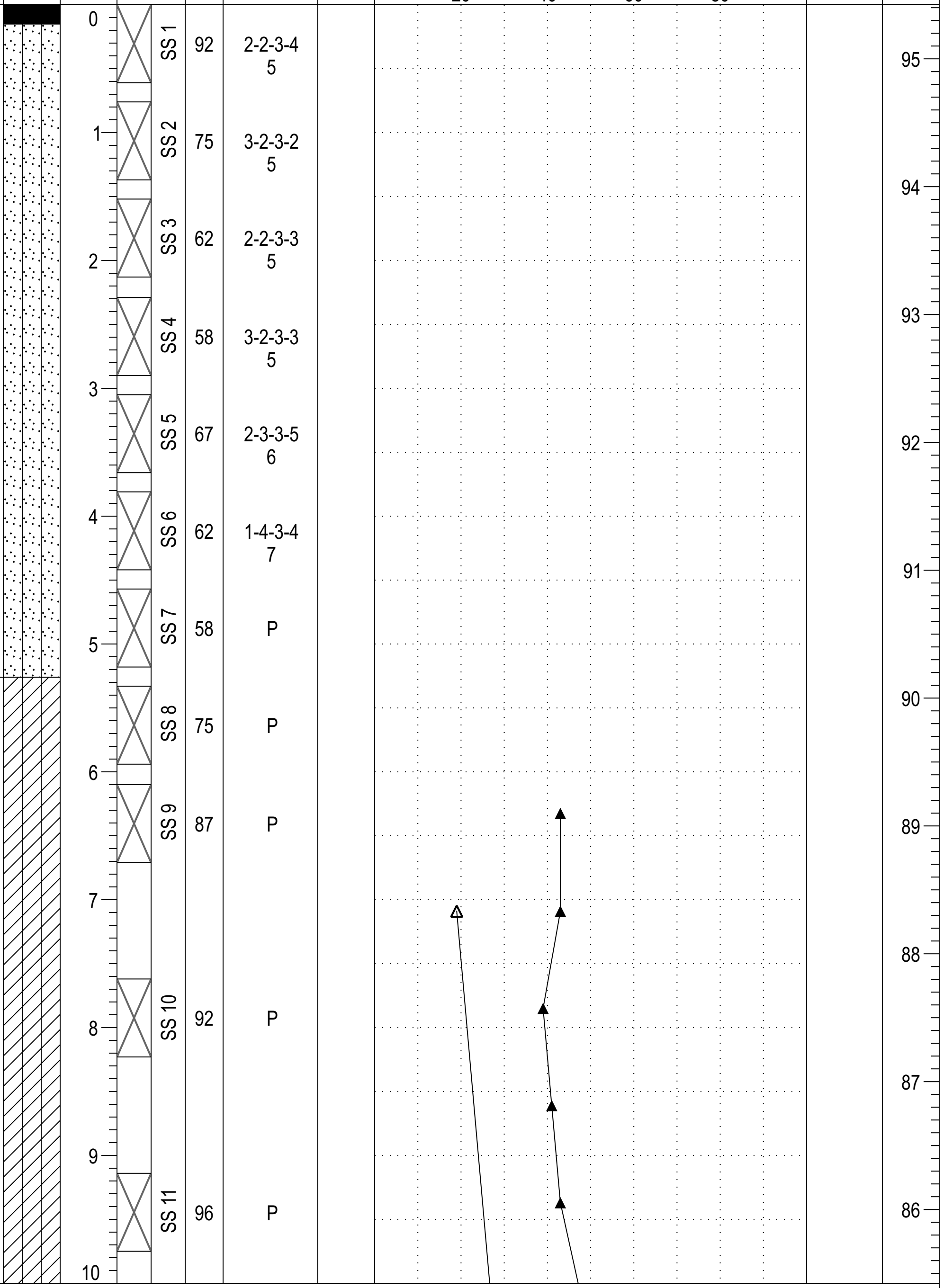
- Trace sand and sea shells at 6.65 m depth

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	92	2-2-3-4 5	
SS 2	75	3-2-3-2 5	
SS 3	62	2-2-3-3 5	
SS 4	58	3-2-3-3 5	
SS 5	67	2-3-3-5 6	
SS 6	62	1-4-3-4 7	
SS 7	58	P	
SS 8	75	P	
SS 9	87	P	
SS 10	92	P	
SS 11	96	P	

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

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

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



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350196.88 **NORTHING:** 5018629.61 **ELEVATION:** 95.42

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

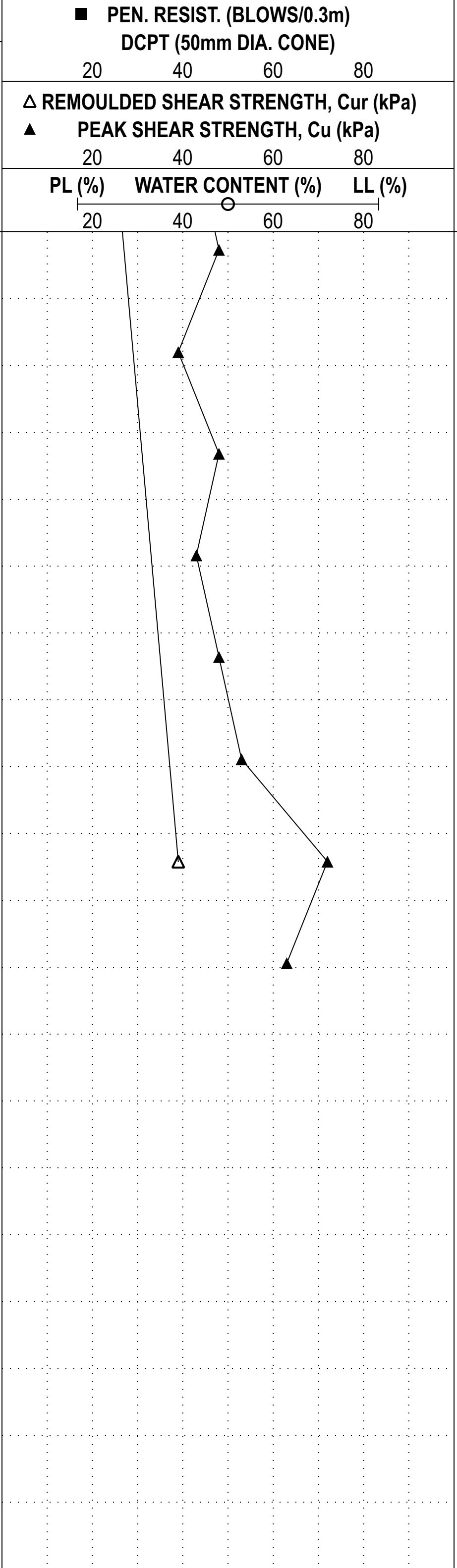
REMARKS:

DATE: September 04, 2024

FILE NO. : **PG6934**

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

SAMPLE DESCRIPTION GROUND SURFACE	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
	▨	10						
	▨	11	SS 12	100	P			84.5
	▨	12	SS 13	100	P			83.0
- Stiff at 13.72 m depth	▨	14	SS 14	100	P			81.5
15.85m [79.57m]	▨	15	SS 15	75	P			80.0
End of Borehole		16						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350379.23 **NORTHING:** 5018606.18 **ELEVATION:** 98.55

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 04, 2024

FILE NO. : PG6934

HOLE NO. : BH 2-24

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL: Brown, silty fine sand, with Organics

0.25m [98.30m]

FILL: Loose, brown silty fine sand, trace clay

- Trace clay at 0.76 m depth

1.07m [97.48m]

Loose, brown, **SILTY SAND**, trace clay

- Trace clay at 2.21 m depth

- Running sand encountered from 5.40 m to 8.40 m depth

- Grey by 6.10 m depth

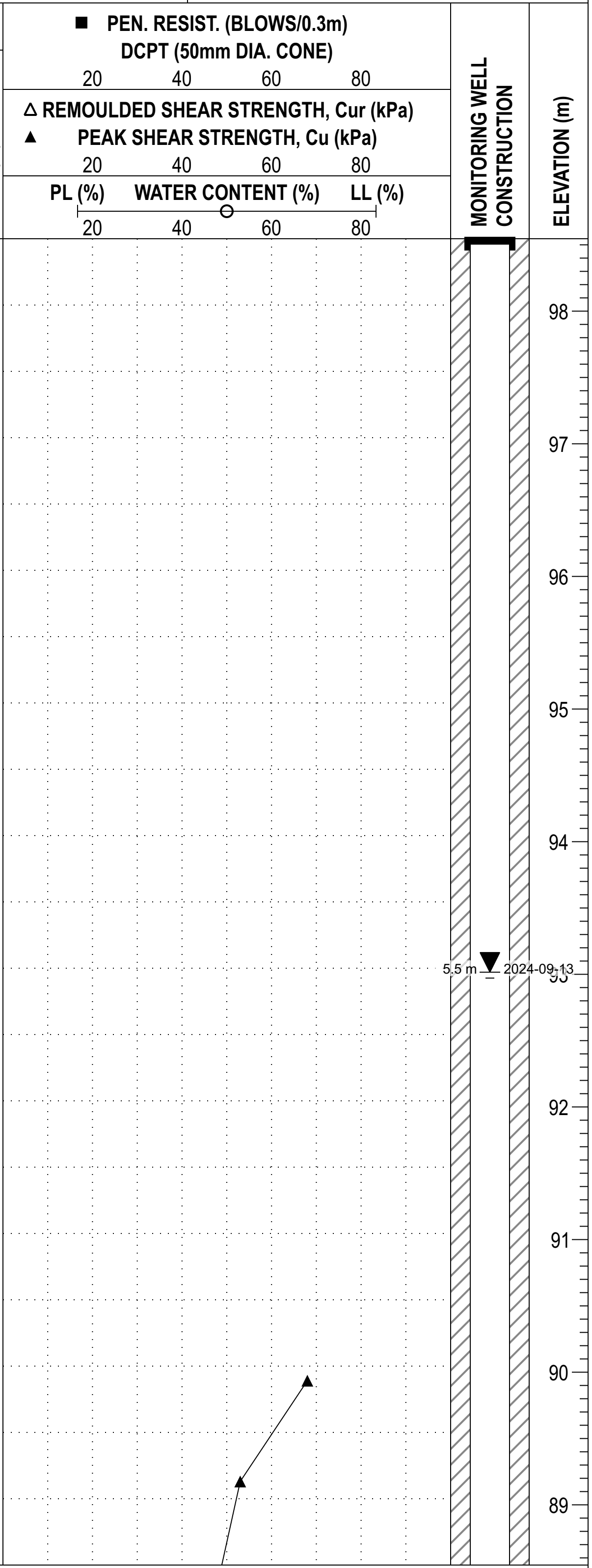
- Trace silty sand with clay at 7.16 m depth

8.46m [90.09m]

Stiff to firm, grey **SILTY CLAY**

- Trace sea shells at 9.14 m depth

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	50	1-3-3-2 6	
SS 2	46	2-4-5-5 9	
SS 3	62	2-3-2-3 5	
SS 4	46	1-3-3-4 6	
SS 5	21	2-4-5-5 9	
SS 6	62	2-3-3-4 6	
SS 7	33	3-5-8-9 13	
SS 8	75	1-2-3-2 5	
SS 9	37	None-2-2-2 4	
SS 10	29	None-1-3-2 4	
SS 11	71	1-1-1-1 2	
SS 12	75	P	
SS 13	25	P	



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SOIL PROFILE AND TEST DATA

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8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350379.23 **NORTHING:** 5018606.18 **ELEVATION:** 98.55

PROJECT: Proposed Development

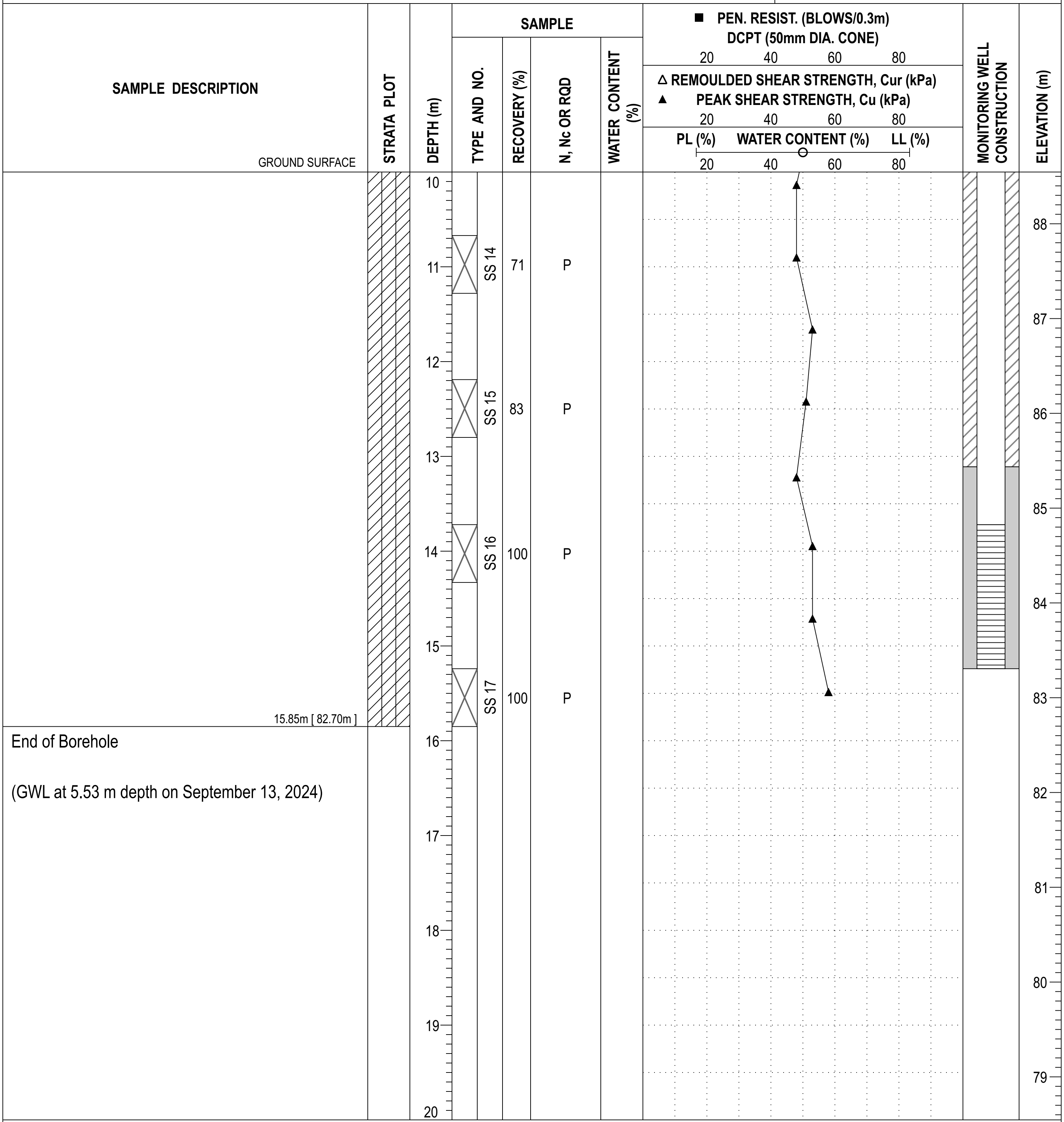
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 04, 2024

FILE NO. : PG6934

HOLE NO. : BH 2-24



TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD
SS 14	71	P
SS 15	83	P
SS 16	100	P
SS 17	100	P

15.85m [82.70m]

End of Borehole

(GWL at 5.53 m depth on September 13, 2024)

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350271.91 **NORTHING:** 5018526.69 **ELEVATION:** 95.09

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

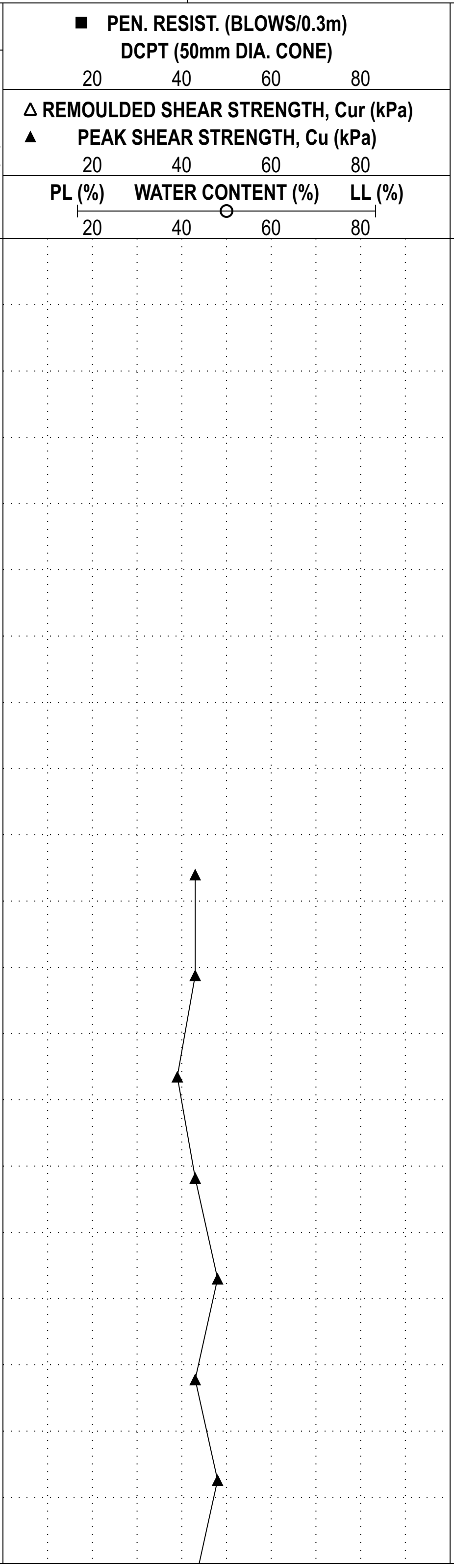
REMARKS:

DATE: September 05, 2024

FILE NO. : **PG6934**

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

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE	0					95
Loose, brown **SILTY SAND**, trace clay		SS 1	42	1-2-3-2 5		
	1	SS 2	37	2-2-3-3 5		94
- Grey by 2.13 m to 3.73 m depth - Running sand encountered from 2.13 m to 3.73 m depth	2	SS 3	46	2-3-3-4 6		93
		SS 4	62	1-1-1-2 2		
	3	SS 5	62	1-1-12-/ 13		92
3.73m [91.36m]						
Firm, grey **SILT CLAY**	4	SS 6	87	P		91
	5	SS 7	79	P		90
		SS 8	92	P		
	6	SS 9	71	P		89
	7					88
	8	SS 10	100	P		87
	9	SS 11	71	P		86
	10					



PIEZOMETER CONSTRUCTION

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350271.91 **NORTHING:** 5018526.69 **ELEVATION:** 95.09

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

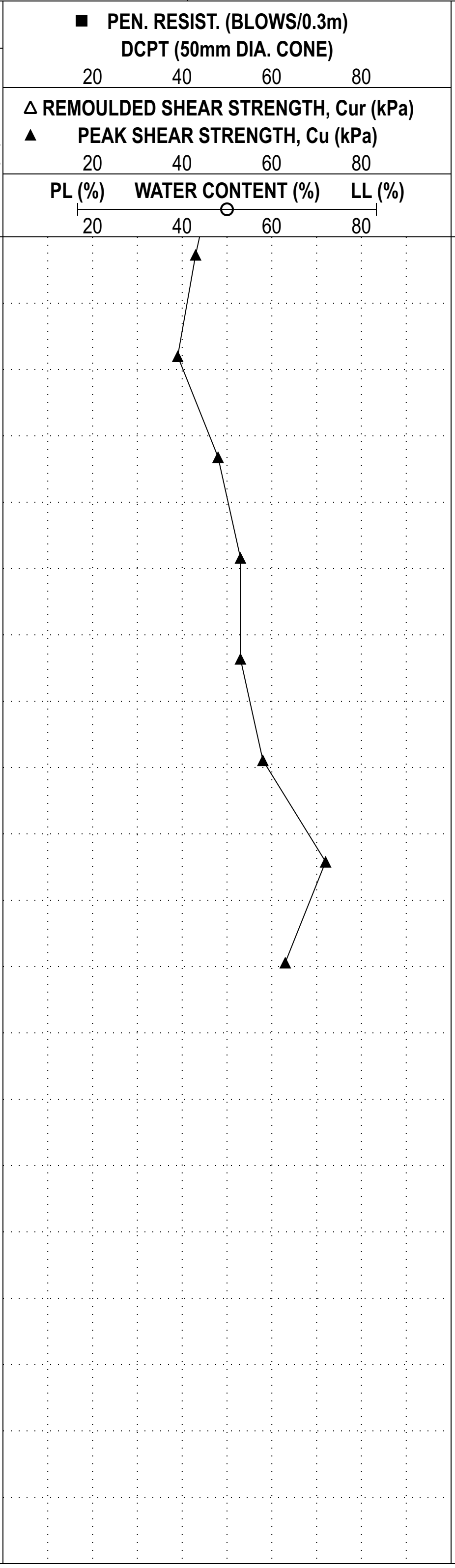
REMARKS:

DATE: September 05, 2024

FILE NO. : PG6934

HOLE NO. : BH 3-24

SAMPLE DESCRIPTION GROUND SURFACE	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N, Nc OR RQD	SAMPLE WATER CONTENT (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
	▨	10						85
	▨	11	SS 12	42	P			84
	▨	12						83
	▨		SS 13	83	P			
	▨	13						82
	▨	14	SS 14	71	P			81
	▨	15						80
15.85m [79.24m]	▨		SS 15	75	P			
End of Borehole		16						79
		17						78
		18						77
		19						76
		20						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350205.42 **NORTHING:** 5018431.76 **ELEVATION:** 95.12

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 09, 2024

FILE NO. : PG6934

HOLE NO. : BH 4-24

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Compact, brown silty sand, with crushed stone, trace clay, some topsoil and organics

0.46m [94.66m]

Stiff, brown **SILTY SAND**, some clay

1.45m [93.67m]

Loose, brown **SILTY SAND**

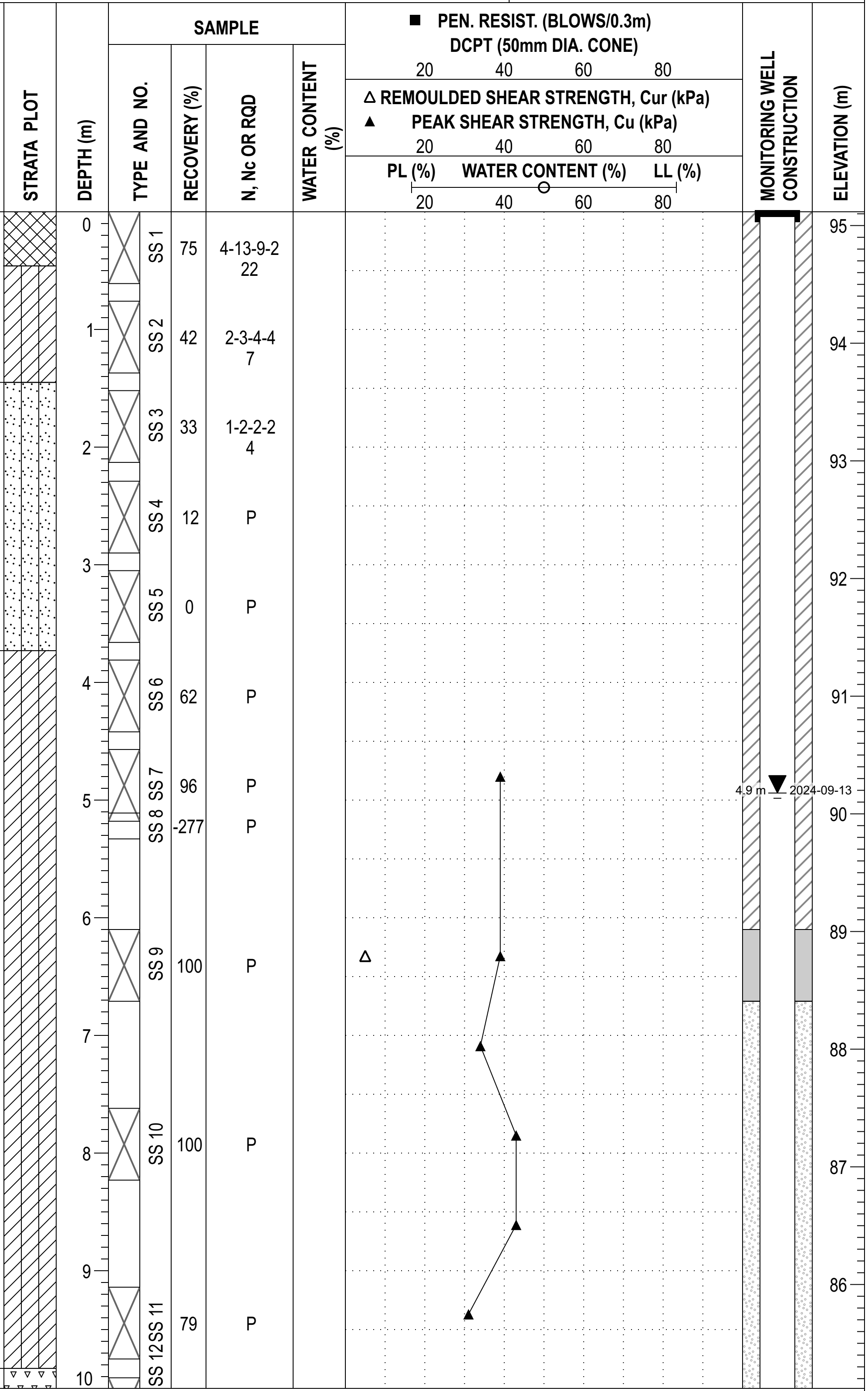
- Running sand encountered from 1.80 m to 3.73 m depth

3.73m [91.39m]

Firm, grey **SILTY CLAY**

9.83m [85.29m]

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	75	4-13-9-2 22	
SS 2	42	2-3-4-4 7	
SS 3	33	1-2-2-2 4	
SS 4	12	P	
SS 5	0	P	
SS 6	62	P	
SS 7	96	P	
SS 8	-277	P	
SS 9	100	P	
SS 10	100	P	
SS 11	79	P	
SS 12			



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350205.42 **NORTHING:** 5018431.76 **ELEVATION:** 95.12

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 09, 2024

FILE NO. : PG6934

HOLE NO. : BH 4-24

SAMPLE DESCRIPTION

GROUND SURFACE

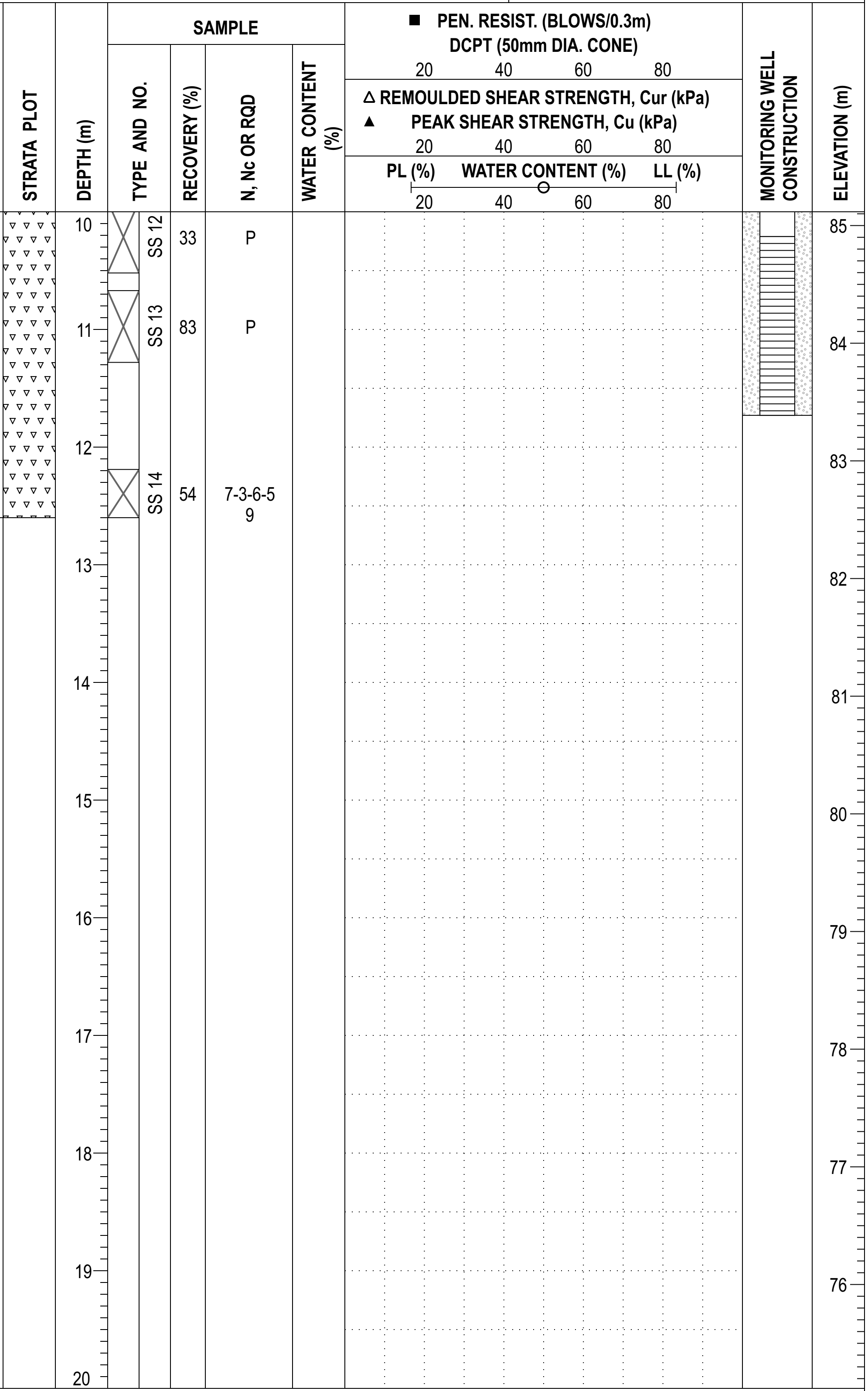
GLACIAL TILL: firm to stiff, grey silty clay, with trace sand, occasional cobbles and boulders

12.60m [82.52m]

End of Borehole

Practical refusal to augering at 12.6 m depth

(GWL at 4.94 m depth on September 13, 2024)



TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 12	33	P	
SS 13	83	P	
SS 14	54	7-3-6-5 9	

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

8201 Campeau Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 350375.24 **NORTHING:** 5018494.15 **ELEVATION:** 96.63

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

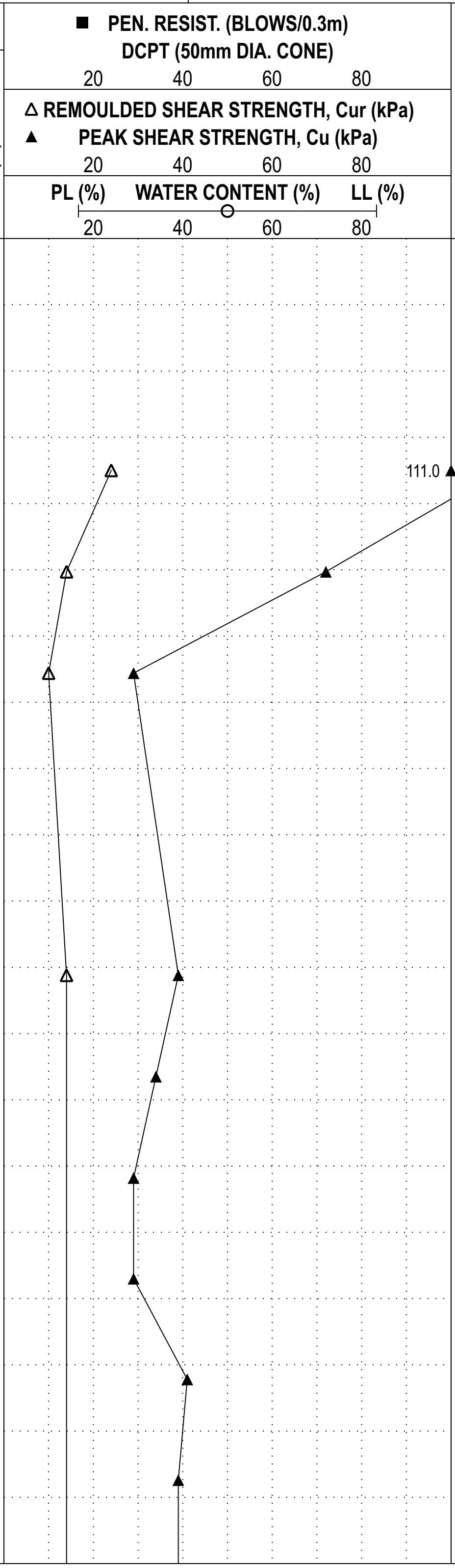
DATE: September 10, 2024

FILE NO. : PG6934

HOLE NO. : BH 5-24

SAMPLE DESCRIPTION (GROUND SURFACE)	Bottom depth [elevation]
TOPSOIL and organics	0.25m [96.38m]
FILL: Loose, brown silty fine sand, with clay	1.07m [95.56m]
Very stiff to stiff, brown **SILTY CLAY**, trace sand	2.97m [93.66m]
Loose, grey **SILTY** fine **SAND**, with trace clay	4.50m [92.13m]
Firm, grey, **SILTY CLAY**, with trace sand	

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	67	1-2-3-3 5	
SS 2	96	2-3-3-4 6	
SS 3		P	
SS 4	71	P	
SS 5	33	P	
SS 6	87	P	
SS 7	100	P	
SS 8	92	P	
SS 9	71	P	
SS 10	96	P	
SS 11	92	P	



PIEZOMETER CONSTRUCTION

DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10

ELEVATION (m): 96 95 94 93 92 91 90 89 88 87

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SOIL PROFILE AND TEST DATA

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8201 Campeau Drive, Ottawa, Ontario

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PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

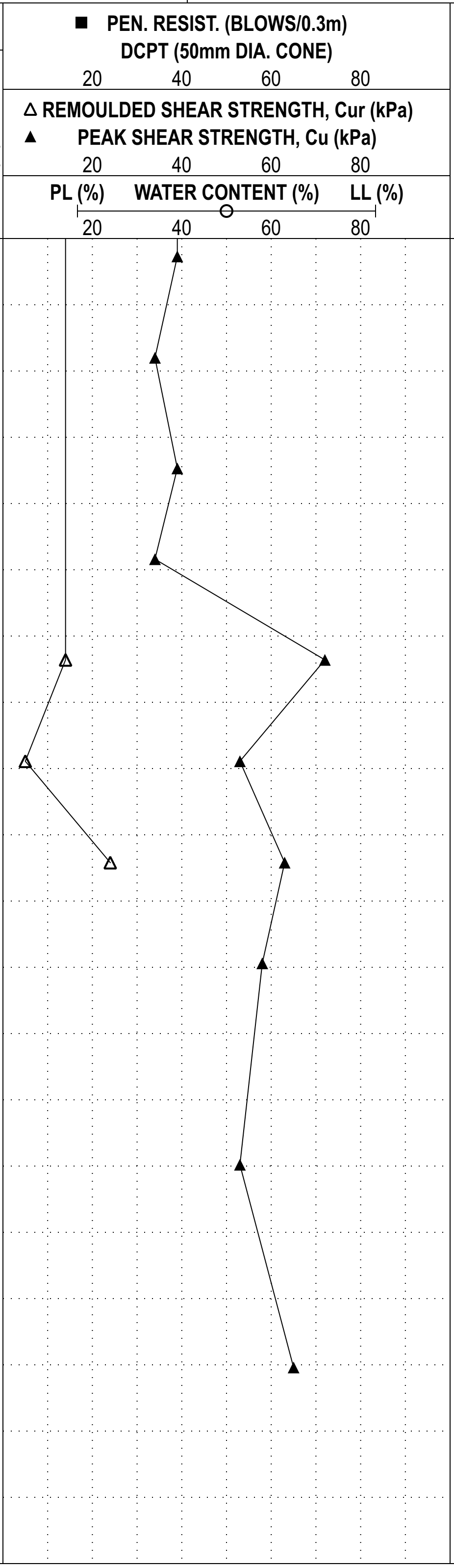
REMARKS:

DATE: September 10, 2024

FILE NO. : PG6934

HOLE NO. : BH 5-24

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE					
	10				
	11	SS 12	25	P	
	12	SS 13	100	P	
	13				
	14	SS 14	100	P	
- 50 mm sand seams encountered at 15.00 m depth	15				
	15.5	SS 15	100	P	
	16				
	17	SS 16	100	P	
	18				
	18.5	SS 17	100	P	
19.35m [77.28m]	19				
Very dense, grey **SILTY SAND**, with gravel, some clay, occasional cobbles and boulders	20	SS 18			



PIEZOMETER CONSTRUCTION

ELEVATION (m): 86, 85, 84, 83, 82, 81, 80, 79, 78, 77

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SOIL PROFILE AND TEST DATA

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PROJECT: Proposed Development

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REMARKS:

DATE: September 10, 2024

FILE NO. : PG6934

HOLE NO. : BH 5-24

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE						
	20	SS 18	65	36-21-50-/ 71		
20.93m [75.70m]						76
BEDROCK: excellent quality limestone bedrock	21					75
	22	RC 1	100	RQD 95		74
	23	RC 2	100	RQD 100		73
24.05m [72.58m]	24					72
End of Borehole	25					71
	26					70
	27					69
	28					68
	29					67
	30					

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

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

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

PIEZOMETER CONSTRUCTION

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

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

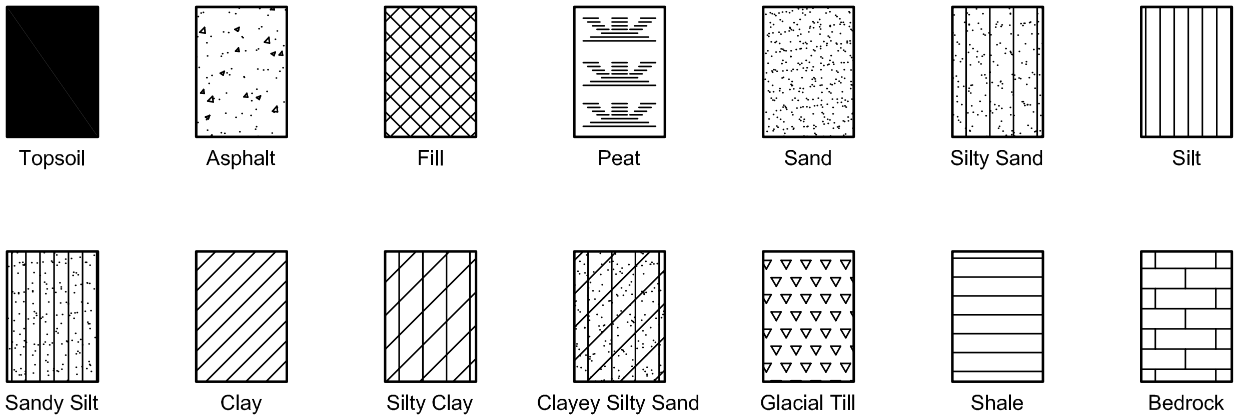
Wo - Initial water content (at start of consolidation test)

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.

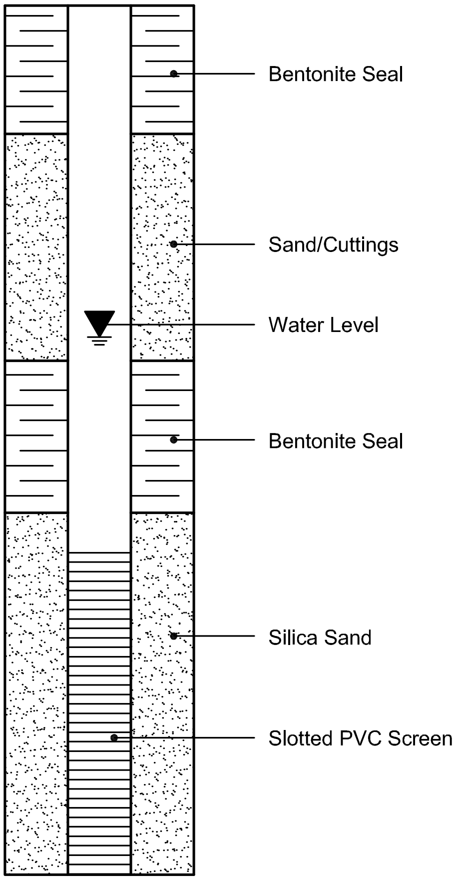
SYMBOLS AND TERMS (continued)

STRATA PLOT

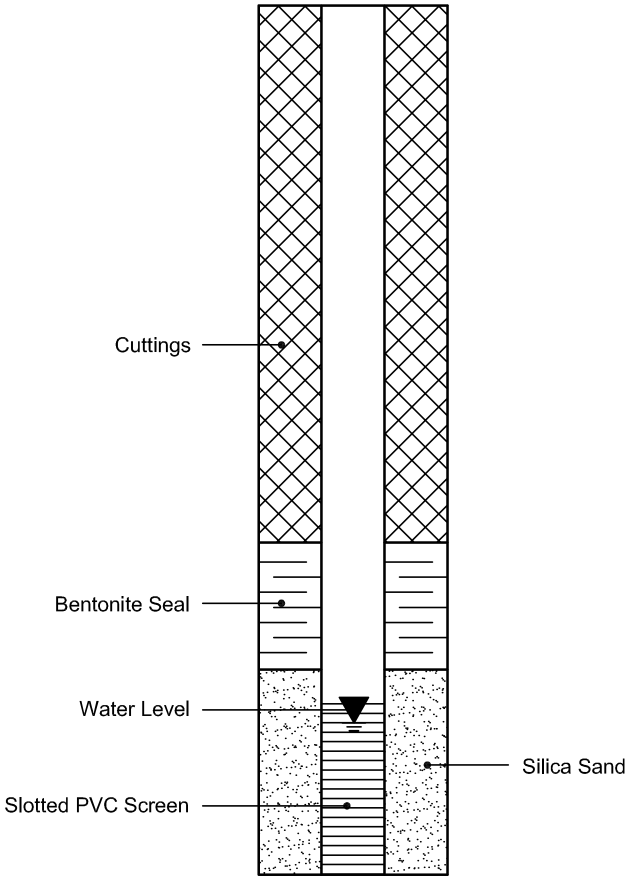


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Log of Borehole BH-1

Project No: OTT-23015656-A0

Project: Borehole and Probehole Investigation. Proposed Residential Developmet

Location: 303 Didsbury Crescent. Ottawa, Ontario

Figure No. 3

Page. 1 of 2

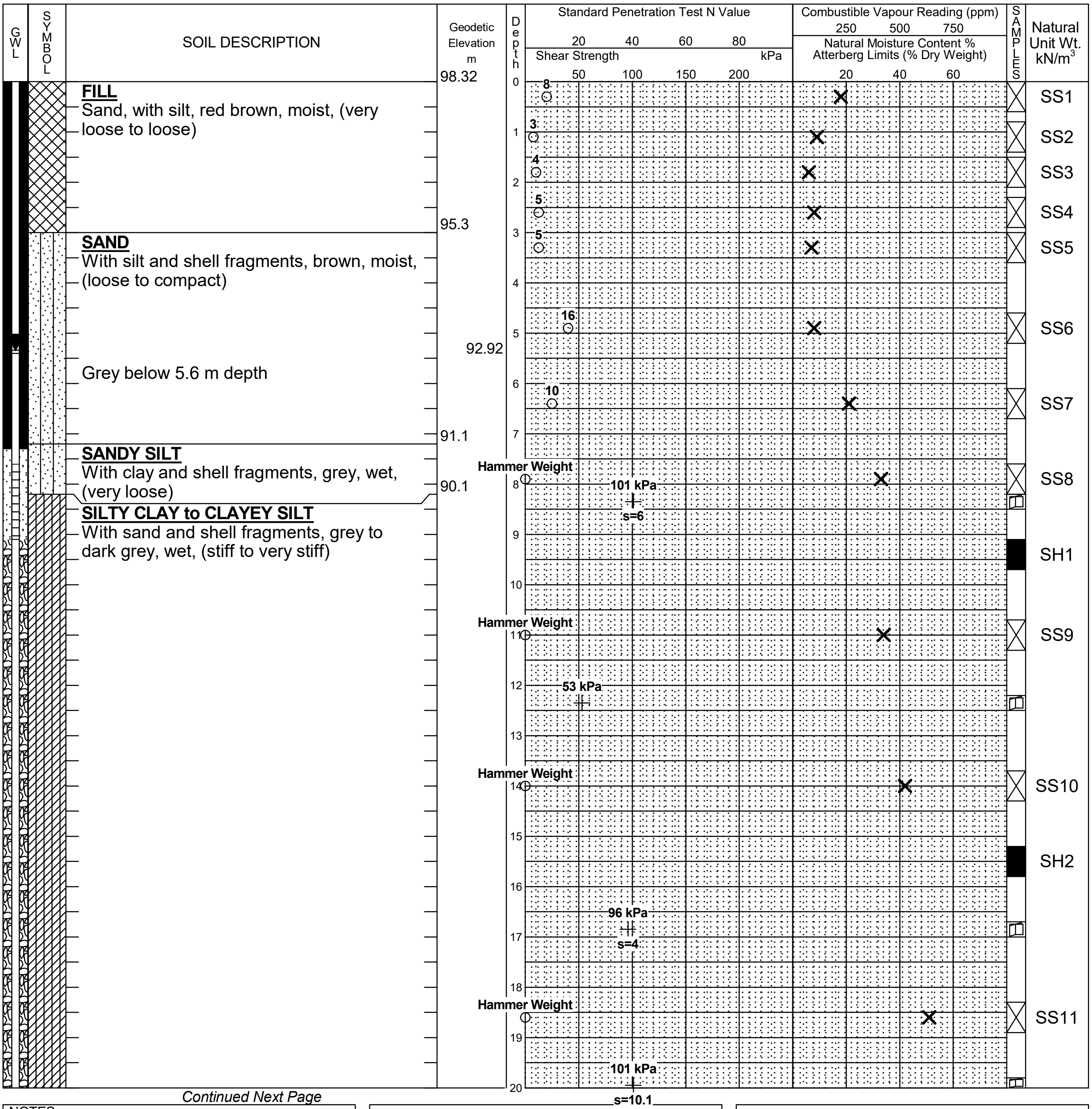
Date Drilled: 'January 29 2024

Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: J.E Checked by: I.T

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	┼ S	Shear Strength by Penetrometer Test	▲



SOIL DESCRIPTION

98.32

FILL
Sand, with silt, red brown, moist, (very loose to loose)

95.3

SAND
With silt and shell fragments, brown, moist, (loose to compact)

92.92

Grey below 5.6 m depth

91.1

SANDY SILT
With clay and shell fragments, grey, wet, (very loose)

90.1

SILTY CLAY to CLAYEY SILT
With sand and shell fragments, grey to dark grey, wet, (stiff to very stiff)

Continued Next Page

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter piezometer well was installed, as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-23015656-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
37 days	5.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-1

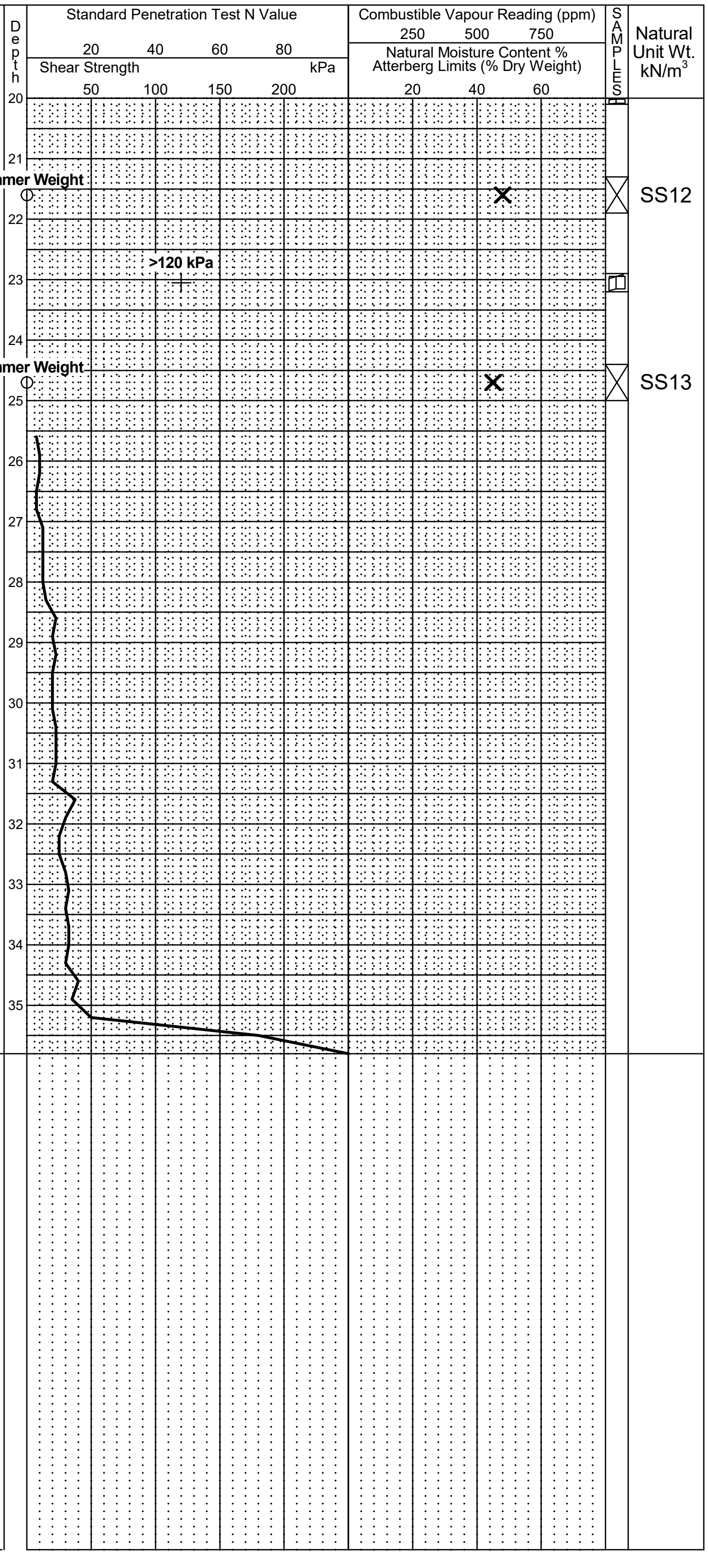
Project No: OTT-23015656-A0

Project: Borehole and Probehole Investigation. Proposed Residential Developmet

Figure No. 3

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m
		SILTY CLAY to CLAYEY SILT With sand and shell fragments, grey to dark grey, wet, (stiff to very stiff) *(continued)*	78.32
		Dynamic Cone Penetration Test (DCPT) conducted from 25.6 m depth. Termination depth of 35.8 m.	72.7
		Borehole Terminated upon Cone Refusal at 35.8 m Depth	62.5



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter piezometer well was installed, as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-23015656-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
37 days	5.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-2

Project No: OTT-23015656-A0

Project: Borehole and Probehole Investigation. Proposed Residential Developmet

Location: 303 Didsbury Crescent. Ottawa, Ontario

Figure No. 4

Page. 1 of 1

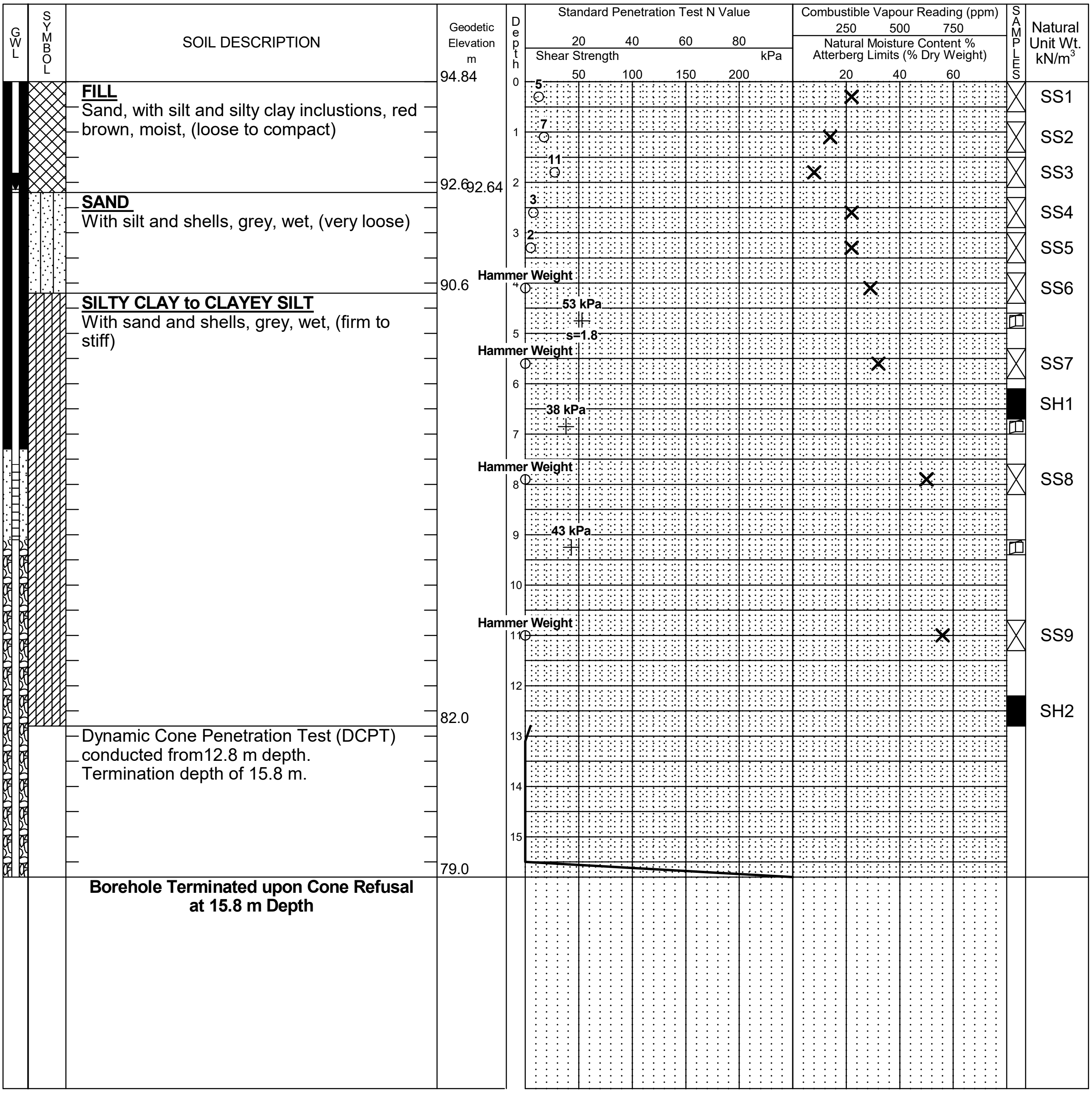
Date Drilled: 'January 30 2024

Drill Type: CME-55 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: J.E Checked by: I.T

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



94.84

FILL
Sand, with silt and silty clay inclusions, red brown, moist, (loose to compact)

92.6 92.64

SAND
With silt and shells, grey, wet, (very loose)

90.6

SILTY CLAY to CLAYEY SILT
With sand and shells, grey, wet, (firm to stiff)

82.0

Dynamic Cone Penetration Test (DCPT) conducted from12.8 m depth.
Termination depth of 15.8 m.

79.0

Borehole Terminated upon Cone Refusal at 15.8 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter piezometer well was installed, as shown.
3. Field work was supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-23015656-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
36 days	2.2	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Table I ; Inferred Bedrock Elevation at Probehole Locations. 303 Disburry Crescent, ottawa, ON

Probehole No	GS Elev.(m)	Inferred Bedrock Depth (m)	Inferred Bedrock Elevation (m)	Comment
P1	97.01	30.78	66.22	
P2	98.39	28.96	69.43	
P3	98.95	31.70	67.25	
P4	97.54	28.96	68.59	
P5	97.09	28.35	68.74	
P6	96.60	37.80	<58.81	No Bedrock to 37.8 m
P7	98.75	36.27	62.48	
P8	98.96	35.97	62.99	
P9	96.15	28.65	67.50	
P10	96.77	27.43	69.34	
P11	95.93	30.18	65.75	
P12	97.46	31.09	66.37	
P13	98.42	27.43	70.99	
P14	95.65	21.64	74.00	
P15	96.46	19.20	77.26	
P16	95.28	22.56	72.72	
P17	94.95	24.08	70.87	
P18	95.00	19.51	75.49	
P19	95.33	17.68	77.65	
P20	95.47	16.15	79.32	
P21	94.50	20.12	74.38	
P22	94.77	17.98	76.78	
P23	94.78	13.41	81.37	
P24	95.26	12.50	82.76	
P25	95.30	11.58	83.72	
P26	95.18	13.41	81.76	
P27	94.87	18.90	75.97	
P28	94.84	17.07	77.77	
P29	94.89	11.58	83.31	
P30	95.12	13.11	82.01	
P31	94.79	10.36	84.42	
P32	95.13	17.37	77.75	
P33	94.94	17.37	77.57	
P34	95.14	21.03	74.11	
P35	95.26	22.56	72.70	
P36	95.29	23.16	72.13	
P37	96.69	27.74	68.96	
P38	97.82	25.60	72.22	
P39	95.57	19.81	75.76	
P40	95.56	18.59	76.96	

Table I ; Inferred Bedrock Elevation at Probehole Locations. 303 Disburry Crescent, ottawa, ON

Probehole No	GS Elev.(m)	Inferred Bedrock Depth (m)	Inferred Bedrock Elevation (m)	Comment
P41	97.53	28.04	69.48	
P42	97.26	37.80	< 59.46	No Bedrock to 37.8 m
P43	98.32	33.83	64.49	
P44	98.89	37.80	61.09	
BH-1	98.31	35.80	62.51	DCPT Refusal
BH-2	94.84	15.80	79.04	DCPT Refusal

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consulting engineers

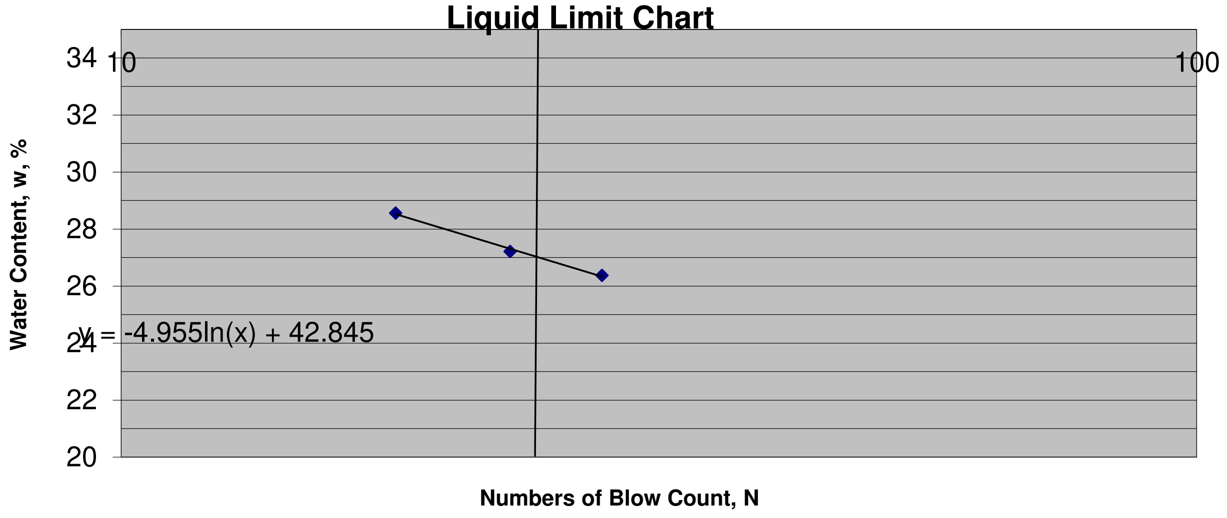
ATTERBERG LIMITS
LS-703/704

CLIENT:	Theberge Developments	FILE NO.:	PG6934
PROJECT:	8201 Campeau Drive	DATE SAMPLED:	05-Sep-24
LOCATION:	BH3-24 -SS7(15' - 17')	DATE REPORTED:	20-Sep-24

CAN NO.	**x98**	**X18**	**n6**				
WT. OF CAN	6.91	6.95	7.23				
WT. OF SOIL & CAN	19.2	19.15	19.16				
WT. OF DRY SOIL & CAN	16.47	16.54	16.67				
WT. OF MOISTURE	2.73	2.61	2.49				
WT. OF DRY SOIL & CAN	9.56	9.59	9.44				
WATER CONTENT, w, %	**28.56**	**27.22**	**26.38**				
NO. OF BLOWS, N	18	23	28				

CAN NO.	**x24**	**X7**	
	4.54	4.52	
WT. OF SOIL & CAN	11.24	10.92	
WT. OF DRY SOIL & CAN	10.40	10.12	
WT. OF MOISTURE	0.84	0.8	
WT. OF DRY SOIL & CAN	5.86	5.6	
WATER CONTENT, w, %	**14.33**	**14.29**	

RESULTS	
LIQUID LIMIT	**27**
PLASTIC LIMIT	**14**
PLASTICITY INDEX	**13**
NATURAL MOISTURE CONTENT	**40.35%**



TECHNICIAN:	**REVIEWED BY:**		

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consulting engineers

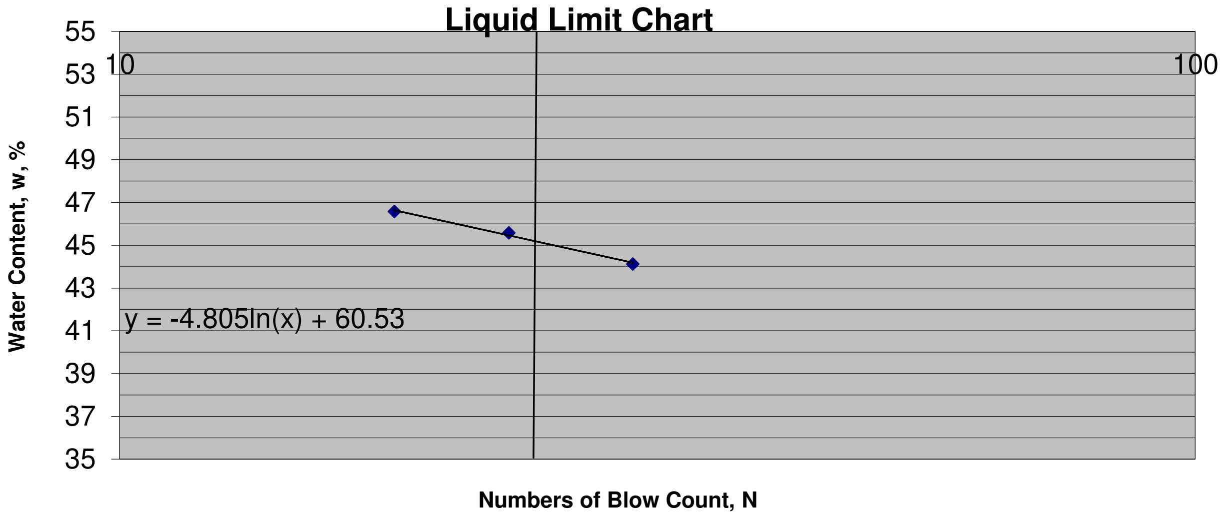
ATTERBERG LIMITS
LS-703/704

CLIENT:	Theberge Developments	FILE NO.:	PG6934
PROJECT:	8201 Campeau Drive	DATE SAMPLED:	09-Sep-24
LOCATION:	BH5-24 -SS3(5' - 7')	DATE REPORTED:	20-Sep-24

CAN NO.	**S31**	**X41**	**C**				
WT. OF CAN	6.8	6.84	6.96				
WT. OF SOIL & CAN	17.72	18.85	18.49				
WT. OF DRY SOIL & CAN	14.25	15.09	14.96				
WT. OF MOISTURE	3.47	3.76	3.53				
WT. OF DRY SOIL & CAN	7.45	8.25	8				
WATER CONTENT, w, %	**46.58**	**45.58**	**44.13**				
NO. OF BLOWS, N	18	23	30				

CAN NO.	**106**	**X13**	
	4.92	4.98	
WT. OF SOIL & CAN	11.35	12.48	
WT. OF DRY SOIL & CAN	10.20	11.18	
WT. OF MOISTURE	1.15	1.3	
WT. OF DRY SOIL & CAN	5.28	6.2	
WATER CONTENT, w, %	**21.78**	**20.97**	

RESULTS	
LIQUID LIMIT	**45**
PLASTIC LIMIT	**21**
PLASTICITY INDEX	**24**
NATURAL MOISTURE CONTENT	**40.63%**

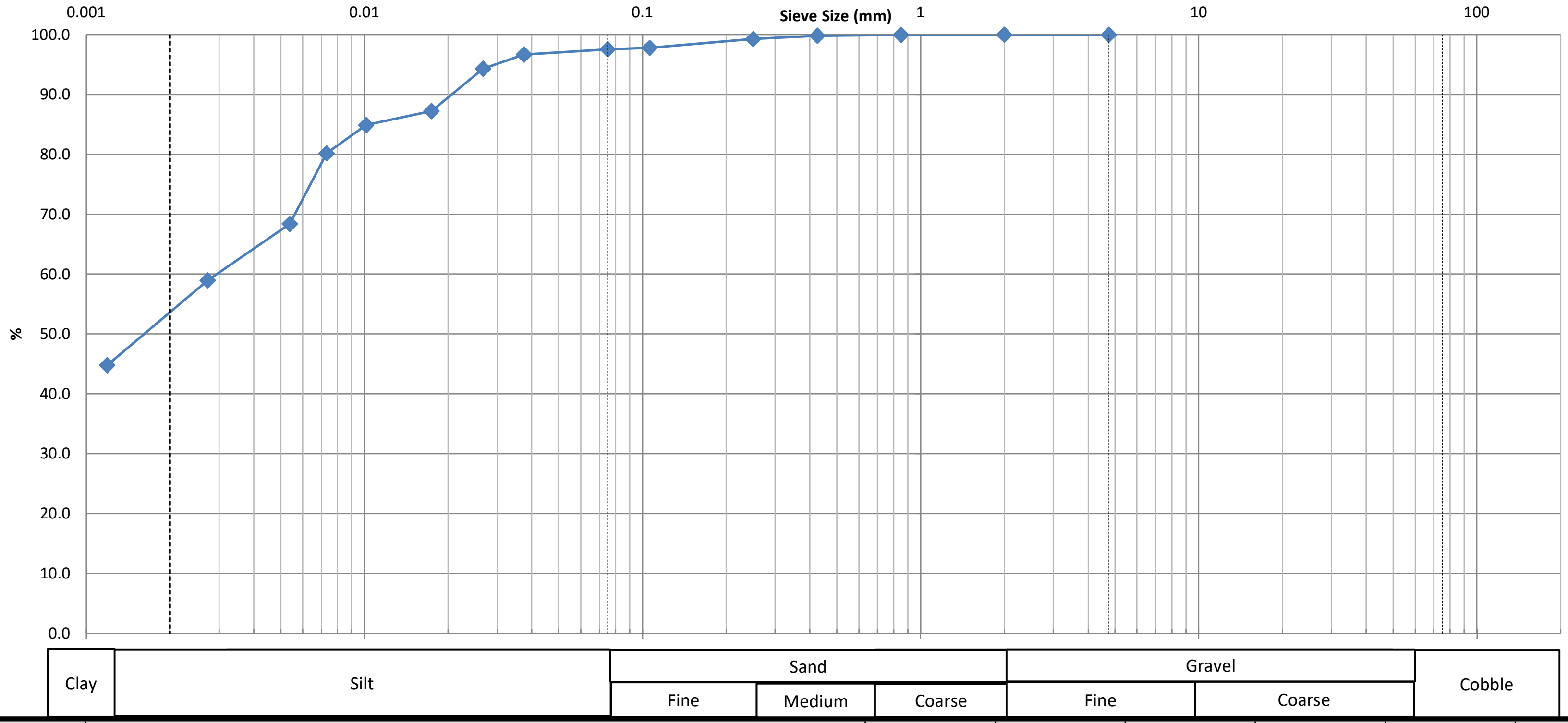


TECHNICIAN:	**REVIEWED BY:**	

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Theberge Developments Ltd.	DEPTH:	15' - 17'	FILE NO:	PG6934
CONTRACT NO.:		BH OR TP No.:	BH4-24 SS7	LAB NO:	56337
PROJECT:	8201 Campeau Drive, Ottawa, ON			DATE RECEIVED:	12-Sep-24
				DATE TESTED:	13-Sep-24
DATE SAMPLED:	4-Sep-24			DATE REPORTED:	26-Sep-24
SAMPLED BY:	K.S.			TESTED BY:	D.K



Identification	**Soil Classification**					**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
						65.7%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	2.4		44.6		53.0	

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Theberge Developments Ltd.	DEPTH:	15' - 17'	FILE NO.:	PG6934
PROJECT:	8201 Campeau Drive, Ottawa, ON	BH OR TP No.:	BH4-24 SS7	DATE SAMPLED:	4-Sep-24
LAB No. :	56337	TESTED BY:	D.K	DATE RECEIVED:	12-Sep-24
SAMPLED BY:	K.S.	DATE REPT'D:	26-Sep-24	DATE TESTED:	13-Sep-24

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
90.5	2.700

INITIAL WEIGHT	50.00	HYGROSCOPIC MOISTURE		
WEIGHT CORRECTED	33.92	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	1.22	AIR DRY	133.40	133.40
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	90.50	90.50
		CORRECTED	0.678	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**0.0**	0.0	**100.0**
2.0	**0.0**	0.0	**100.0**
Pan	**90.5**		
0.850	**0.02**	0.0	**100.0**
0.425	**0.08**	0.2	**99.8**
0.250	**0.35**	0.7	**99.3**
0.106	**1.10**	2.2	**97.8**
0.075	**1.21**	2.4	**97.6**
Pan	**1.22**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:28	47.0	6.0	23.0	0.0374	96.7	**96.7**
2	10:29	46.0	6.0	23.0	0.0267	94.3	**94.3**
5	10:32	43.0	6.0	23.0	0.0174	87.3	**87.3**
15	10:42	42.0	6.0	23.0	0.0102	84.9	**84.9**
30	10:57	40.0	6.0	23.0	0.0073	80.2	**80.2**
60	11:27	35.0	6.0	23.0	0.0054	68.4	**68.4**
250	14:37	31.0	6.0	23.0	0.0027	59.0	**59.0**
1440	10:27	25.0	6.0	23.0	0.0012	44.8	**44.8**

Moisture = 65.7%

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

PATERSON GROUP

Linear Shrinkage
ASTM D4943-02

CLIENT:	Theberge Developments Ltd.	DEPTH	20' - 22'	FILE NO.:	PG6934
PROJECT:	8201 Campeau Drive	BH OR TP No:	BH1-24 SS9	DATE SAMPLED	4-Sep-24
LAB No:	56338	TESTED BY:	C.P	DATE RECEIVED	12-Sep-24
SAMPLED BY:	K.S.	DATE REPORTED:	26-Sep-24	DATE TESTED	13-Sep-24

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(7)	Calibration (Two Trials)	Tin NO.(x21)	
Tare	5.02	Tin	4.84	4.84
Soil Pat Wet + Tare	74.98	Tin + Grease	5.03	5.03
Soil Pat Wet	69.96	Glass	43.23	43.23
Soil Pat Dry + Tare	56.19	Tin + Glass + Water	85.34	85.34
Soil Pat Dry	51.17	Volume	37.08	37.08
Moisture	**36.72**	**Average Volume**	**37.08**	

Soil Pat + String	51.33
Soil Pat + Wax + String in Air	57.5
Soil Pat + Wax + String in Water	23.55
Volume Of Pat (Vdx)	33.95

RESULTS:

Shrinkage Limit	**17.06**
Shrinkage Ratio	**1.894**
Volumetric Shrinkage	**37.245**
Linear Shrinkage	**10.014**

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

Certificate of Analysis

Report Date: 19-Sep-2024

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

Order Date: 13-Sep-2024

Client PO: 61265

Project Description: PG6934

Client ID:		BH3_24 SS3	-	-	-	-	-
Sample Date:		05-Sep-24 09:00	-	-	-		
Sample ID:		2438027-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	70.2	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.07	-	-	-	-	-
Resistivity	0.1 Ohm.m	123	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	13	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6934-1 - TEST HOLE LOCATION PLAN

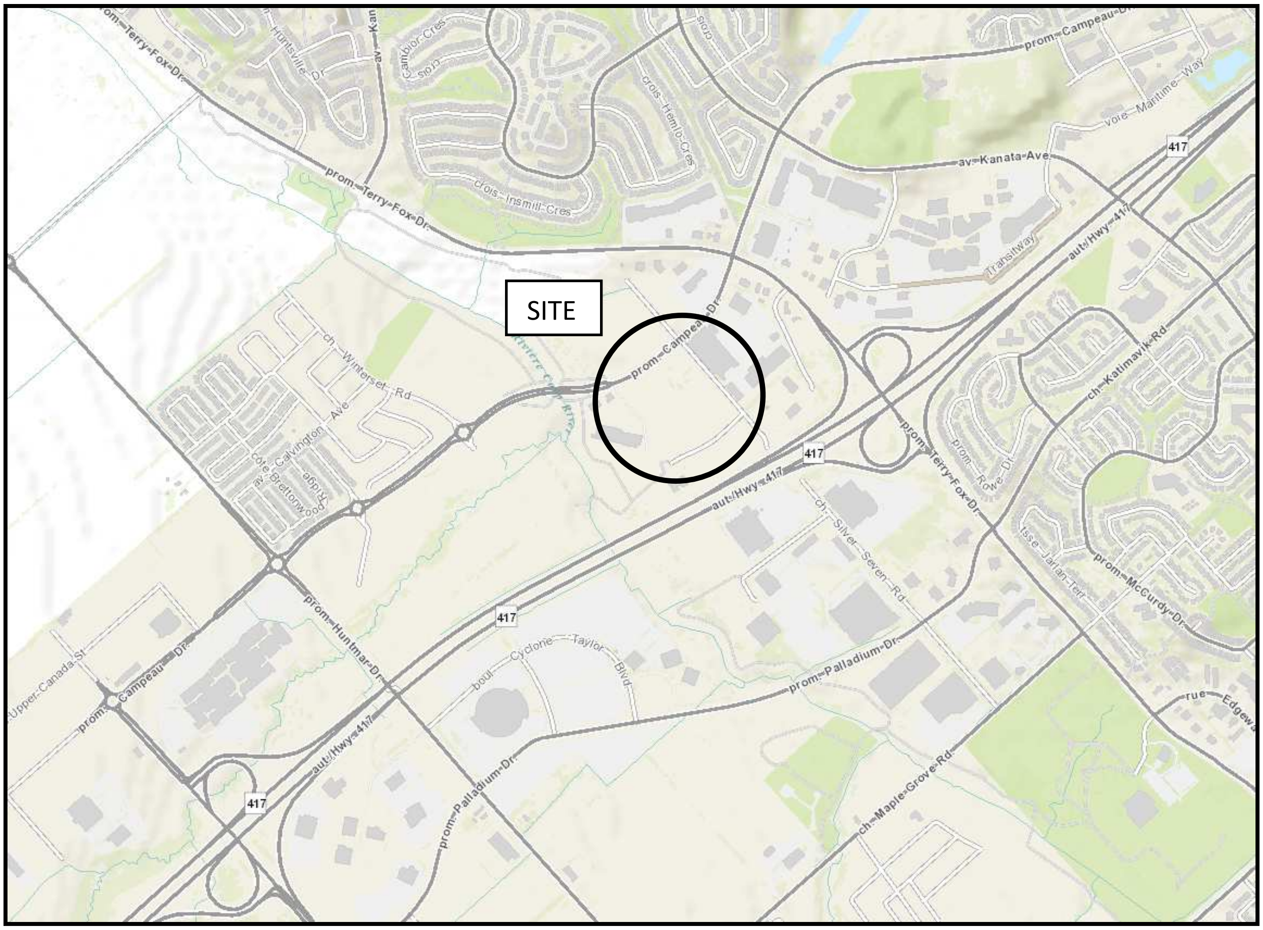


FIGURE 1

KEY PLAN

PATERSON GROUP

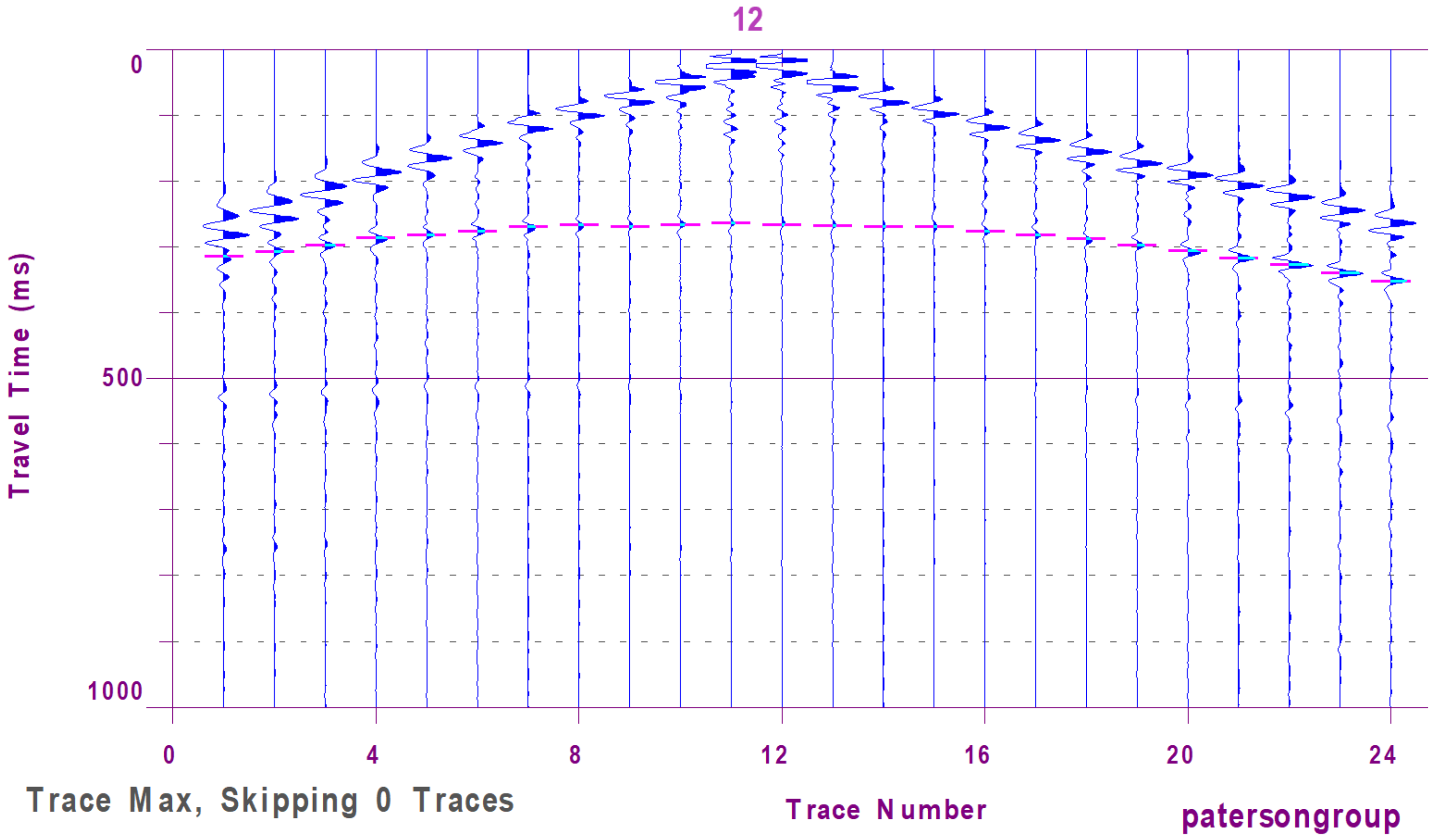
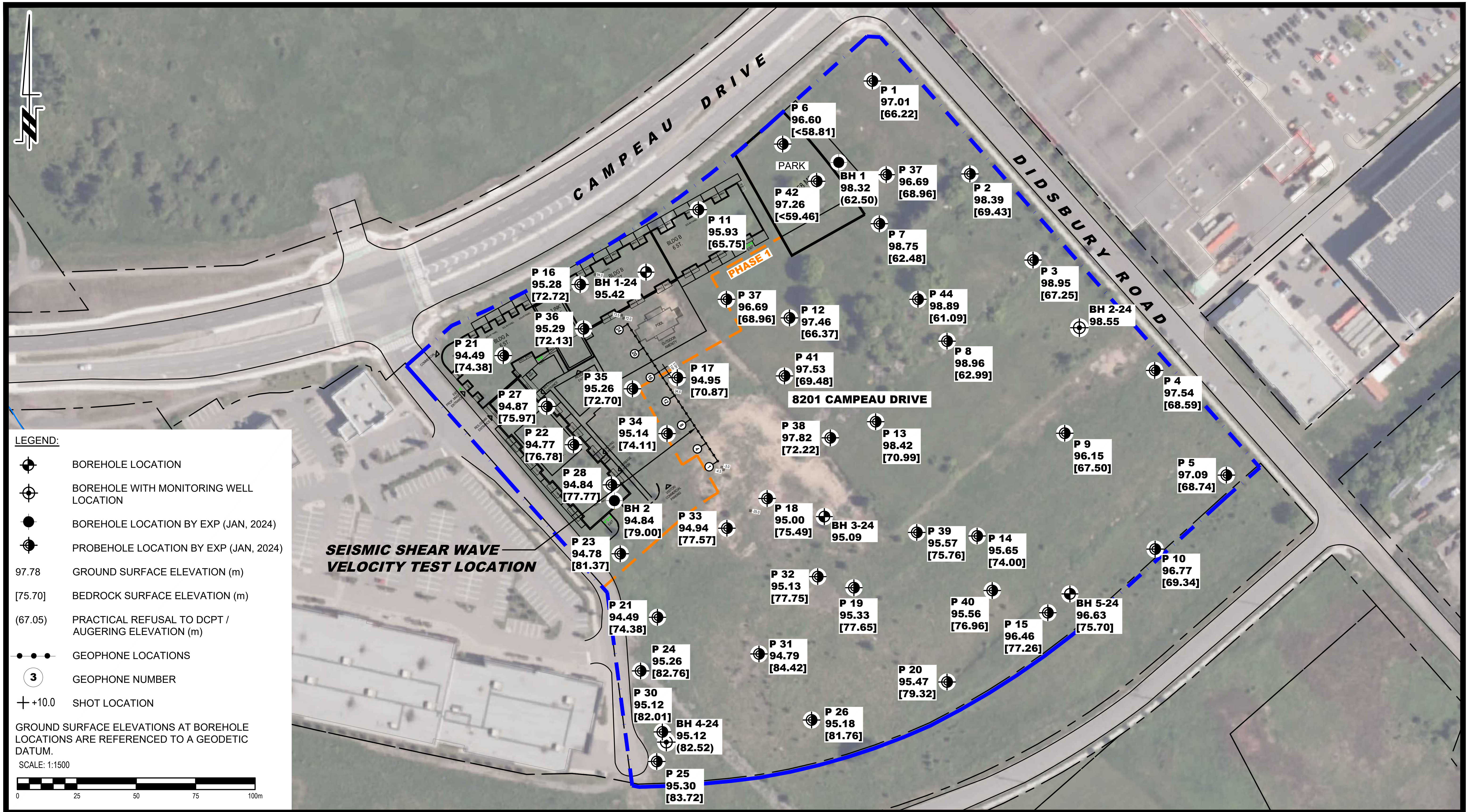


Figure 2 – Shear Wave Velocity Profile at Shot Location 34.5 m

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



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
●	BOREHOLE LOCATION BY EXP (JAN, 2024)
⊕	PROBEHOLE LOCATION BY EXP (JAN, 2024)
97.78	GROUND SURFACE ELEVATION (m)
[75.70]	BEDROCK SURFACE ELEVATION (m)
(67.05)	PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)
• • •	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
+ +10.0	SHOT LOCATION

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

SCALE: 1:1500

0 25 50 75 100m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
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NO.	REVISIONS	DD/MM/YYYY	INITIAL
1	UPDATED CONCEPTUAL PLAN	27/03/2026	SD

KANATA WOODS INC.
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
8201 CAMPEAU DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1500
Date: 10/2024
Drawn by: GK
Report No.: PG6934-1
Checked by: KS
Dwg. No.: PG6934-1
Approved by: SD
Revision No.: 1

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