Geotechnical Investigation

Proposed Commercial Building

333 Huntmar Drive
Ottawa, Ontario

Prepared for RioCan Holdings (TJV) Inc.
c/o Chick-Fil-A Canada ULC

Report PG6571-1 Revision 2 dated May 16, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by RioCan Holdings (TJV) Inc. on behalf of Chick-Fil-A Canada ULC to complete a geotechnical investigation for the proposed commercial building to be located within the existing commercial development at 333 Huntmar Drive in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the investigation were to:

- ❑ Determine the subsurface soil and groundwater conditions by means of boreholes.

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

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

Investigating 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 drawings, it is understood that a one-storey building of slab-on-grade construction is proposed. Associated asphalt paved-parking areas and access lanes with landscaped margins are also anticipated surrounding the proposed building. It is expected that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was conducted on February 6, 2023 and consisted of a total of five (5) boreholes advanced to a maximum depth of 6.7 m below the existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG6571-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, 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, using field vanes. Reference should be made to the Soil Profile and Test Data Sheets provided in Appendix 1.

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

Groundwater

Standpipe piezometers were also installed in the remaining borehole locations permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Groundwater levels are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG6571-1 – 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 two (2) samples were submitted for Atterberg limit tests, one (1) grain-size distribution analysis, and one (1) Shrinkage test. The test results are presented in Subsection 4.2 and on Atterberg Limit Results, presented in Appendix 1.

Additionally, moisture content sampling was completed for the subject site from between all boreholes. The results of the moisture contents are presented in the Soil Profile and Test Data Sheets in Appendix 1.

All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site consists of a relatively flat open grassed area. The site gradually slopes in the west to east direction between approximate geodetic elevations of 104 to 101 m. A shallow ditch is located along the east property boundary with an approximate depth of 0.5 to 1 m. Furthermore, it is understood that a temporary granular roadway was constructed within the north property boundary of the subject site during the construction of Tanger Outlets Ottawa. It is anticipated that the temporary roadway was covered with existing landscaping.

The site is bordered by Palladium Drive to the west, an access road to the Tangers development to the north, and Tanger Outlets Ottawa to the east and south.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of a vegetative layer underlain by a 0.7 to 1.5 m layer of fill. The fill layer was generally observed to consist of a loose to compact, brown silty sand to sandy silt with gravel and crushed stone.

The fill layer was observed to be underlain by a deposit of hard to very stiff brown extending to approximate depths varying between 4.1 to 4.3 m. The silty clay deposit was further underlain by a deposit of loose grey silt with a variable amount of sand.

Practical refusal to DCPT was encountered at BH 2-23 at a depth of approximately 6.7 m below ground surface.

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

Atterberg Limits

Atterberg limits testing was completed on select silty clay samples where encountered.

The results of the Atterberg limits test are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

The tested silty clay samples classify as inorganic clay of low plasticity (CL) and inorganic silt of high plasticity (MH) in accordance with the Unified Soil Classification System.

Table 1 – Summary of Atterberg Limits Tests					
Sample	**Moisture Content %**	**Liquid Limit %**	**Plastic Limit %**	**Plasticity Index %**	**Classification**
BH 1-23 – SS3	35	55	30	25	MH
BH 2-23 – SS4	38	55	26	29	CL
Note: CL – Inorganic Clay of Low Plasticity MH – Inorganic Silt of High Plasticity					

Grain Size Distribution and Hydrometer Testing

Grain size distribution analysis was completed on one (1) select recovered silty clay sample. The results of the grain size distribution analysis are presented in Table 2 below.

Table 2 – Grain Size Distribution Results					
Sample	**Depth, m**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
BH 1-23 – SS7	4.6	0.3	23.9	70.8	5.0

Shrinkage Test

Linear shrinkage testing was completed on a sample recovered from a depth of 1.5 to 2.1 m below ground surface from borehole BH 3-23 and yielded a shrinkage limit of 18.9 and a shrinkage ratio of 1.81.

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of limestone with interbedded shale of the Verulam Formation with an approximate overburden drift thickness of 5 to 10 m.

4.3 Groundwater

Groundwater levels were measured in the standpipe piezometers installed at all boreholes locations on February 14, 2023. The observed groundwater levels are summarized in Table 3 below.

Table 3 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1-23	103.97	3.37	100.60	February 14, 2023
BH 2-23	103.02	2.53	100.49	
BH 3-23	102.91	2.14	100.77	
BH 4-23	103.69	2.74	100.95	
BH 5-23	103.75	2.78	100.97	

Note: Borehole elevations are referenced to a geodetic datum.

The recorded groundwater levels are also indicated on the applicable Soil Profile and Test Data sheet presented in Appendix 1. It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Additionally, groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

Long-term groundwater levels can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is anticipated at approximate depths of **3.5** to **4.5 m** below the existing ground surface.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed commercial building be founded on conventional spread footings bearing upon the in-situ, undisturbed, hard to very stiff brown silty clay.

Due to the presence of a silty clay layer, the subject site is subjected to a permissible grade restriction. Our permissible grade raise recommendations are discussed in Subsection 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 materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by Paterson at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

Where the fill is deemed acceptable, sub-excavation of the existing fill down to the native subgrade will only be required to be completed below the proposed footings, including the lateral support zone of each footing. Any fill left in place will be required to be proof-rolled using suitable compaction equipment in dry conditions and above freezing temperatures. The compaction efforts should also be reviewed and approved by Paterson personnel at the time of construction.

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. This material 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 the material’s 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.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane as discussed under subsection 6.1.

5.3 Foundation Design

Bearing Resistance Values

Pad footings (up to 5 m wide) and strip footings (up to 2 m wide) placed upon an undisturbed, hard to very stiff silty clay 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 bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the soils above the groundwater table when a plane extending horizontally and vertically from the perimeter of the footing at a minimum of 1.5H:1V passes through in situ soil of the same or higher capacity as the bearing medium soil.

Settlement and Permissible Grade Raise

Based on the undrained shear strength testing results, a permissible grade raise of **2.0 m** is recommended for the subject site. A long-term groundwater table of 0.5 m was assumed as part of our assessment.

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

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the potential for liquefaction of the underlying soil layers and to determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG6571-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 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 2 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 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 15, 3 and 2 m away from the first geophone, 2, 3 and 10 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for 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 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 buildings. 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 **215 m/s**, while the bedrock shear wave velocity is **2,456 m/s**. Based on the interpretation, the bedrock surface is encountered at approximately 9 m below existing ground surface.

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

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

$$V_{s30=} \frac{30\ m}{\left(\frac{9\ m}{215\ m/s} + \frac{21\ m}{2{,}456\ m/s}\right)}$$

$$V_{s30=}\ 595\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building is **595 m/s**. Therefore, a **Site Class C** is applicable for design of the proposed building as per Table 4.1.8.4.A of the OBC 2012.

Liquefaction Potential

Based on the seismic testing data, the subject site is not susceptible to liquefaction. A liquefaction hazard is commonly expressed in terms of a factor of safety. The factor of safety is defined as the available liquefaction resistance - cyclic resistance ratio (CRR) divided by the cyclic stresses generated by the design earthquake - cyclic stress ratio (CSR). Both of these stress parameters are normalized with respect to the effective overburden stress. The factor of safety against liquefaction for the subject site was determined to vary between 2.5 to 3.2 for the 9 m overburden profile using the normalized seismic shear wave velocity, Vs_1, and local seismic hazard parameters for a probability of 2% in 50 years corresponding to a Site Class C. Generally, a factor of safety of 1.2 to 1.5 is appropriate for building sites. Therefore, the soils underlying the subject site are not susceptible to liquefaction.

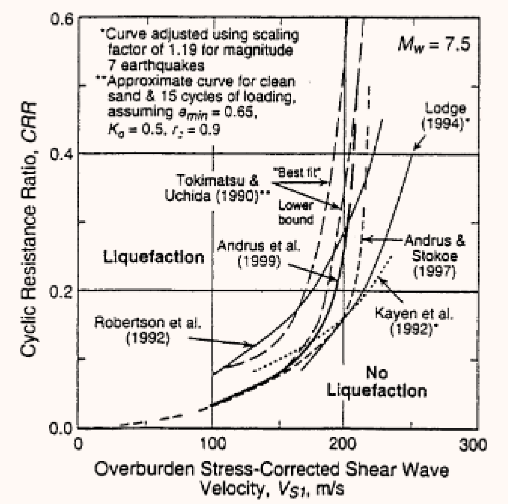


FIG. 8. Comparison of Seven Relationships between Liquefaction Resistance and Overburden Stress-Corrected Shear Wave Velocity for Granular Soils

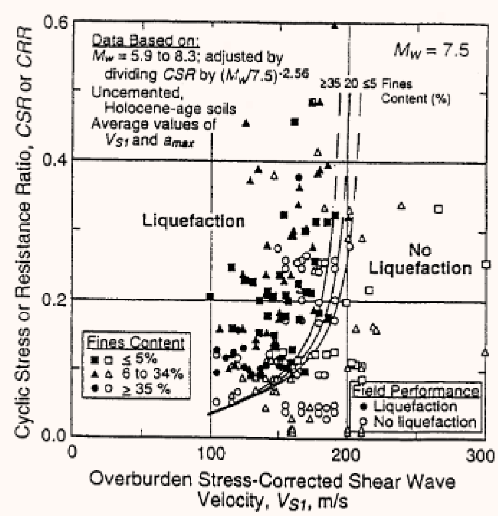


FIG. 9. Liquefaction Relationship Recommended for Clean, Uncemented Soils with Liquefaction Data from Compiled Case Histories (Reproduced from Andrus and Stokoe 2000)

Figure 1 – Reference Charts from Journal of Geotechnical and Geoenvironmental Engineering/October 2001

5.5 Slab on Grade Construction

With the removal of the topsoil layer and fill, containing deleterious or organic materials, the native soil will be considered to be an acceptable subgrade surface on which to commence backfilling for slab on grade construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of the SPMDD.

5.6 Pavement Structure

Car only parking areas and access lanes are proposed as part of the development at this site. The proposed pavement structures are shown in Tables 4 and 5 below.

Table 4 – Recommended Pavement Structure – Light Vehicle Parking

Thickness (mm)	Material Description
50	**Wear Course –** Superpave 12.5-FC2 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
400	**SUBBASE –** OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ silty clay or sand/crushed stone material placed over in situ soil	

Table 5 – Recommended Pavement Structure –Local Roadways, Access Lanes and Heavy Vehicle Parking

Thickness (mm)	Material Description
40	**Wear Course –** Superpave 12.5-FC2 Asphaltic Concrete
50	**Upper Binder Course –** Superpave 19.0 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ silty clay or sand/crushed stone material placed over in situ soil	

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 A or 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 vibratory equipment.

Minimum Performance Graded PG58H-34 asphalt cement should be used for this project. Cement asphalt should be compacted to a minimum average density of 93% and no more than 98%.

Rigid Pavement Structure

It is understood that a rigid pavement structure may be considered for the drive-through-lane. The rigid pavement structure presented in Table 6 is recommended for the subject drive-through lane for areas where a concrete pad is required. It should be noted that the reinforced concrete will be susceptible to frost heave if frost protection is not provided. Therefore, control and isolation joints are required for the subject concrete slabs.

Table 6 – Rigid Pavement Structure	
Thickness (mm)	**Material Description**
As Specified by Others	**Reinforced Concrete –** Minimum 32 MPa -with 5 to 8% air entrainment
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II Crushed Stone
See Paragraph Below	**RIGID INSULATION** – High-density extruded polystyrene rigid insulation boards such as DuPont Styrofoam HL-40 and/or Styro Rail Inc. SR.P400
SUBGRADE - Either fill, in situ silty clay or sand/crushed stone material placed over in situ soil	

To minimize the potential differential frost heave at the interface between the rigid pavement structure and adjacent asphalt pavement structures, a frost taper should be over-excavated below the asphalt pavement structure.

It is recommended that a minimum 600 mm thick frost taper, consisting of a Granular B Type II placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the SPMDD using suitable vibratory equipment, extend horizontally at least 1.5 m beyond the outside edge of the concrete pad.

The frost taper beyond the horizontal section should slope up to match the pavement structure subgrade level at a 3H:1V slope. It is recommended that insulation for frost mitigation is considered below the rigid pavement structure to prevent movement from frost action. For preliminary purposes, it is recommended that 100 mm thick rigid insulation panels extending 1.8 m beyond all faces of the slab are utilized. However, the insulation panel thickness and frost taper extent can be confirmed at the time of grading plan review.

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 granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD using suitable vibratory equipment.

Full depth isolation joints consisting of approximately 12 mm thick compressible material are recommended adjacent to any existing rigid structure such as curbs, poles, sidewalks, and buildings to allow minor movement to occur independently from each other.

Control joints, also known as contraction joints, provide a location where drying shrinkage cracks or cracking attributed to frost heave can occur without affecting the appearance of the concrete pad. The saw cut control joints should be placed at a minimum 2.4 m grid with a depth of 50 mm and a maximum width of 5 mm.

Where the proposed pavement structure meets an existing pavement structure, the following recommendations should be followed:

- ❑ A 300 mm wide section of the existing asphalt roadway should be saw cut from the existing pavement edge to provide a sound surface to abut the proposed pavement structure.
- ❑ It is recommended to mill a 300 mm wide and 40 mm deep section of the existing asphalt at the saw cut edge.
- ❑ The proposed pavement structure subbase materials should be tapered no greater than 3H:1V to meet the existing subbase materials.
- ❑ Clean existing granular road subbase materials can be reused upon assessment by Paterson at the time of excavation (construction) as to its suitability.

Light Post Construction

It is expected that light posts will be constructed within the proposed development. Light post pole bases are considered unheated structures and therefore their subgrade would require a minimum of 2.1 m of soil cover or equivalent to be protected from frost action. Generally, it is recommended that the post bases be founded below the depth of frost migration for unheated structures.

Furthermore, it is anticipated that the proposed pole bases are to be installed using open excavation and will be founded upon an undisturbed very stiff silty clay. The site excavated material is considered frost susceptible, and not suitable for backfill directly against the bases.

To mitigate potential frost action from the backfill material surrounding the bases, it is recommended that the buried portion of the bases be surrounded by a layer of engineered fill such as OPSS Granular A or Granular B Type II with a minimum thickness of 600 mm surrounding the pole base. This thickness may be reduced to 300 mm if the proposed bases are surrounded by a suitable non-fixed casing material such as a sono-tube shell or other approved product capable of remaining intact below the ground surface.

The pole bases may be founded on a silty clay bearing surface once approved by Paterson at the time of construction. Based on these recommendations, the bearing resistance values noted in Section 5.3 may be applied for end bearing. It should be noted that the pole bases founded on the silty clay deposit will be subject to post-construction settlements of up to 25 mm.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended for proposed structures. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless placed in conjunction with a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure, 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. 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 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.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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.

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.

6.4 Pipe Bedding and Backfill

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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The bedding layer thickness should be increased to a minimum thickness of 300 mm if the subgrade consists of grey silt. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD. The bedding material should extent at a minimum to the spring line of the pipe.

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

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should consist of the soils exposed at the trench walls to minimize differential frost heaving. The brown silty clay above the cover material could be placed if the excavation and backfilling operations are conducted in dry and above-freezing weather conditions. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps if excavations will be restricted to being within the silty clay deposit.

Should site servicing and/or other excavations be advanced into the underlying saturated silt deposit, it would be recommended to carry out dewatering of this layer in advance of those excavations. Additional details may be provided by Paterson once preliminary site servicing plans have been provided for our review.

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.

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR).

A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Persons as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the samples 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 a moderate to slightly aggressive environment.

6.8 Tree Planting Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks.

Atterberg limits testing was completed for the recovered silty clay samples at selected locations throughout the subject site. The soil samples were recovered from elevations below the anticipated design underside of footing elevation. The results of our testing are presented in Table 1 in Subsection 4.2 and in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 30% in all the tested clay samples.
In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture level and consistency, the silty clay across the subject site is considered to be a clay of low to medium potential for soil volume change.

The following tree planting setbacks are recommended for a clay of low to medium potential for soil volume change and where trees are located near buildings founded on cohesive soils.

❒ Large trees (mature height over 14 m) can be planted within these areas provided that a tree to foundation setback equal to the full mature height of the tree can be provided.

❒ Tree planting setback limits may be reduced to 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m), provided that the conditions noted below are met.

❒ A small tree must be provided with a minimum of 25 m^3 of available soils volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

❒ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

❒ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

❒ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the Grading Plan.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures.

Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e., Manitoba Maples) and, as such, they should not be considered in the landscaping design.

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.
- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.
- Review of architectural plans pertaining to foundation and underfloor drainage systems and waterproofing details for elevator shafts.

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

- Review and inspection of the installation of the foundation drainage systems.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than RioCan Holdings (TJV) Inc., Chick-Fil-A Canada ULC, 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
May 17, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Andre Benoist, EIT

David J. Gilbert, P.Eng.

Report Distribution:

- ❑ RioCan Holdings (TJV) Inc. (e-mail copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION RESULTS

ATTERBERG LIMITS TEST RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 333 Huntmar Drive
Ottawa, Ontario

DATUM Geodetic

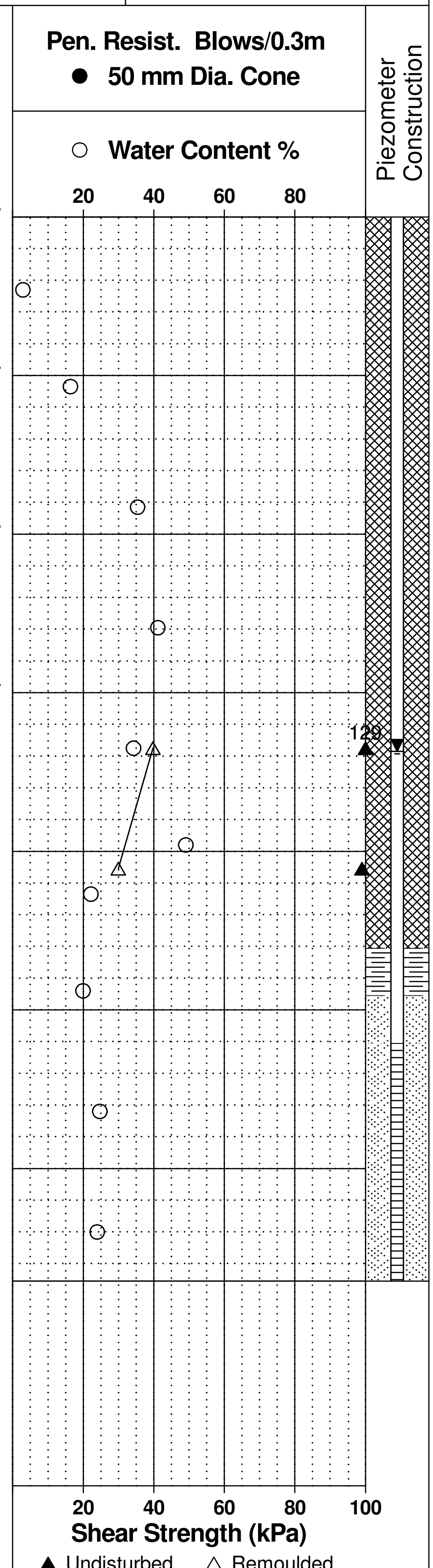
REMARKS

BORINGS BY Track-Mount Power Auger **DATE** February 6, 2023

FILE NO. PG6571

HOLE NO. BH 1-23

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.97
TOPSOIL 0.05						
FILL: Brown silty sand to sandy silt with gravel and crushed stone 0.69	AU	1				
Compact, brown **SILTY SAND to SANDY SILT** 1.45	SS	2	33	16	1	102.97
Very stiff to stiff, brown **SILTY CLAY** 4.27	SS	3	83	11	2	101.97
	SS	4	100	4		
	SS	5	100	P	3	100.97
	SS	6	100	P	4	99.97
Compact to very loose, brown **SILT,** trace fine sand - grey by 4.5m depth 6.71	SS	7	54	12	5	98.97
	SS	8	58	2		
	SS	9	75	2	6	97.97
End of Borehole (GWL @ 3.37m - Feb. 14, 2023)						



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 333 Huntmar Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE February 6, 2023

FILE NO. PG6571

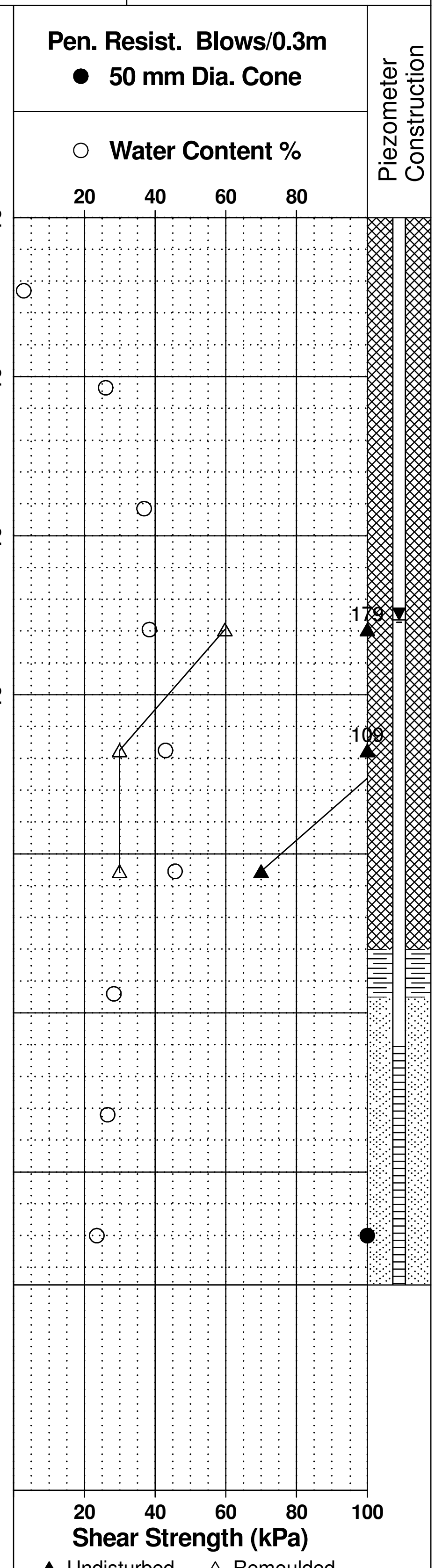
HOLE NO. BH 2-23

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.05				
FILL: Brown silty sand to sandy silt with clay, gravel and crushed stone		AU	1		
	1.45	SS	2	29	6
Very stiff to stiff, brown **SILTY CLAY**		SS	3	100	4
		SS	4	100	P
		SS	5	100	P
	4.11	SS	6	83	P
Very loose, grey **SILT,** trace fine sand		SS	7	75	1
		SS	8	50	2
	6.71	SS	9	50	1

DEPTH (m)	ELEV. (m)
0	103.02
1	102.02
2	101.02
3	100.02
4	99.02
5	98.02
6	97.02

Dynamic Cone Penetration Test attempted at 6.71m, cone refusal at 6.71m depth, borehole terminated.

(GWL @ 2.53m - Feb. 14, 2023)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 333 Huntmar Drive
Ottawa, Ontario

DATUM Geodetic

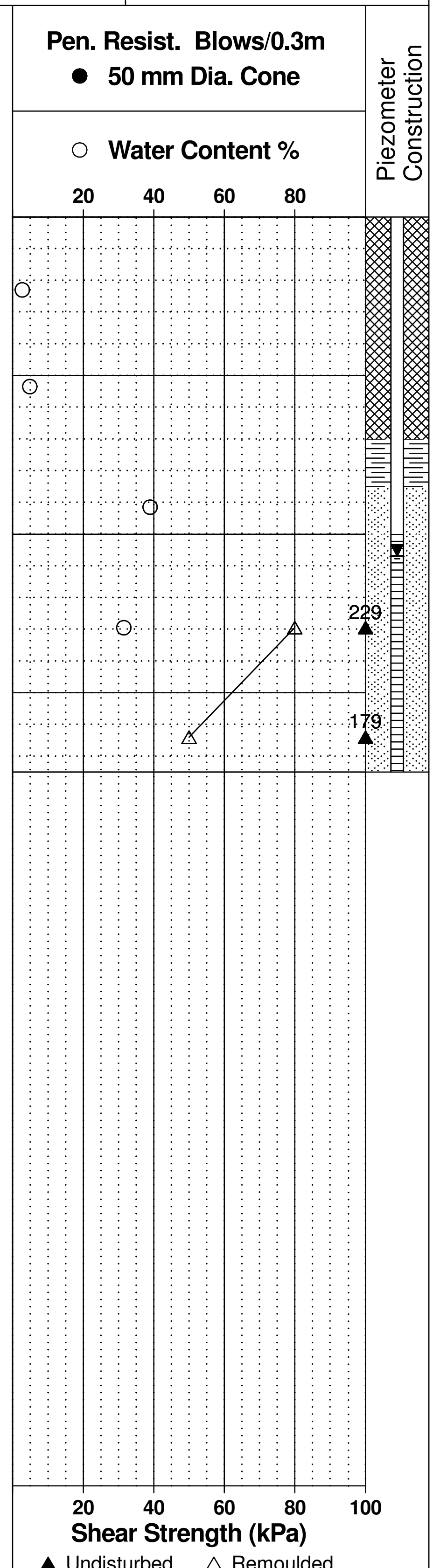
REMARKS

BORINGS BY Track-Mount Power Auger **DATE** February 6, 2023

FILE NO. PG6571

HOLE NO. BH 3-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.91
TOPSOIL 0.05							
FILL: Brown silty sand to sandy silt with gravel and crushed stone 0.69		AU	1				
Hard to very stiff, brown **SILTY CLAY**		SS	2	42	6	1	101.91
		SS	3	100	5	2	100.91
		SS	4		P	3	99.91
3.50 End of Borehole							
(GWL @ 2.14m - Feb. 14, 2023)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 333 Huntmar Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

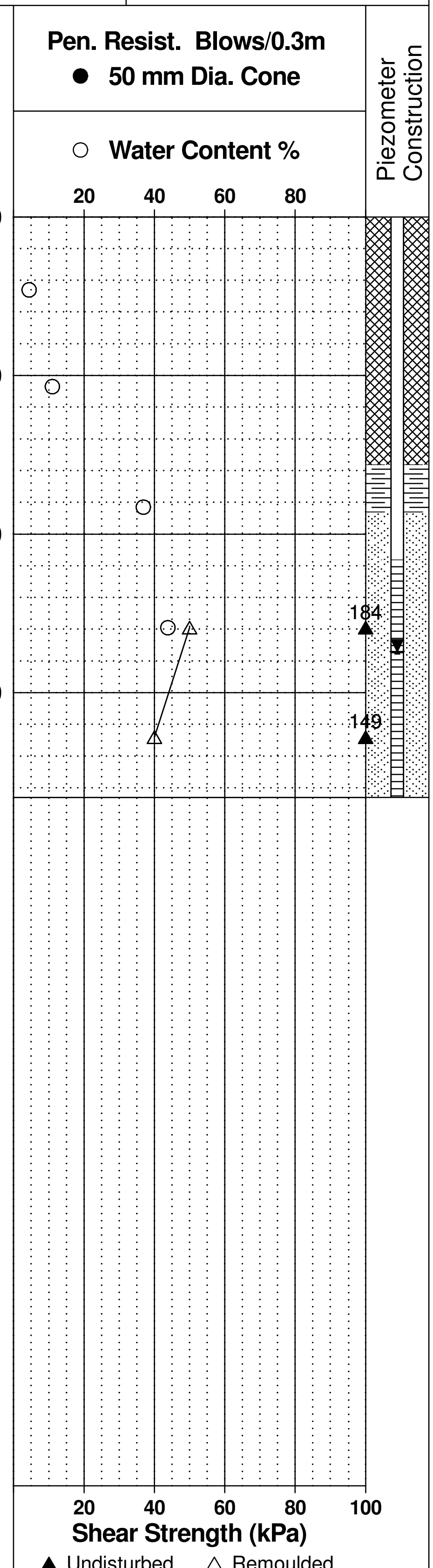
BORINGS BY Track-Mount Power Auger

DATE February 6, 2023

FILE NO. PG6571

HOLE NO. BH 4-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.69
TOPSOIL 0.05							
FILL: Brown silty sand to sandy silt with gravel and crushed stone, trace clay		AU	1				
		SS	2	50	24	1	102.69
1.45							
Very stiff, brown **SILTY CLAY**		SS	3	100	8	2	101.69
		SS	4	100	P		
						3	100.69
3.66							
End of Borehole (GWL @ 2.74m - Feb. 14, 2023)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 333 Huntmar Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

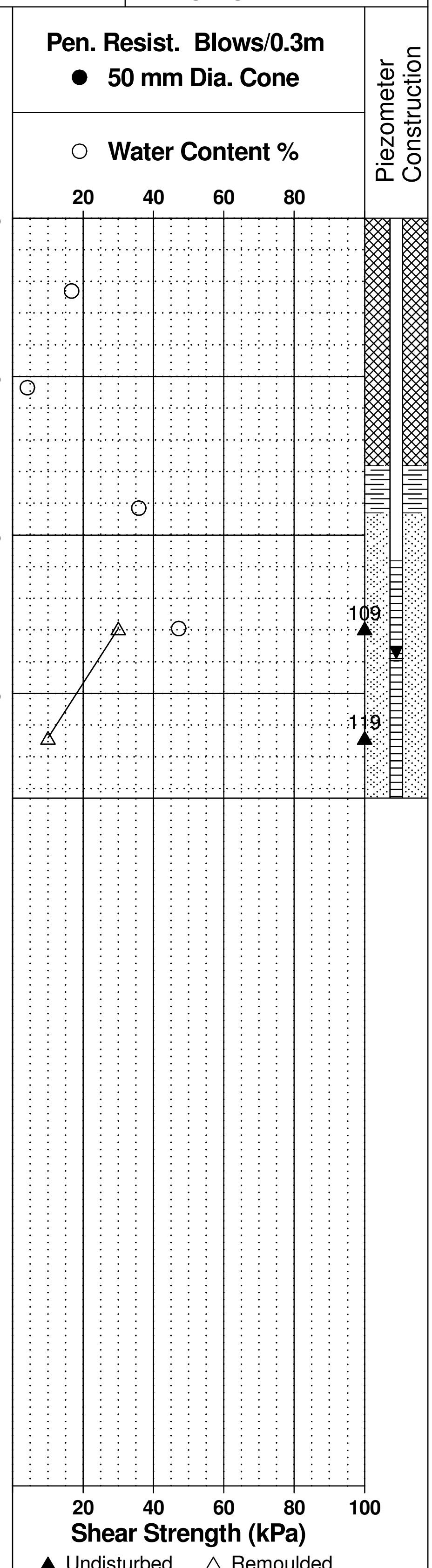
BORINGS BY Track-Mount Power Auger

DATE February 6, 2023

FILE NO. PG6571

HOLE NO. BH 5-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.75
TOPSOIL 0.05							
FILL: Brown silty sand to sandy silt with gravel and crushed stone		AU	1				
		SS	2	33	18	1	102.75
1.45							
Very stiff, brown **SILTY CLAY**		SS	3	71	7	2	101.75
		SS	4	100	P	3	100.75
3.66							
End of Borehole							
(GWL @ 2.78m - Feb. 14, 2023)							



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

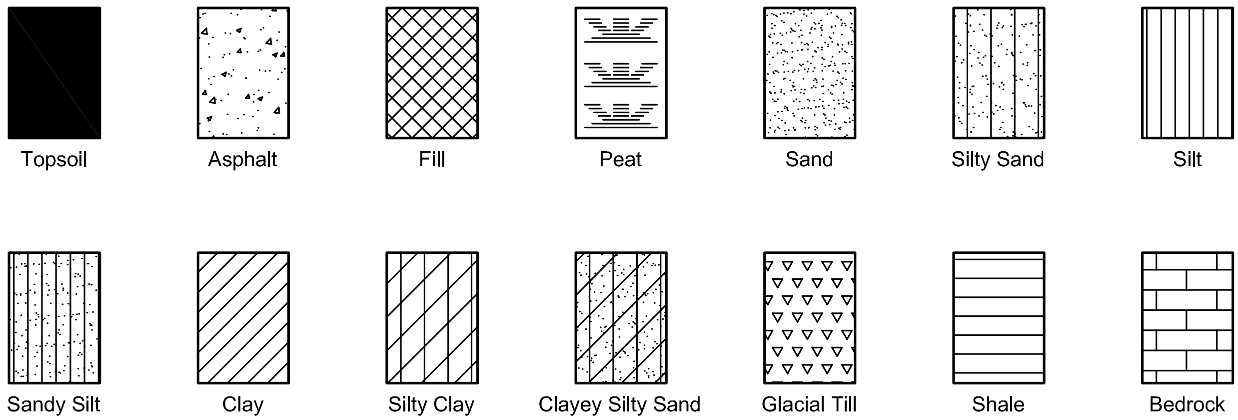
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

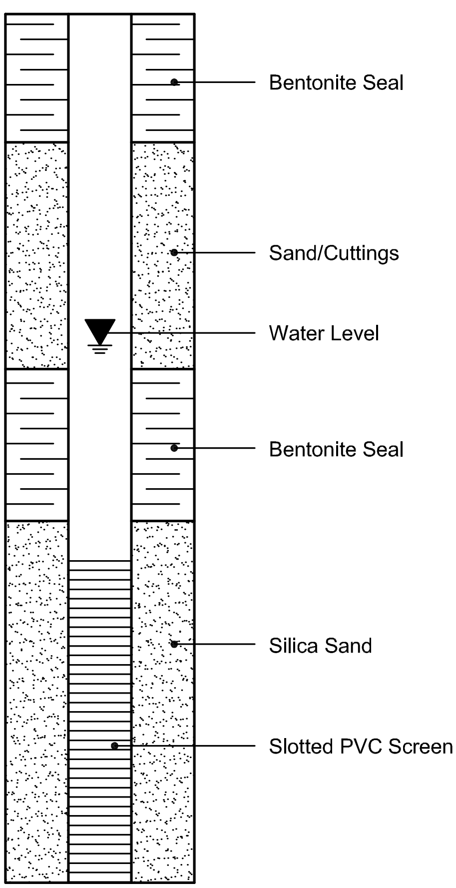
SYMBOLS AND TERMS (continued)

STRATA PLOT

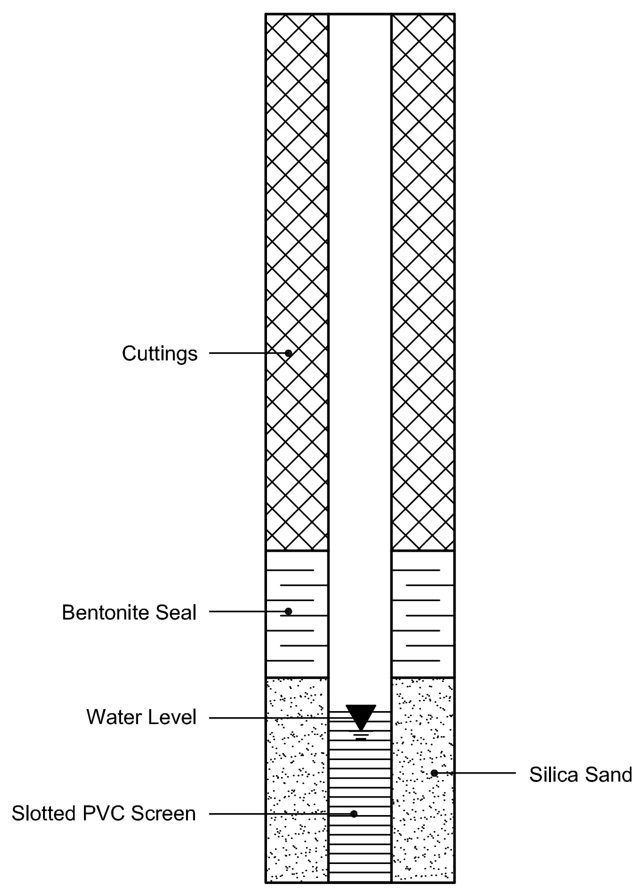


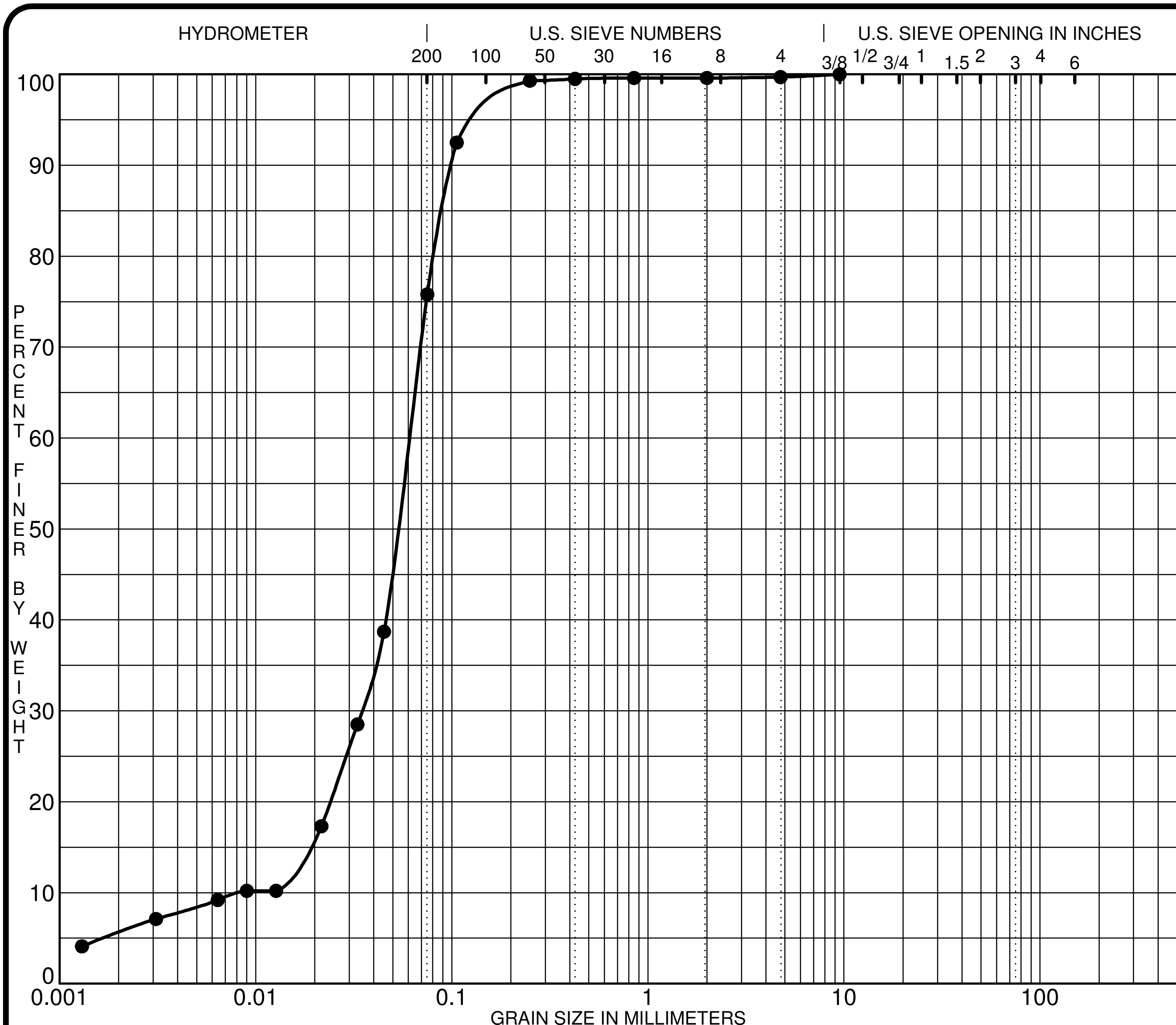
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 1-23**	**SS7**						**2.35**	**7.2**
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 1-23**	**SS7**	**9.50**	**0.06**	**0.035**	**0.0084**	**0.3**	**23.9**	**75.8**	
⊠										
▲										
★										

CLIENT **Timmins Square LP**

FILE NO. **PG6571**

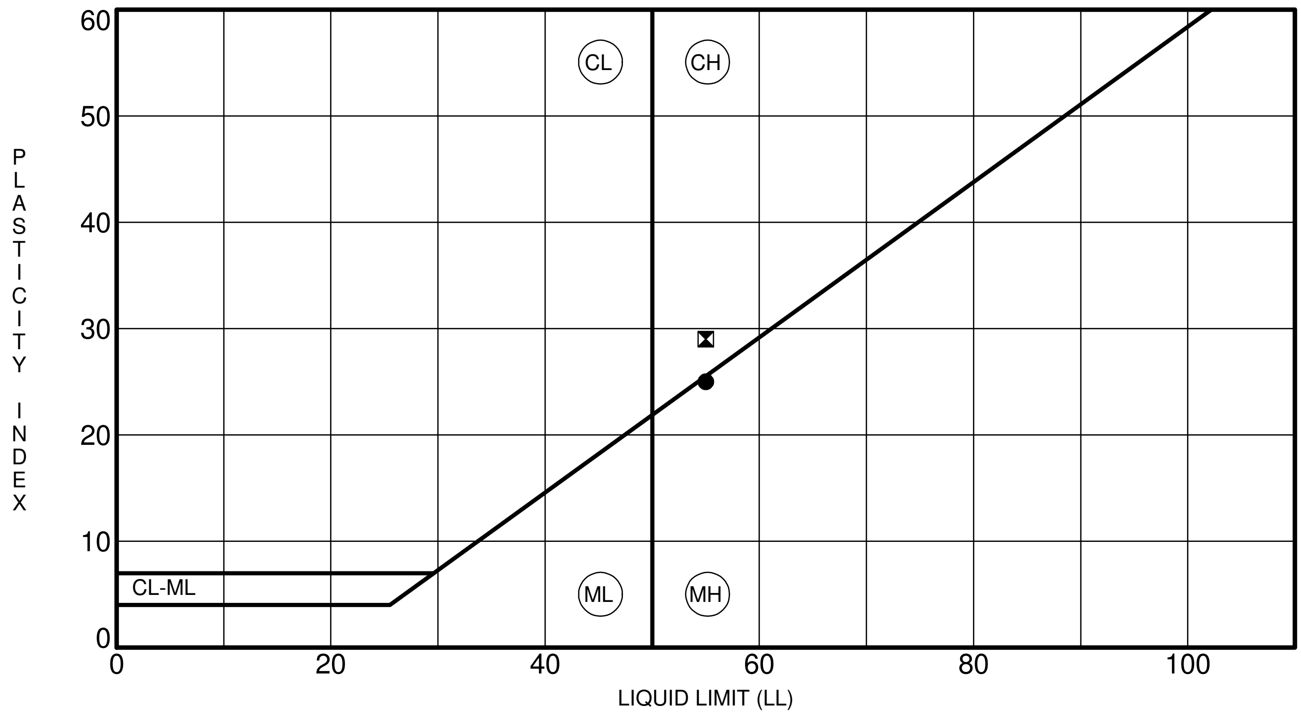
PROJECT **Geotechnical Investigation - Prop. Commercial Development - 333 Huntmar Drive**

DATE **6 Feb 23**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-23**	**SS3**	**55**	**30**	**25**		**MH - Inorganic silt of high plasticity**
⧗	**BH 2-23**	**SS4**	**55**	**26**	**29**		**CH - Inorganic clay of high plasticity**

CLIENT **Timmins Square LP**

FILE NO. **PG6571**

PROJECT **Geotechnical Investigation - Prop. Commercial Development - 333 Huntmar Drive**

DATE **6 Feb 23**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: **56793**

Report Date: 15-Feb-2023

Order Date: 9-Feb-2023

Project Description: PG6571

	Client ID:	BH2-23-SS3	-	-	-
	Sample Date:	09-Feb-23 00:00	-	-	-
	Sample ID:	2306399-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	71.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.54	-	-	-
Resistivity	0.10 Ohm.m	10.8	-	-	-
Anions					
Chloride	10 ug/g dry	366	-	-	-
Sulphate	10 ug/g dry	148	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6571-1 - TEST HOLE LOCATION PLAN

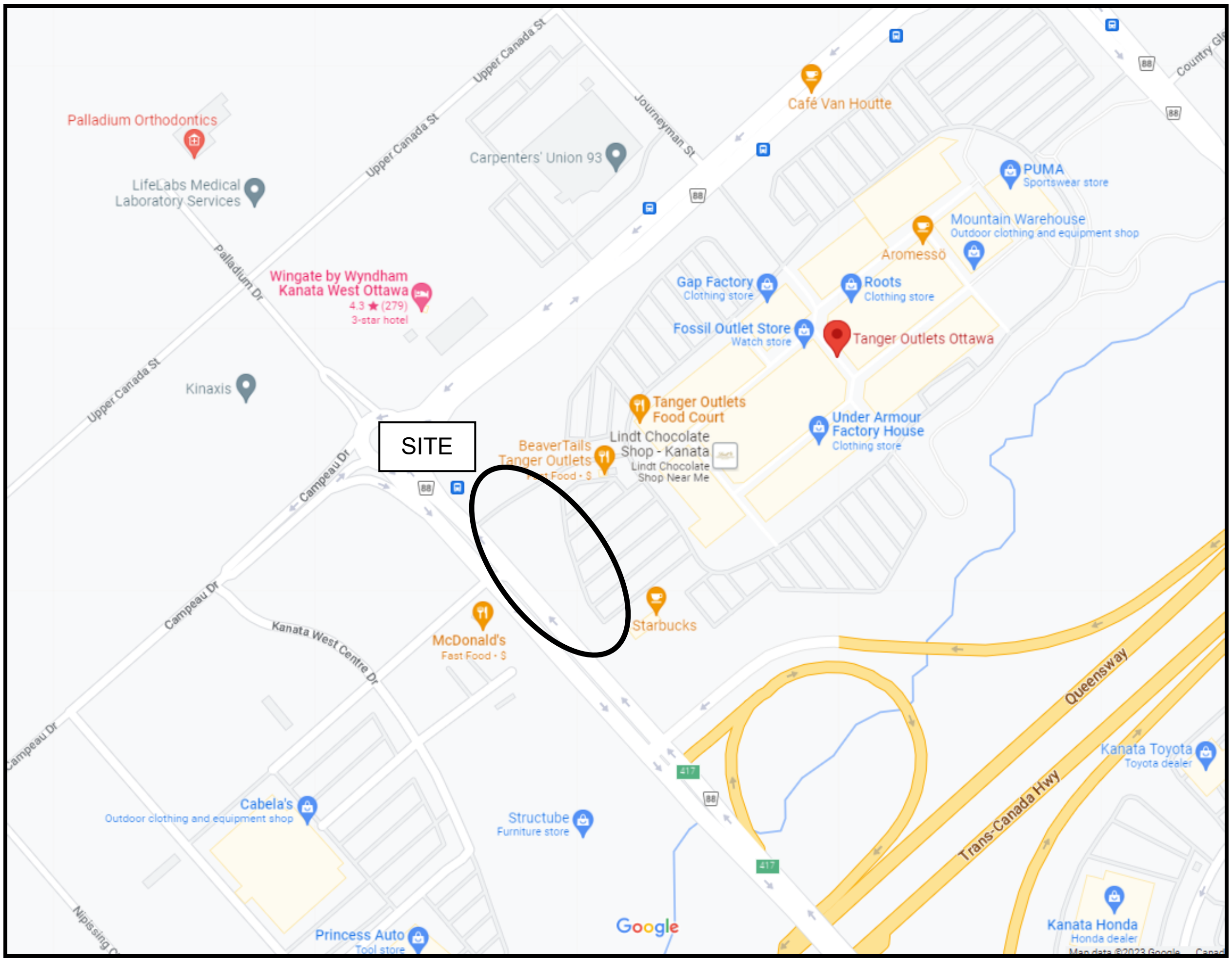


FIGURE 1

KEY PLAN

PATERSON GROUP

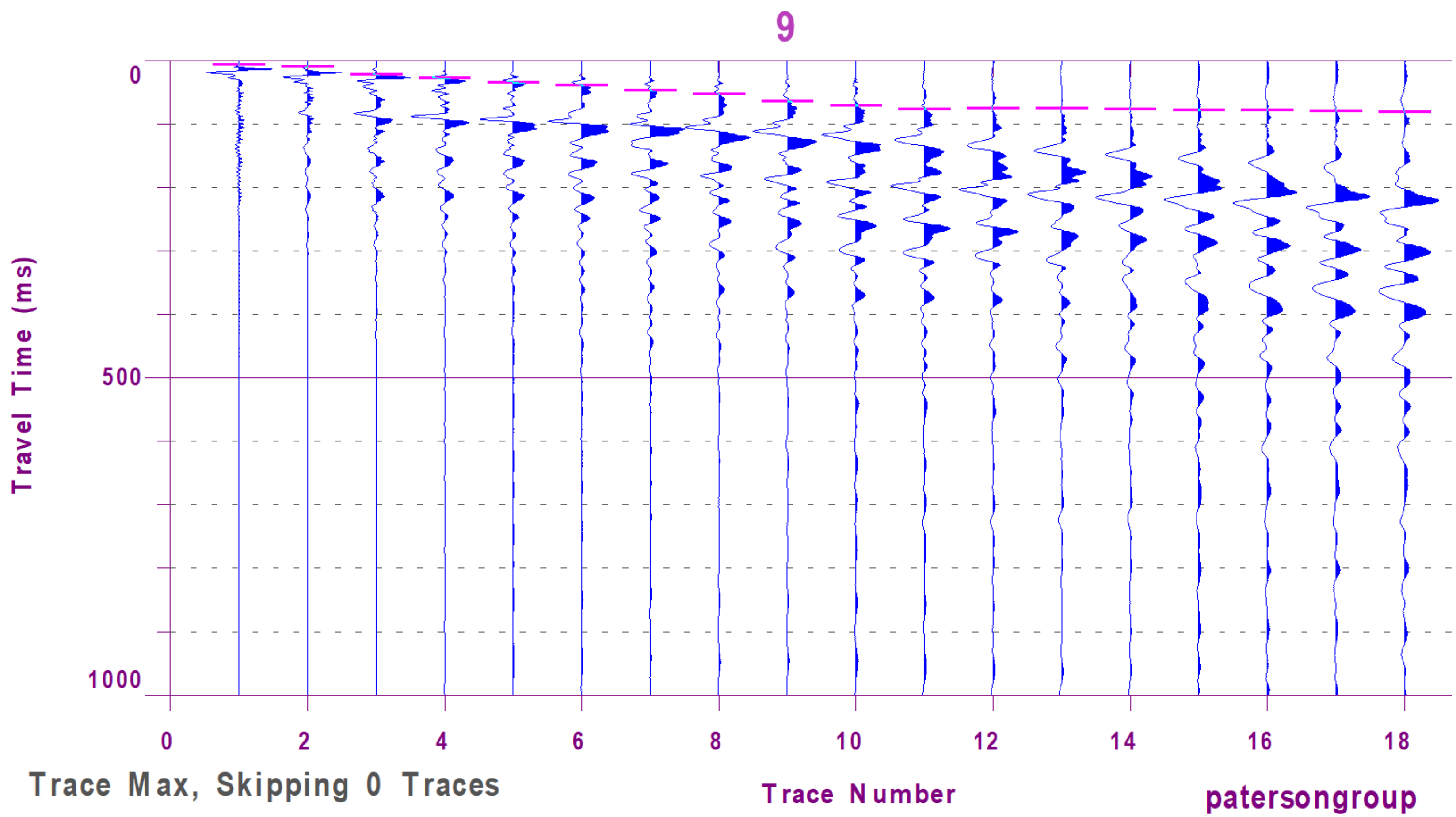


Figure 2 – Shear Wave Velocity Profile at Shot Location -2 m

PATERSON GROUP

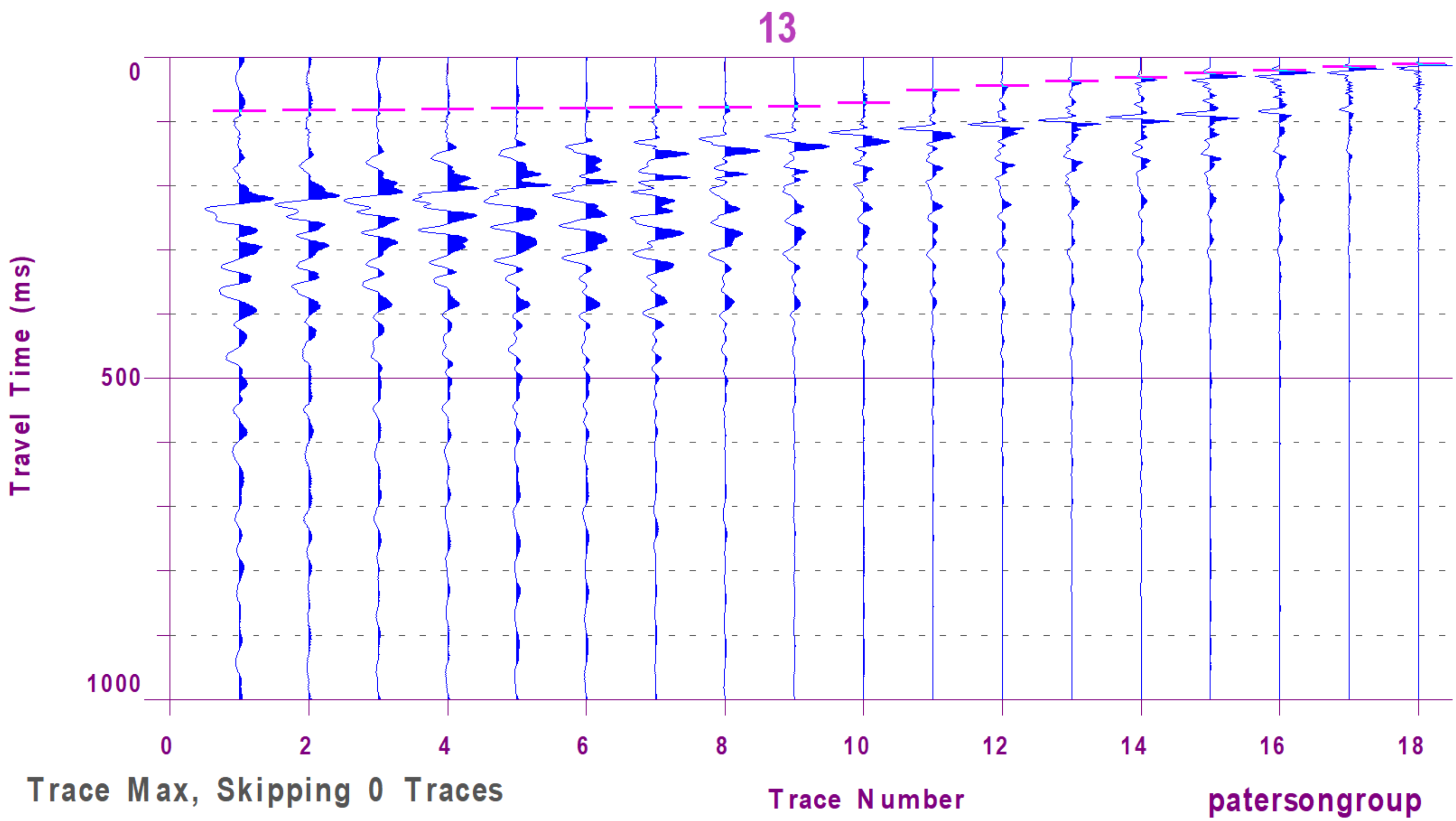
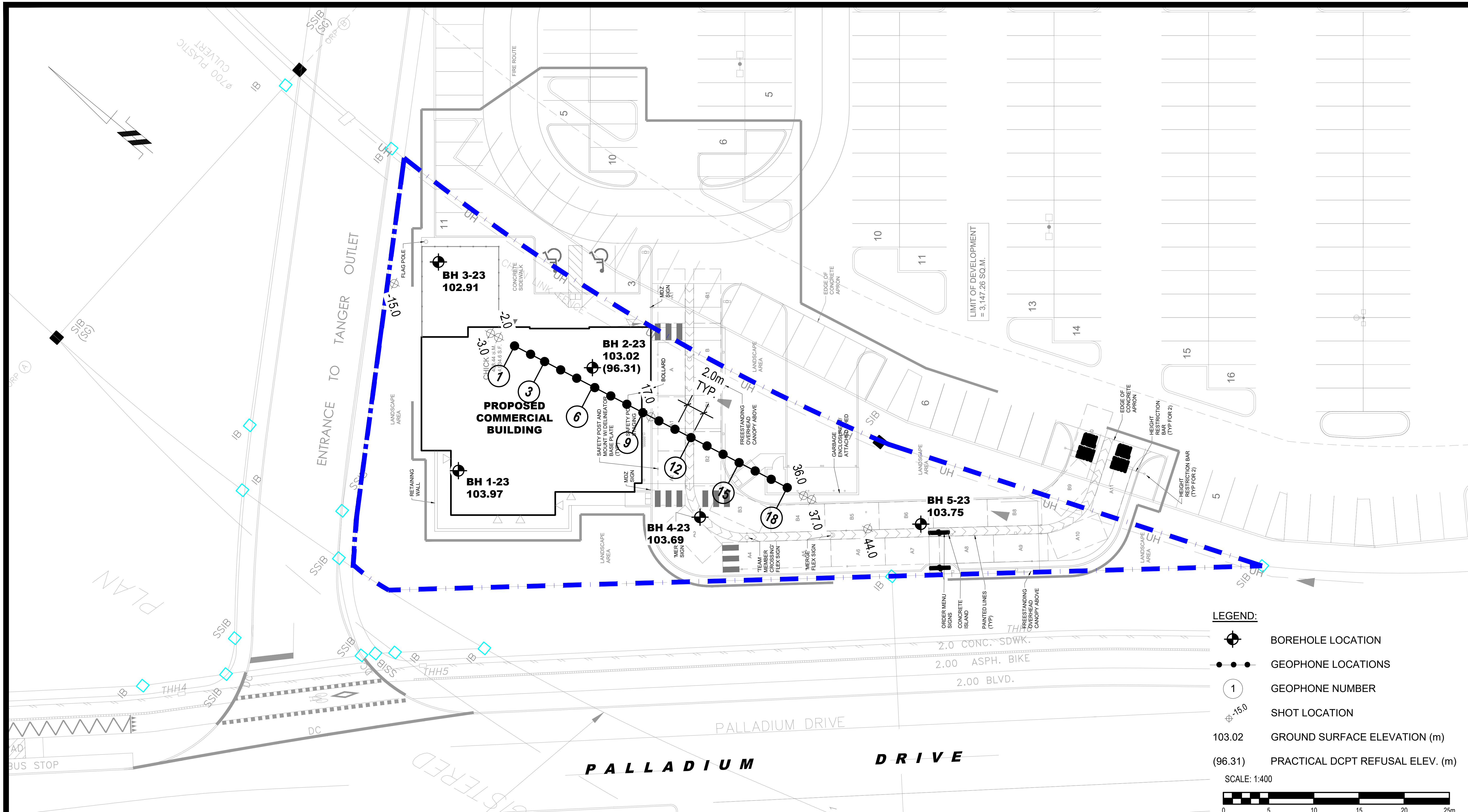


Figure 3 – Shear Wave Velocity Profile at Shot Location 36 m



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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED SEISMIC SHEAR WAVE VELOCITY TEST LOCATION	15/05/2023	FC

RIOCAN HOLDINGS (TJV) INC. c/o CHICK-FIL-A CANADA ULC

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL BUILDING - 333 HUNTMAR DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 02/2022
Drawn by: MPG	Report No.: PG6571-1
Checked by: AB	Dwg. No.: PG6571-1
Approved by: DP	Revision No.: 1

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11x17