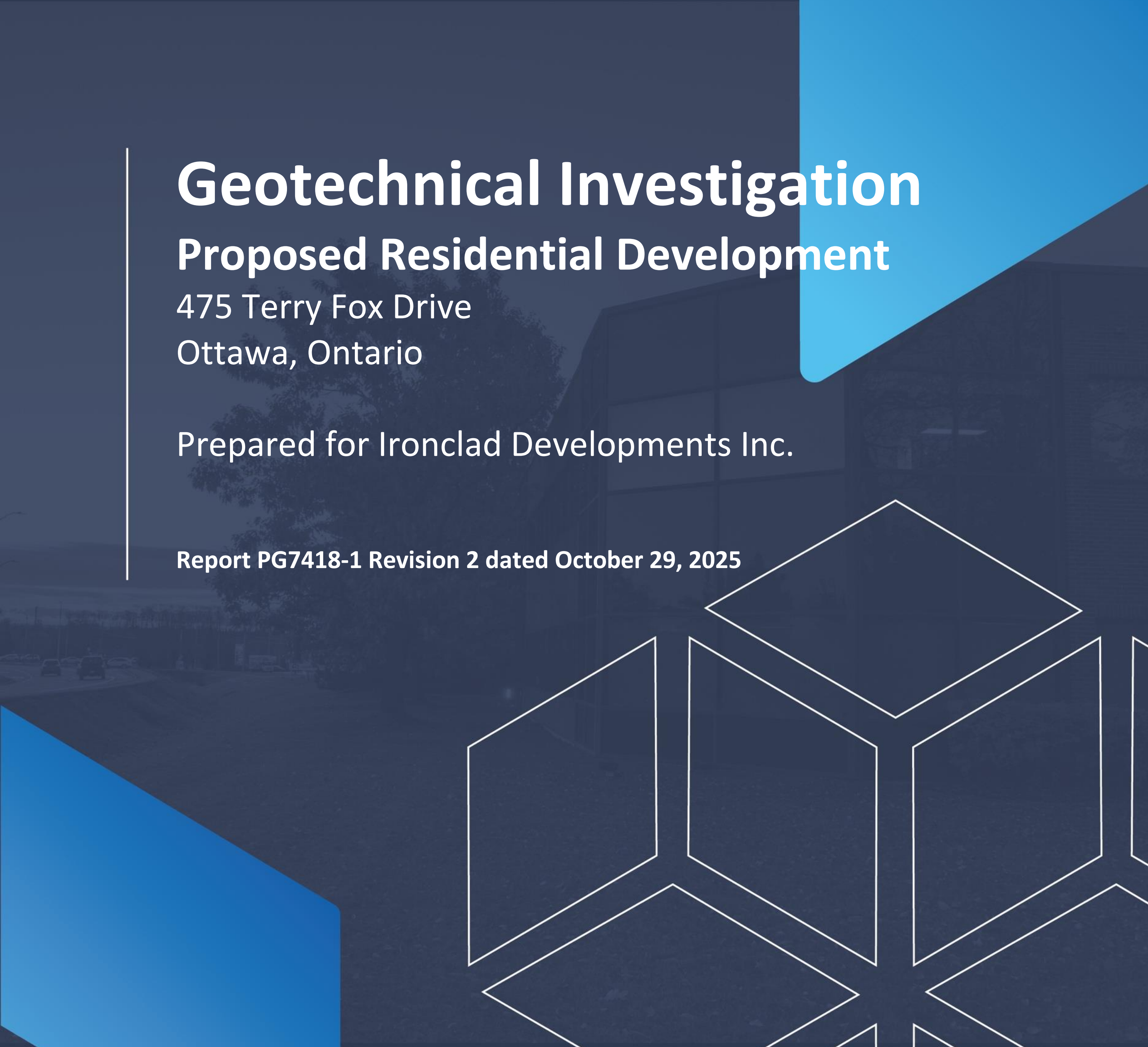
PATERSON GROUP



Geotechnical Investigation

Proposed Residential Development

475 Terry Fox Drive
Ottawa, Ontario

Prepared for Ironclad Developments Inc.

Report PG7418-1 Revision 2 dated October 29, 2025

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ironclad Developments Inc. to conduct a geotechnical investigation for the proposed residential development to be located at 475 Terry Fox Drive in the City of Ottawa (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

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

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

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

2.0 Proposed Development

Based on the available conceptual site plans provided by the client, it is anticipated that the future development will consist of three (3) multi-storey residential buildings. All three buildings (Building A, B and C) are anticipated to be six (6) storeys in height. Paterson understands each building will be provided with one level of underground parking, and that Building A and Building B will share and be connected by their basement level.

Associated access lanes, at-grade parking, landscaped and hardscaped areas and a retaining wall are also anticipated as part of the development. The development is anticipated to be municipally serviced and will be supported by municipal service extensions throughout Terry Fox Drive.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out from February 5 to 7, 2025 and consisted of advancing a total of 10 test pits and 5 boreholes below the existing ground surface to a maximum depth of 10.4 m and 4.1 m, respectively.

The test hole locations were distributed in a manner to provide general coverage of the subject site while taking into consideration underground utilities and site features. The test pit and borehole locations are shown on the attached Drawing PG7418-1 – Test Hole Location Plan included in Appendix 2.

The test pits were advanced using an excavator. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from our geotechnical department. The test pitting procedure consisted of advancing to the required depths at select locations, sampling and testing the overburden.

The boreholes were completed using a rubber-track low-clearance auger drilling rig operated by a two-person crew. The testing procedure consisted of augering, excavating, and coring to the required depth at the selected location and sampling the overburden and/or bedrock.

A previous subsurface investigation was undertaken by Paterson in October 2015 within the subject site. At that time, 6 boreholes were advanced throughout the subject site to a maximum depth of 12.6 m below the ground surface. The test hole locations from the previous study are shown on Drawing PG7418-1 - 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. The bedrock was cored to assess the bedrock type and quality. 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 (RC) 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. Photographs of the rock core are presented in Appendix 1.

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

Diamond drilling was completed at borehole BH 2-25 to confirm the bedrock type and quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage.

The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

The overburden thickness was evaluated by completing dynamic cone penetration tests (DCPT) at boreholes BH 1-25 and BH 3-25. A DCPT test was completed at borehole BH 4 of the previous investigation. 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.

Soil samples from the test pits were recovered from the side walls of the open excavation. Grab samples were collected from the test pits at selected intervals. The samples were initially classified on site, placed in sealed plastic bags and transported to our laboratory. The depths at which the grab samples were recovered from the test pits are shown as G on the Soil Profile and Test Data sheets in Appendix 1.

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The subsurface conditions observed in the test pits and boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data Sheets in Appendix 1 of this report. The Soil Profile and Test Data sheets from the previous subsurface investigation are also included in Appendix 1.

Groundwater

For the current investigation, monitoring wells were installed in BH 1-25 and BH 2-25. Flexible polyethylene standpipe piezometers were installed in the remainder of the boreholes from the current investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program

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

Monitoring Well Installation

Typical monitoring well construction details are described below:

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

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

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The locations of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG7418 - 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 one linear shrinkage analysis, two grain size distribution testing, and two Atterberg limit tests were completed on selected soil samples. One unconfined axial compressive strength test was completed on a select rock core sample. Moisture content testing was complete on all recovered soil samples from the current investigation. The results of the testing are presented in Section 4.2 and are provided in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless directed otherwise.

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 submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

3.5 Hydraulic Conductivity Testing

Hydraulic conductivity testing was conducted at one (1) monitoring well location to provide insight on the hydraulic properties of the bedrock at the subject site. The test data was analyzed using AQTESOLV Pro Version 4.5 aquifer analysis software package by HydroSOLVE Inc and the results were processed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous aquifer of infinite extent and a screen length significantly greater than the monitoring well diameter.

The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 1.5 m and a diameter of 0.03 to 0.05 m. While the idealized assumptions regarding aquifer extent and homogeneity are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site. The testing results are further discussed in Subsection 4.4 of this report.

4.0 Observations

4.1 Surface Conditions

The majority of the subject site currently consists of undeveloped vacant grassed land, with the exception of a sparsely treed area within the central portion of the parcel. The subject site is bordered by Terry Fox Drive along the southwestern property boundary, by Kanata Avenue along the northwestern property boundary, and by residential dwellings along the eastern property boundary. A traffic light intersection between Kanata Avenue and Terry Fox Drive was noted beyond the western property boundary.

The ground surface across the subject site is relatively flat, sloping downwards gently from east to west between approximate geodetic elevations of 102.5 to 95 m. The grade across the site is relatively similar to the adjacent roadways. An existing ditch was noted along the northwestern and southwestern property boundaries along Kanata Avenue and Terry Fox Drive, respectively. The ditch along Kanata Avenue was noted to be relatively flat and at grade with the surrounding ground surfaces, while the ditch along Terry Fox Drive was noted to be approximately 1 m deeper than the surrounding ground surfaces.

4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consists of a 0.1 to 0.3 m thick layer of topsoil underlain by a layer of fill. A deposit of brown to grey silty clay was noted below the fill, which was further underlain by glacial till and further by the underlying bedrock formation.

Fill was observed extending to depths between 0.5 and 2.2 m below the existing ground surface at all test hole locations, with the exception of TP 2-25, TP 6-25, and TP 10-25. The fill was generally observed to consist of silty clay with sand, gravel, and organics.

The fill layer was observed to generally be underlain by a deposit of silty clay. The silty clay deposit consisted of very stiff to firm, brown silty clay which extended to approximate depths between 2.0 to 5.3 m below the ground surface. The brown silty clay layer was observed to be underlain by a layer of stiff to firm, grey silty clay which extended to approximate depths between 4.1 to 8.3 m below ground surface.

In several areas, the fill material was seen to be underlain by a 0.3 to 0.4 m thick layer of sandy silt, notably at TP 4-25, TP 5-25, and TP 7-25. A 0.4 m thick layer of silty sand was noted below the fill layer at BH 2-25.

The clay deposit was observed to be underlain by a deposit of glacial till. The glacial till deposit generally consists of loose to dense, grey silty sand or silty clay with variable amounts of clay, sand, gravel, cobbles and rock fragments. The glacial till deposit was observed to extend to approximate depths between 1.1 and 10.4 m below ground surface.

Practical refusal to excavation, augering, or DCPT testing was encountered at depths ranging from 0.2 to 10.4 m below the existing ground surface at several test hole locations across the subject site.

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

Bedrock

Granite with interbedded granodiorite bedrock was cored in BH 2-25 to a depth of 6.9 m below ground surface. The recorded average RQD values ranged from 83 to 58, while the recovery values were consistently 100%. Based on these results the quality of the bedrock ranges from good to fair quality.

Based on available geological mapping, the bedrock in the subject area consists of intrusive igneous bedrock of the Precambrian period, with an overburden drift thickness ranging between 0 to 10 m depth.

Atterberg Limits Testing

Atterberg limits testing as well as shrinkage limit testing was completed on recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH 2-25 SS4	2.3 – 2.9	68	32	36	54.7	CH
BH 5A-25 SS7	4.6 – 5.2	53	24	29	59.2	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
CH: Inorganic Clays of High Plasticity

The results of the moisture content tests are presented on the Soil Profile and Test Data Sheet in Appendix 1.

Unconfined Compressive Strength Testing of Bedrock Core Sample

One (1) select bedrock core sample obtained by Paterson as part of the current investigation was tested for unconfined compressive strength. The results of the test are summarized in Table 2 below and presented on Unconfined Compressive Strength Testing Results on Appendix 1.

Table 2 – Summary of Unconfined Bedrock Compressive Strength Testing Results

Borehole	Sample	Test Core Depth (m)	Test Core Elevation (m)	Unconfined Compressive Strength (MPa)
BH 2-25	RC1	4.73 – 4.82	91.62 – 91.53	107.7

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer) analysis was completed on a selected recovered silty clay and glacial till deposit sample. The result of the grain size distribution analysis is presented in Table 3 and on the Grain Size Distribution sheets in Appendix 1.

Table 3 – Grain Size Distribution Results

Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 1-25 SS5	4.6 – 5.2	0.3	2.6	50.6	46.5
BH 1-25 SS8	9.1 – 9.8	31.3	54.3	12.3	2.2

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

Shrinkage Testing

Linear shrinkage testing was completed on a silty clay sample recovered at a depth of 2.3 – 2.9 m from BH 2-25 and yielded a shrinkage limit of 19.35 and a shrinkage ratio of 1.78.

4.3 Groundwater

Groundwater levels were recorded at each test hole location on February 18, 2025, and are presented in Table 4 below. The groundwater level readings are also presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 4 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-25	96.43	2.13	94.30	February 18, 2025
BH 2-25	96.35	2.10	94.25	
BH 4-25	95.78	Frozen		
BH 5A-25	97.17	2.61	94.56	

Note: The ground surface elevation at each borehole location was referenced to a geodetic datum

It should be noted that surface water can become trapped within a backfilled borehole column, which can lead to higher-than-normal groundwater level readings. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, groundwater levels could vary at the time of construction

4.4 Hydraulic Conductivity Testing Results

Hydraulic conductivity tests were conducted at two monitoring well locations throughout the subject site during the month of February 2025. The testing results are summarized in Table 5 below.

Table 5 – Summary of Hydraulic Conductivity Testing Results.

Test Hole ID	Ground Surface Elevation (m)	Testing Depth Interval (m)	Testing Elevation Interval (m)	K (m/s)	Test Type	Soil Type
BH 1-25	96.43	7.31-8.83	89.12-87.60	$2.29\text{x}10^{-7}$	Falling Head	Glacial Till
BH 2-25	96.35	5.33-6.86	91.02-89.49	$7.15\text{x}10^{-5}$	Falling Head	Bedrock
				$7.11\text{x}10^{-5}$	Rising Head	

Summary of Results

Hydraulic conductivity testing conducted at monitoring well screened within the glacial till yielded a hydraulic conductivity value of approximately 2.29×10^{-7} m/s. Hydraulic conductivity testing conducted at monitoring well screened within the bedrock yielded hydraulic conductivity values ranging between approximately 7.11×10^{-5} to 7.15×10^{-5} m/s.

These values are generally consistent with typical published values for glacial till, and bedrock. It should be noted that hydraulic conductivity may vary across the subject site depending on the composition/compaction and hydrostatic properties at a given location for the overburden and bedrock, respectively.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. The proposed buildings may be founded on conventional shallow foundations placed on an undisturbed, very stiff silty clay, compact glacial till and/or placed directly upon a clean, surface sounded bedrock bearing surface. Due to the transition in subsoil conditions for the area of Building A and Building B, the Building B portion of the parking garage structure may be required to be supported by the use of a raft foundation if the provided bearing resistance values for conventional spread footings are insufficient to support design building loads, as further discussed in Subsection 5.3.

If structural loading exceeds the bearing resistance values provided herein for conventional spread footing foundations and raft slabs, a deep foundation support system may alternatively be considered as foundation support for the proposed structure.

Bedrock removal is expected to be required to complete the excavation of the proposed basement levels for several buildings. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. All contractors should be prepared for bedrock and oversized boulder removal. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations. Prior to undertaking blasting, pre-construction surveys will be required to be undertaken.

Due to the presence of a silty clay layer, proposed grading throughout the subject site will be subjected to a permissible grade restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Due to the shallow nature of the bedrock surface throughout the northern portion of the subject site, it is expected that the footings of the proposed buildings may extending below the bedrock surface. As such, it is expected all overburden material throughout those areas will be excavated from within the proposed building footprints.

Existing construction remnants, such as foundation walls and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade, or the underlying bedrock formation.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming, rock grinding and controlled blasting will be required to remove the bedrock for the underground parking levels for the proposed buildings. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed the below noted vibration limits during the blasting program to reduce the risks of damage to the existing surrounding structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using near vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing of the overburden.

Vibration Considerations

Construction operations are the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited. Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations.

As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Overbreak in Bedrock

Igneous bedrock formations, such as granite, often contain inclusions of other rock types, like sedimentary or metamorphic rocks, which can create natural planes of weakness. These inclusions, known as xenoliths, differ in mineral composition and hardness from the surrounding igneous material, making them prone to fracturing more easily. When blasting occurs, these inclusion planes can guide the blast energy unpredictably, causing overbreak. Additionally, variations in lithology within the igneous rock, such as quartz-rich or feldspar-rich zones, as well as variations in heterogeneity of the rock, with varying textures, mineral compositions, and grain sizes, can lead to differential fracturing, further contributing to overbreak.

Weathering from glacial activity has also impacted the strength of the upper layers of bedrock in the Ottawa area. Glaciers, through processes of erosion and abrasion, can remove the surface material of the rock, exposing underlying layers that may have been subjected to prolonged weathering. The freeze-thaw cycles, along with the physical and chemical weathering associated with glaciation, can cause the upper layers of the igneous rock to become more fractured and weakened. These weathered zones are typically more prone to fragmentation when blasted, leading to overbreak that extends deeper than intended into the solid, unweathered rock below.

The above-noted differences in the physical properties of the rock, combined with natural discontinuities and the effects of weathering, can lead to complex failure patterns that may result in overbreak that are difficult to account for in the blast design.

Based on this, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of the building excavation footprint with conventional drill and blast bedrock removal methods. This could be reduced where consideration is given by the bedrock removal contractor to setback line-drilling from the perimeter of the building excavation footprint and to undertake grinding of the perimeter sidewalls. However, this is expected to result in additional time to complete the building excavation.

Overbreak is expected throughout the lowest level of blasting due to the combination of natural weaknesses from inclusion planes and varying lithology and heterogeneity of the in-situ bedrock surface, which all contribute to more easily fractured surface layers. It is very difficult to mitigate significant overblasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak is expected to add unforeseen time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas, existing ditches throughout the subject site and the western City of Ottawa right-of-way boulevard.

This material should be compacted in maximum 300 mm thick loose lifts to at least 98% of the material's SPMDD using a suitably sized vibratory sheepsfoot roller. The fill should be prepared by segregating all cobbles and boulders larger than 200 mm in diameter, significant amounts of organics (i.e., peat, topsoil, roots, stumps, logs, etc.) and inorganic debris (i.e., construction debris, plastics, PVC, metals, etc.).

Sampling and testing of the fill material for grain-size distribution and standard proctor values should be completed by Paterson prior to re-use of the subject fill. Frozen material may not be considered for this purpose. This process should be reviewed and approved by Paterson field personnel upon completion of each lift and who are experienced in reviewing the placement of soil fill in this manner. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as CCW MiraDRAIN 2000 or Delta-Teraxx.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved by Paterson 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 paved areas should be compacted to at least 100% of its SPMDD.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 100 mm. Where the fill is open graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized. Providing a heat source during winter construction may be recommended should compacted fill material is intended to be exposed for long periods of time.

Protection of Subgrade (Conventional Spread Footings)

It is anticipated that the grey silty clay and glacial till subgrade soils located below the water table will become readily disturbed by construction traffic. Therefore, it is recommended that a minimum 50 mm thick mud slab layer be placed over the prepared bearing medium for all footings once the bearing surface has been reviewed and approved by Paterson personnel, and that provisions be carried to sub-excavate readily disturbed saturated glacial till subgrade soils at the time of subgrade preparation.

The mud slab is recommended to consist of a minimum 15 MPa (28-day compressive strength) concrete, extend a minimum of 150 mm beyond all faces of the overlying footing, and should not be placed until the bearing medium has been reviewed and approved at the time of construction by Paterson field personnel.

Protection of Subgrade (Raft Foundation)

Where a raft foundation is utilized, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed, silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance to the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance due to drying, and immediately (i.e., within 48 hours) of exposing the clay bearing medium. It should be understood that the mud slab alone is not considered sufficient to mitigate the potential for the migration of frost within the clay bearing medium if construction is undertaken during winter conditions.

Compacted Granular Fill Working Platform (Deep Foundation)

Since it is expected the proposed buildings may be supported on a deep foundation, the use of heavy equipment would be required to install piles (i.e., pile driving crane) or other deep foundation elements. It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance to the underlying soil.

It is recommended that a minimum 600 mm thick layer of OPSS Granular B Type II crushed stone or a combination of blast-rock and OPSS Granular B Type II crushed stone be placed as working platform throughout the building footprint which will support heavy equipment to facilitate deep foundation installations. The working pad granular should be compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in maximum 300 mm thick lifts.

Once the piles have been driven and cut off, the working platform can be re-graded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for the slab structure.

5.3 Foundation Design

Bearing Resistance Values – Conventional Spread Footings

Footings placed on an undisturbed, soil bearing surface or surface sounded bedrock can be designed using the following bearing resistance values provided in Table 6 in the following page.

An undisturbed soil 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 footings. A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Table 6 - Bearing Resistance Values

Bearing Surface	Bearing Resistance Values (kPa)	
	SLS	ULS
Very Stiff Brown Silty Clay	150	225
Firm Grey Silty Clay	75	110
Compact Glacial Till	200	300
Granite Bedrock	-	4,000

Note: Strip and pad footings, up to 3 and 4 m wide, respectively, can be designed using the bearing resistance values provided for an undisturbed, brown silty clay bearing surface.

Strip and pad footings, up to 3 m wide can be designed using the bearing resistance values provided for an undisturbed, grey silty clay bearing surface.

Bearing resistance values for footing design should be confirmed on a per building basis by the Paterson personnel at the time of construction.

A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance values at ULS. Footings placed on an undisturbed soil bearing surface and designed using the bearing resistance values at SLS provided above will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance value provided herein will be subjected to negligible potential postconstruction total and differential settlements.

It is further recommended that a construction joint be provided to allow for differential settlement between portions of the structures founded directly upon bedrock and the remainder upon a soil bearing medium.

It is recommended that Paterson reviews structural design drawings for the proposed buildings foundations during the design phase to confirm appropriate bearing resistance values are being considered for the anticipated founding depth of the proposed foundations.

Raft Foundation (Building B and Building C)

Consideration could be given to using a raft foundation for Building B and Building C if design building loads exceed the design bearing resistance values provided for conventional spread footings. For design purposes, it was assumed that the base of the raft foundation would be located at an approximate depth of 3.5 to 4.5 m below the existing ground surface since one level of underground parking is anticipated for the subject buildings.

However, the currently provided raft slab bearing resistance value is considered subject to further review based on the actual potential excavation depth and founding elevation. Therefore, the current design value is considered preliminary and subject to further review and coordination during the future design phase. The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **70 kPa** will be considered acceptable for a raft supported on the undisturbed, firm grey silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **110 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the following assumptions for the raft foundation, the proposed buildings can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

Based on one underground parking level, it is expected that the raft foundation will be installed on the grey silty clay deposit. The modulus of subgrade reaction was calculated to be **3.0 MPa/m** for a contact pressure of **70 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Lean-Concrete In-Filled Trenches

Where bedrock is encountered below the design underside of footing elevation and a Site Designation X_A or a bedrock bearing medium is sought as part of the foundation design, consideration may be given to lowering the bearing surface to a suitable bedrock bearing medium by placing the footings on a lean-concrete in-filled trench extending to sound bedrock.

Footings placed on a lean-concrete in-filled trench extending to weathered to sound bedrock bearing surface may be designed using a bearing resistance value of **4,000 kPa (ULS)**. This may be accomplished by excavating near-vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (minimum 15 MPa, 28-day compressive) to the design underside of footing level.

Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below 1.5 m depth).

Once approved by Paterson, lean concrete can be poured up to the proposed founding elevation. The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

It is anticipated water will be perched upon the bedrock and within the overburden, and mostly within the glacial till layer. Where water infiltration cannot be controlled using open sumps within the excavation footprints, it is recommended to install a well point adjacent to excavation footprints to lower the water table in advance of sub-excavations, if required and as determined at the time of construction.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil (i.e., near-vertical trenches would not extend between underside of footing and across the overburden to the underlying bedrock formation), it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long-term total and differential settlements.

At the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the sub-excavation should be at least the proposed footing width plus 500 mm. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

End Bearing Piles

A deep foundation method, such as end bearing piles, may be considered as foundation support for the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 7. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

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

This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values.

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted by Paterson field personnel during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

Table 7 – Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1,100	9	27
245	11	1,050	1,250	9	31
245	13	1,200	1,400	9	35

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

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

Down Drag Loads

Due to the presence of the clay deposit below the subject site, and potential long-term degradation of organics within the clay deposit, down drag loads should be considered during the final design of the piles. Based on the available subsurface information, it is expected that the piles will be driven through approximately 8 to 10 m of clay. Assigning an adhesion factor of 1.0 (as per the Canadian Foundation Engineering Manual), the clay can be taken to have an ultimate adhesion of 30 kPa against the sides of the piles.

The down drag load is effectively applied to each pile at the location of the "neutral plane," where negative (i.e., down drag) skin friction becomes positive shaft resistance. In the case of the end-bearing piles at this site, the neutral plane will be located near the bedrock surface.

The down drag load is a structural pile capacity criterion and does not affect the geotechnical capacity of the piles. The structural axial capacity of the pile is governed by its structural strength at the neutral plane when subjected to the permanent load plus the down drag load. Transient live load is not to be included. At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to the permanent load plus the transient live load, but down drag load is to be excluded.

At the depth of the neutral plane where the down drag load is applied, the pile structure is well confined. The 5th edition of the Canadian Foundation Engineering Manual recommends that the allowable structural axial capacity of piles at the neutral plane, for resisting permanent load plus the down drag load, can be determined by applying a factor of safety of 1.5 to the pile material strength (steel yield and concrete 28-day compressive strength).

Settlement for Deep Foundations

Foundations supported by end-bearing piles terminating upon the bedrock surface herein will be subjected to negligible potential 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.

Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil.

Adequate lateral support is provided to bedrock bearing medium when a plane extending down and out from the bottom edges of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

A heavily fractured, weathered bedrock and/or overburden bearing medium will require a lateral support zone of 1H:1V (or flatter).

Frictional Resistance

An unfactored coefficient of friction of 0.7 is considered applicable for the design of concrete footings supported on clean, surface sounded bedrock at this site.

Proof Rolling and Subgrade Improvement for Loose Soils Below Footings

Where the glacial till bearing surface for foundations is considered loose or disturbed by previous dewatering by Paterson at the time of construction, it may be recommended to proof roll the bearing surface using suitable compaction equipment prior to forming for foundations. Improving the bearing surface compaction will provide a suitable sand bearing medium.

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

It should be understood that the glacial till layer is susceptible to disturbance by heavy equipment and construction traffic. Therefore, it is recommended that the layer be reviewed by Paterson personnel once exposed to evaluate its suitability for the placement of a mud slab (as described in Section 5.2) or improvement prior to placing a mud slab.

Permissible Grade Raise Recommendations

Our current permissible grade raise recommendations for the proposed development are presented on Drawing PG7418-2 Permissible Grade Raise Plan in Appendix 2.

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. Provided sufficient time is available to induce the required settlements, consideration could be given to surcharging the subject site.

5.4 Design for Earthquakes

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

Field Program

The seismic array was located as presented in Drawing PG7418-1 - Test Hole Location Plan attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

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

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 15.0, 10.0, 3.0 and -3.0 m away from the first and last geophone, and at the centre of the geophone array.

Data Processing and Interpretation

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

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

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases. Based on our testing results, the average overburden shear wave velocity is **165 m/s**, while the bedrock shear wave velocity is **2,664 m/s**.

For foundations placed directly or indirectly (i.e., using lean-concrete in-filled trenches) upon a clean, sounded bedrock surface, or entirely or partially on soil, the Vs_{30} was calculated using the standard equation for average shear wave velocity calculation provided in the Ontario Building Code 2024, and as presented below, where in this case Layer 1 represents the overburden and Layer 2 depicts the bedrock.

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

Based on the results of the shear wave velocity testing, the Site Designation for the proposed buildings is described below.

Seismic Site Designation for Buildings A and B

Based on the geotechnical investigation, the bedrock surface within the proposed footprint of Building A and Building B has an elevation ranging approximately between 96.0 to 84.0 m. It is expected that buildings A and B will be founded at an approximate elevation of 95.30 m. Based on the above, it is anticipated that the bedrock surface will be located within 11.3 m of the founding depth.

Using the standard equation for average shear wave velocity calculation, the average shear wave velocity testing Vs_{30} is 397 m/s for the proposed buildings at the subject site. Therefore, **a Site Designation X_{397}** would be applicable as per OBC 2024 for the proposed buildings with foundation elevation up to 11.3 m from bedrock surface.

It should be noted that the Seismic Site Designation should be reviewed by Paterson once the foundation design is available.

Seismic Site Designation for Building C

Based on the geotechnical investigation, the bedrock surface within the proposed footprint of Building C has an elevation ranging approximately between 93.0 to 88.0 m.

It is expected that the foundation elevation for building C will be approximately at 94.95 m. Based on the above, it is anticipated that the bedrock surface will be located within 7 m of the founding depth. Using the standard equation for average shear wave velocity calculation, the average shear wave velocity testing Vs_{30} is 588 m/s. Therefore, **a Site Designation X_{588}** is applicable as per OBC 2024 for the proposed building with foundation elevation up to 7 m from bedrock surface.

It should be noted that the Seismic Site Designation should be reviewed by Paterson once the foundation design is available. The soils underlying the subject site are not susceptible to liquefaction or cyclic softening.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the in-situ soil and/or bedrock surfaces will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. However, if storage or other uses of the lower level involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill should consist of OPSS Granular A. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab where a basement level is provided. The spacing of the sub-slab drainage pipes should be advised by Paterson during the design phase and once the footing and sump pit locations are known. The footprint would be confirmed at the time of construction once groundwater infiltration can be best assessed, if any. This is discussed further in Subsection 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 18 kN/m³. However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 11 kN/m³, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slab, which should be designed to accommodate these pressures. A hydrostatic pressure should be added for the portion below groundwater level.

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

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at 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 0.32 g according to the latest revision of the Ontario Building Code. 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 the OBC 2012 and 2024.

5.7 Pavement Design

Pavement Structure Over Overburden

For design purposes, the pavement structures presented in Tables 8, 9 and 10 below are recommended for the design of car only parking, local residential roadways, and rigid pavement. It should be understood the pavement structures provided in Tables 8, 9 and 10 are not intended for construction truck traffic without requiring additional measures to prepare the base and subbase layers for the placement of asphalt. This is discussed further in this subsection of the report.

Table 8 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 9 – Recommended Pavement Structure – Local Residential Roadways, Access Lanes and Heavy Truck Parking Areas

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

Table 10 – Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
Specified by Others	**32 MPa Concrete**
200	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. For residential driveways and car-only parking areas, an Ontario Traffic Category A will be used. For local roadways, an Ontario Traffic Category B should be used for design purposes.

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. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program and is discussed further in the following portion of this report.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment., noting that excessive compaction can result in subgrade softening.

Pavement Structure Over Bedrock

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) may be reduced to 300 mm for the above-noted pavement structures. The bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised during the bedrock removal program to ensure that the bedrock subgrade does not have depressions that will trap water, as this could impact the service life of the pavement structure.

Temporary Access Roads and Construction Traffic

Paterson anticipates that the earthworks contractor will require several haul roads, staging areas and other temporary access lanes to facilitate construction traffic. Paterson also anticipates construction traffic will be directed over unpaved access paths constructed using the base and subbase layers identified in the above-noted tables and will be used throughout the duration of the construction phase.

Omitting the asphalt layer, the above-noted pavement designs are not considered suitable to support temporary construction traffic without requiring additional measures to remediate the proposed base and subbase layers to accommodate the placement of asphalt to complete the pavement design.

Therefore, provisions should be carried to either reinstate temporary construction access and haul roads prior to placing asphalt or improve the durability of the temporary unpaved construction access and haul roads to minimize additional efforts for preparing the base course for the placement of asphalt once construction traffic would no longer be required.

Examples of scenarios that would require these provisions would consist of areas which construction traffic results in rutting and compromising subgrade soils, placement of subbase layers directly over subgrade shortly following periods of spring thaw, snowmelt and rainfall events or over service trenches that may consist of poorly compacted backfill.

For planning purposes, temporary construction haul roads and working pads should be planned to be 600 mm of crushed stone consisting of a 500 mm of a combination of OPSS Granular B Type I or Type II crushed stone and/or blast-rock covered with a minimum 50 to 100 mm thick layer of OPSS Granular B Type II or OPSS Granular A crushed stone (to provide suitable surface for vehicle tires) over a Paterson-reviewed and -approved subgrade.

These types of roads should also be underlain by a non-woven geotextile layer, such as Terraifx 200R, where they would be integrated into the final pavement structure and accommodate the placement of asphalt to minimize pumping of fines into the subbase layer. Cow-pathing site-generated soil may also be considered to provide suitable haul and access roads.

Temporary access roads that will not support heavy truck traffic (i.e., conventional light-duty vehicles only) may be prepared using a minimum of 150 mm of OPSS Granular A and 400 mm of OPSS Granular B Type II crushed stone.

However, provisions should be carried to provide a non-woven geotextile separation layer, such as Terrafix 200R, over the subgrade soils to lessen the amount of fines that migrate into the subbase layers in response to a combination of construction traffic and seasonal fluctuations in the subgrades performance. Provisions should also be carried to scarify and replace the upper 100 to 150 mm of these areas with clean OPSS Granular A crushed stone prior to placing asphalt.

Provisions should also be carried by the earthworks contractor to suitably compact trench backfill placed over services when reinstating servicing trenches below areas proposed to support paved areas. Since it is anticipated this material would consist of workable brown silty clay or silty sand fill (and not wet, non-workable grey silty clay) it would be recommended to place this material in maximum 400 mm thick loose lifts compacted using a suitably sized vibratory sheepsfoot roller making several passes under the supervision of Paterson field personnel.

The subgrade surface is also recommended to be provided with a layer of bi-axial geogrid, such as Terrafix TBX2500, to improve the stiffness of the reinstated trench backfill subgrade for supporting the final pavement structures.

These efforts would be reviewed, approved and advised upon by Paterson field staff during the construction program. Further, Paterson should review design, tender and construction documents associated with temporary and permanent pavement design throughout those phases of the project.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on 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.

Where silty clay is anticipated at the pavement subgrade level, consideration should be given to installing subdrains during the pavement construction. The subdrain inverts should be approximately 300 mm below subgrade level, and the subgrade surface should be crowned to promote water flow to the drainage lines.

Off-Site Road Cut Service Trench Pavement Design

It is understood off-site services within the Terry Fox Drive right-of-way will be extended to the subject site and require additional excavation of the existing roadway and associated reinstatement upon completion of the sewer installation.

Based on Paterson’s review of the Transportation Impact Assessment completed by Dillon (25-9532 dated October 2025), the subject sections of Terry Fox Drive is characterized as a major collector road and considered a Traffic Category D level of service (LOS) for transit.

Based on this, the following pavement design is recommended to be reinstated throughout the proposed roadcut:

Table 10 – Recommended Flexible Pavement Structure for Terry Fox Drive Reinstatement

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

Minimum Performance Graded (PG) 64-34 asphalt cement should be used for this purpose. Cement asphalt should be compacted to a minimum average density of 93% and no more than 98%. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

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.

All reinstatement efforts must be undertaken in accordance with the City of Ottawa’s *Standard Detail R10 – Standard Trench Reinstatement in Paved Surface* dated March 2023 and other pertinent details, specifications and requirements identified by the City of Ottawa.

All subgrade fill is recommended to be placed in 300 mm maximum thick loose lifts and compacted to a minimum of 95% of the materials SPMDD (or as otherwise advised by the City of Ottawa) and reviewed and approved by Paterson personnel at the time of construction.

All reinstatement efforts are recommended to be reviewed and approved by Paterson at the time of construction.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Perimeter Foundation Drainage System

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by a minimum of 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures where double-sided pours will be undertaken. In areas where blind-sided pours will be considered, the perimeter drainage pipe should be placed along the interior side of the foundation wall and connected to sleeves placed within the foundation wall at a 6 m center-to-center spacing. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

It is anticipated that underfloor drainage will be required to provide an outlet for water captured by the buildings drainage system since it is assumed external gravity outlets will not be able to be accommodated by the sewer design. The layout of the perimeter and underfloor drainage systems should be determined by Paterson during the design phase once the foundation structure and sump pit locations are known. The perimeter drainage pipe would connect to a series of underfloor drainage lines which would direct water to sump pit(s) within the lower basement area.

A positive-side (i.e., placed on exterior faces) waterproofing system should also be provided for any elevator shafts and pools located within the lowest basement level. A continuous PVC waterstop should be installed within the interface between the concrete base slab below the elevator shaft foundation walls. It is recommended that Paterson review and advise on all basement waterproofing/drainage system designs during the design phase.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls unless used in conjunction with a composite drainage system, such as CCW MiraDRAIN 2000 or Delta-Teraxx or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings 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.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized or current structural and architectural drawings for the proposed buildings to provide specific waterproofing and drainage design recommendations for design and tender. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

Perimeter footings, pile caps and grade beams of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the proper structure. These footings should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation Side Slopes

Unsupported Excavations

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.

Excavation side slopes above the groundwater level extending to a maximum vertical height of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

In sound bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors or other means determined by Paterson at the time of construction. A minimum of 1 m horizontal ledge should remain between the unsupported excavation and bedrock surface.

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 Paterson in order to detect if the slopes are exhibiting signs of distress.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes, or other means of erosion protection along their footprint. The tarps should be anchored with stakes embedded a minimum of 600 mm below existing grade at the top of the excavation and on a maximum spacing of 2 m centres.

Soil stockpiles, debris, and other forms of weight should not be considered for the purpose of securing the tarpaulins along the top of the slope. However, consideration may be given to restraining the tarpaulins with soil, sandbags, stone, etc. along the bottom of the side-slope. The tarpaulins should extend beyond the overburden and onto the bedrock surface.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that 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 design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the shoring wall extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur, and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 11.

Table 11 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ’), kN/m^3	13

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

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on grey clay or bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes.

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 crushed stone. The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 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 match the soils exposed at the trench walls to reduce the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

Bedrock/Soil Transitions

In areas where the service subgrade transitions from soil to bedrock, it is recommended that the founding medium be inspected in the field to determine how steeply the bedrock surface, where encountered, drops off. A transition treatment is generally recommended to be provided where the bedrock slopes steeper than 3H:1V. At these locations, the bedrock should be excavated, and a minimum 500 mm thick layer of bedding, such as OPSS Granular A crushed stone, be placed to provide a 3H:1V transition from the bedrock subgrade toward the soil subgrade. This treatment will reduce the propensity for bending stresses to occur in the service pipe alignments.

It is recommended that this condition be reviewed in the field by Paterson personnel at the time of excavation and construction of site services. Paterson field personnel may advise on appropriate treatments where pipe subgrade transition between soil and bedrock surfaces.

Glacial Till to Clay Deposit Transitions

In areas where site servicing trenches advance across transitions between relatively shallow deposits of glacial till (shallow relative to bedrock surface) and deeper deposits of clay, glacial till soils consisting of predominantly fine-grained fines matrixes and high in-situ moisture levels will be difficult to place bedding materials upon. It is expected these soils will be in a relatively loose state of compactness and be readily disturbed by vibrations induced by compaction equipment. It would be expected that satisfactory dewatering efforts would be undertaken ahead of the trenching works to ensure efforts may be undertaken in the dry.

It is recommended that provisions be carried to provide localized bedding layers that may exceed 150 to 300 mm (i.e., in the range of 500 mm to 1 m and potentially higher) to place the bedding material upon compact to dense glacial till soils that would underlie the shallower looser material. Thickened bedding layers would be recommended to consist of OPSS Granular B Type I or II crushed stone and/or suitably fragmented and -sized blast rock, if available. During the detailed design phase, Paterson will review all site servicing drawings to identify areas where the above-noted transition zone treatment would be expected to be considered.

Backfilling Within Trench Boxes

When the bedding and cover material is placed within the confines of a trench box and steel plates, it is recommended that the trench box be placed tightly against the outside of the trench walls and remains approximately 300 mm above the obvert level of the service pipe.

The vertical excavation sidewalls within the lower portion of the trench (below the obvert level of the pipe) can be supported using steel plates extended down to the bottom of the trench. The steel plates can be extended below the base of the excavation to prevent basal heave, in conjunction with adequate dewatering measures when located below the groundwater table.

To minimize the potential for disturbance of the bedding and cover material and subsequent settlement of the service pipe during the removal of the steel plates, it is recommended that the bedding layer be re-compacted tightly against the trench sidewalls upon removal/lifting of the steel plate up to the top of the bedding layer and prior to placing the pipe. This is recommended to mitigate settlement of the pipe that would results from removing the plates without re-compacting the fill that would be left unconfined to the sides of the trench. This procedure would be repeated for the springling and cover layers until the steel plates are removed.

It is generally recommended that this procedure be reviewed by Paterson field personnel at the time of construction.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that surface water infiltration into the excavations undertaken above the groundwater table should be manageable through the sides of the excavation and controllable using open sumps. It is further anticipated that groundwater infiltration into the excavations may be moderate to high throughout the overburden located below the groundwater table and/or bedrock surface.

Further, bedrock removal can lead to increased fracturing (hydraulic pathways), resulting in highly variable groundwater conditions. 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.

It is recommended that Paterson review detailed design drawings and construction timelines associated with excavation works prior to tendering the earthworks portion of the project. It is recommended that Paterson review and advise at that time if additional recommendations are required with regards to planning temporary dewatering and groundwater management efforts during the construction phase.

Permit to Take Water

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

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

Impact to Neighboring Properties

Based on the proposed grading and design details, the proposed structures will be founded within very stiff, brown silty clay and sound bedrock layers and above layers sensitive to long-term dewatering, such as the grey silty clay layer.

Since the structures will be founded within subsoils that have allow hydraulic conductivity, the zone of influence of dewatering by the proposed structures is limited to the area directly beyond the proposed structures and within the boundaries of the subject site.

Based on this, there are no negative impacts due to long-term dewatering that are anticipated to occur to neighboring structures and infrastructure as a result of the construction of the proposed structures from a geotechnical perspective.

6.6 Winter Construction

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

Where excavations are completed in proximity to existing structures which may be adversely affected due to the freezing conditions. The subsurface conditions mostly 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 particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract documents to protect the walls of the excavations from freezing, if and where applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means. 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 foundation is protected with sufficient soil cover to prevent freezing at founding level.

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

Under winter conditions, if snow and ice are present within imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

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 pH of the sample indicates that it is not a significant factor in creating a corrosive environment for exposed ferrous metals at this site, whereas the chloride content and resistivity is indicative of an aggressive corrosive environment.

6.8 Landscaping Considerations

Retaining Walls and Garage Ramp Foundation Walls

It is understood that a retaining wall is expected to be constructed along the eastern property boundary of the subject site as part of the proposed development, as well as entrance ramp foundation walls to the building garages.

It should be noted that proposed retaining walls should be designed by a Licensed Professional Engineer in the Province of Ontario and should be subject to a conforming global stability analysis. The foundation walls will be designed by a licensed structural engineer and should be designed using the earth pressure parameters provided in the following sections.

All sections of the retaining wall(s) should be designed so that their internal and external failure modes comply with CHBD requirements. Furthermore, any proposed retaining wall should be designed to maintain an adequate factor of safety greater than 1.5 under static loading conditions and greater than 1.1 under seismic loading conditions.

The applicable seismic design should incorporate Peak Ground Acceleration (PGA) for the Ottawa area as per the latest Ontario Building Code.

It is also required that the bearing medium of the proposed wall be reviewed by Paterson field personnel at the time of excavation and prior to placement of the granular bedding layer. Based on the results of the geotechnical investigation, it is anticipated that the walls will be founded over an engineered fill pad or undisturbed, in-situ soil bearing surfaces.

The soil parameters presented in Table 12 can be used in the design of the retaining walls.

Table 12 – Soil Parameters for Global Stability Analysis

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (°)	Effective Cohesion (kPa)	Total Cohesion (kPa)
Brown Silty Clay	17	33	5	80
Grey Silty Clay	16	33	10	40
Glacial Till	19	35	0	0

It is recommended that a 100 mm diameter perforated corrugated plastic pipe with geosock, surrounded by 150 mm of 19 mm clear crushed stone on all sides, be placed behind the heel of the wall. The pipe should have a positive outlet, either in front of, below, or to the side of the wall, towards a natural slope or drainage system.

Backfill Materials

Retaining walls should be backfilled with free-draining granular material, as Granular A or Granular B Type II materials. Longitudinal drains and outlets should also be incorporated to ensure proper drainage of the backfill material.

It is further recommended that backfill material be placed within a wedge-shaped area defined by a line drawn from below the rear edge of the wall's base block at a slope of 1H:1V, or a minimum of 1 m behind the rear of the blocks. All material must be compacted to a minimum of 98% of the materials SPMDD.

Geotechnical parameters of the proposed free draining backfill material to be used at the subject site are provided in Table 13 for design purposes.

Table 13 – Geotechnical Parameters for Backfill Material

Material Description	**Unit Weight (kN/m^3)**		**Friction Angle (°) φ'**	**Friction Factor, tan δ**	**Lateral Earth Pressure Coefficients**		
	Drained γ_{dry}	**Effective γ'**			**Active K_a**	**At Rest K_o**	**Passive K_P**
Granular A (Crushed Stone)	22	13.5	38	0.6	0.24	0.38	4.20
Granular B Type II (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.60

Lateral Earth Pressure

It is recommended that a minimum of 1 m of backfill material consisting of clean, imported crushed stone as Granular A or Granular B Type II. The soil parameters shown in Table 12 and Table 13 above should be used for retaining wall design.

Tree Planting Considerations

There are no specific tree planting setbacks applicable to proposed retaining walls. However, the retaining wall designer should consider both the additional loads applied from trees in close proximity to retaining walls, as well as the implementation of root barriers to protect the retaining wall systems to resist pressures exerted by root systems.

Trees located in front of the wall may be accommodated by thickened and reinforced bedding layers where clay soils may be present at the founding depth of the proposed retaining wall bedding layer and as deemed suitable by the designer. Paterson may assist with this portion during the design as required.

Global Stability Verification

Paterson has reviewed the currently proposed retaining wall footprint (Grading Plan Drawings - Revision 2 dated October 28, 2025, prepared by D.B. Gray Engineering Inc.) from a global stability perspective. The analysis of the stability of the slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method.

The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

Subsoil conditions at the cross-sections were inferred based on the boreholes completed throughout the subject site and general knowledge of the geology of the area. For a conservative review of the groundwater conditions, the silty clay deposit was considered to be fully saturated for our analysis. The effective and total strength soil parameters used for static and seismic analyses, respectively, are presented in Table 12.

The results are shown in Figures 1A and 1B in Appendix 2. The results indicate a slope with factors of safety exceeding 1.5 and 1.1 for static and seismic loading conditions, respectively, for the highest proposed segment of the proposed retaining wall structure. *Figure 4 – Typical Retaining Wall Cross Section* is provided in Appendix 2 of this report for further information summarizing the anticipated construction of the proposed retaining wall structure.

These results indicate the proposed retaining wall structure will be able to designed and constructed feasibly from a geotechnical perspective. It should be understood this assessment and associated typical detail are not considered suitable for construction or equivalent to a detailed design and are only for discussion purposes. The proposed retaining wall structure will be required to be designed as indicated in the above-noted sections of this Geotechnical Report at a later stage of design.

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 results of our testing are presented in Table 1 in Subsection 4.2.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to not exceed 40% for all the tested clay samples. Based on this, 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 the low to medium sensitivity silty clay deposit 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.

- ☐ 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 should be noted that these requirements are not applicable to buildings founded upon bedrock, glacial till or deep foundations extending to the bedrock surface.

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.

6.9 Low-Impact Development Considerations

Due to the poor hydraulic properties of in-situ silty clay soils and generally shallow bedrock topography, infiltration-type LID practices are not considered suitable for the subject site.

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, landscaping and structural plan(s) from a geotechnical perspective.

☐ Review Seismic Site Designation from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.

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:

☐ Observation of all waterproofing membranes, sub-slab drainage system and all associated systems and assemblies.

☐ 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 Ironclad Developments 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.

Nicholas F. R. Versolato, CPI, B.Eng.

Drew Petahtegoose, P.Eng.

LICENSED PROFESSIONAL ENGINEER
October 29, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Report Distribution:

☐ Ironclad Developments Inc. (Email Copy)
☐ Paterson Group (1 Copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ROCK CORE PHOTOGRAPHS

ATTERBERG LIMIT TESTING RESULTS

COMPRESSIVE STRENGTH TESTING RESULTS

GRAIN SIZE TESTING RESULTS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349355.88 **NORTHING:** 5019247.87 **ELEVATION:** 96.43

PROJECT: Proposed Residential Development

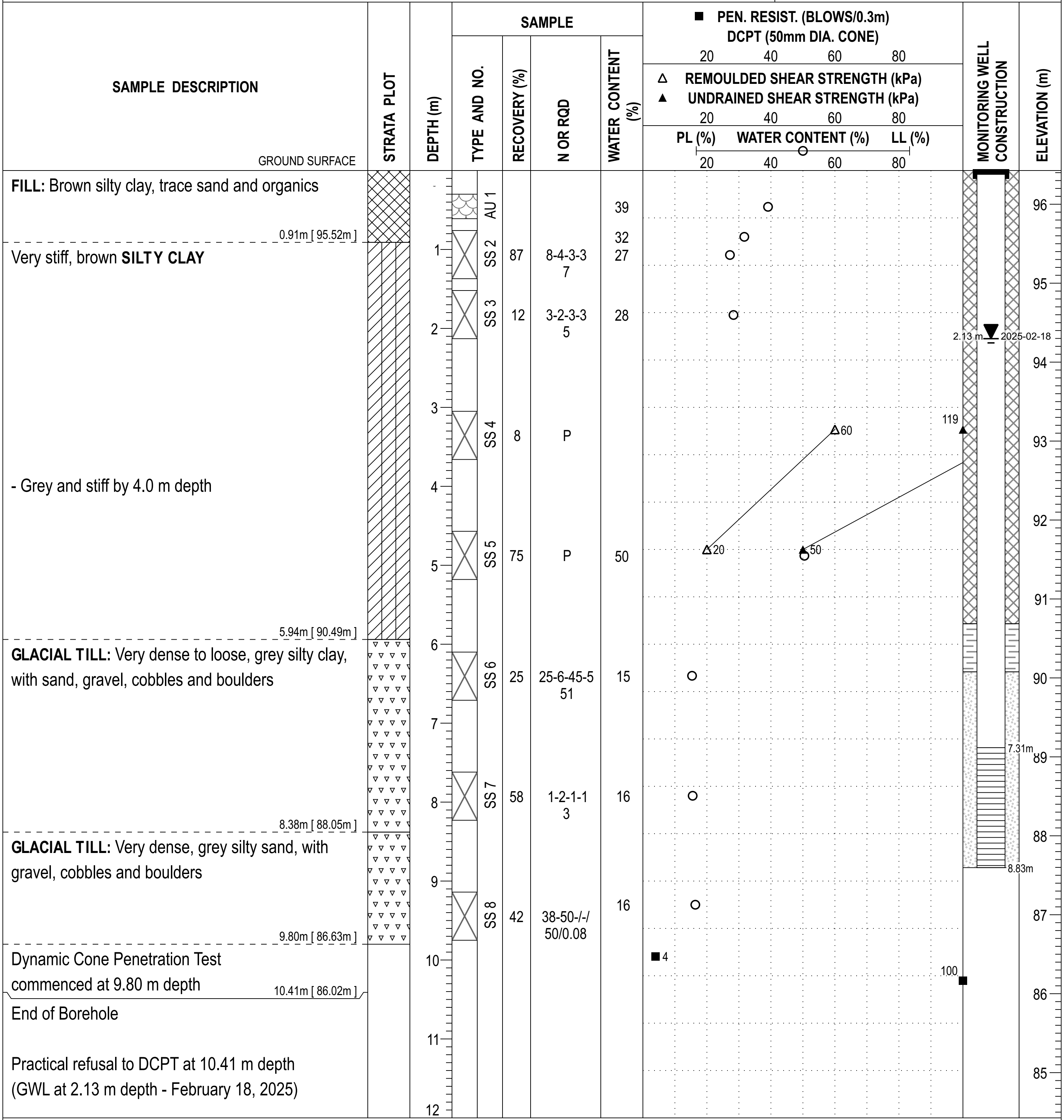
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 6, 2025

FILE NO. : PG7418

HOLE NO. : BH 1-25



SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty clay, trace sand and organics

0.91m [95.52m]

Very stiff, brown **SILTY CLAY**

- Grey and stiff by 4.0 m depth

5.94m [90.49m]

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

8.38m [88.05m]

GLACIAL TILL: Very dense, grey silty sand, with gravel, cobbles and boulders

9.80m [86.63m]

Dynamic Cone Penetration Test commenced at 9.80 m depth

10.41m [86.02m]

End of Borehole

Practical refusal to DCPT at 10.41 m depth

(GWL at 2.13 m depth - February 18, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			39
SS 2	87	8-4-3-3 7	32 27
SS 3	12	3-2-3-3 5	28
SS 4	8	P	
SS 5	75	P	50
SS 6	25	25-6-45-5 51	15
SS 7	58	1-2-1-1 3	16
SS 8	42	38-50-/-/ 50/0.08	16

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349394.11 **NORTHING:** 5019220.87 **ELEVATION:** 96.35

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 6, 2025

FILE NO. : **PG7418**

HOLE NO. : **BH 2-25**

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty clay, trace sand and organics

0.69m [95.66m]

Brown **SILTY SAND**, trace clay

1.07m [95.28m]

Very stiff, brown **SILTY CLAY**

2.84m [93.51m]

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

4.62m [91.73m]

BEDROCK: Good to fair quality granite, with interbedded granodiorite

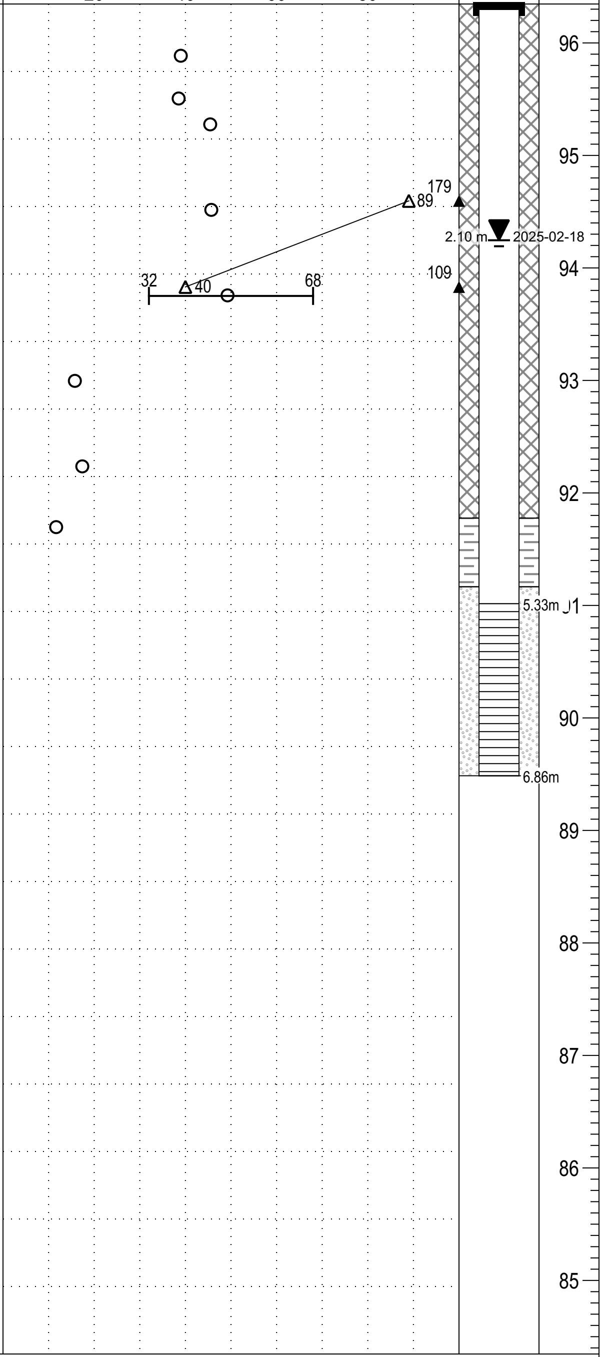
6.86m [89.49m]

End of Borehole

(GWL at 2.10 m depth - February 18, 2025)

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			39
SS 2	46	3-2-2-2 4	39 45
SS 3	75	P	46
SS 4	92	P	49
SS 5	67	2-4-4-50 8	16
SS 6	17	50-/-/-/ 50/0.08	17
SS 7	100	50-/-/-/ 50/0.05	12
RC 1	100	RQD 83	
RC 2	100	RQD 58	

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349363.18 **NORTHING:** 5019236.08 **ELEVATION:** 96.27

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 7, 2025

FILE NO. : PG7418

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION

GROUND SURFACE

Not Sampled

1.52m [94.75m]

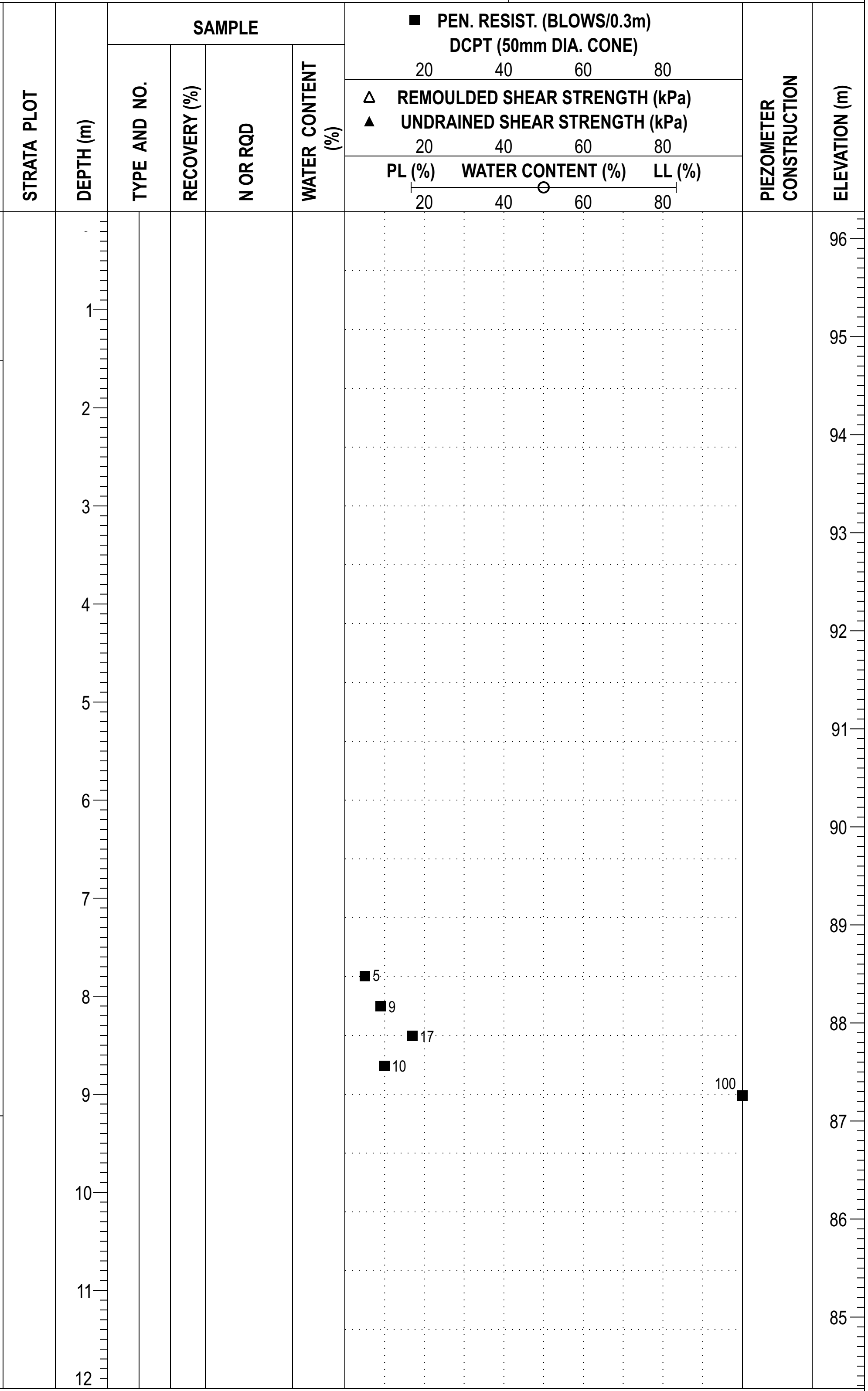
Dynamic Cone Penetration Test commenced at 1.52 m depth

9.22m [87.05m]

End of Borehole

DCPT Pushed from 1.52 m to 9.22 m depth

Practical refusal to DCPT at 9.22 m depth



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

P:/Autocad Drawings/Test Hole Data Files/PG74xx/PG7418/data.sqlite 2025-03-19, 11:53 Paterson_Template MA

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349426.83 **NORTHING:** 5019178.75 **ELEVATION:** 95.78

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 7, 2025

FILE NO. : PG7418

HOLE NO. : BH 4-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty clay, some sand, trace gravel and organics — 0.69m [95.09m]

Stiff to firm, brown **SILTY CLAY**

- Grey by 3.1 m depth

4.11m [91.67m]

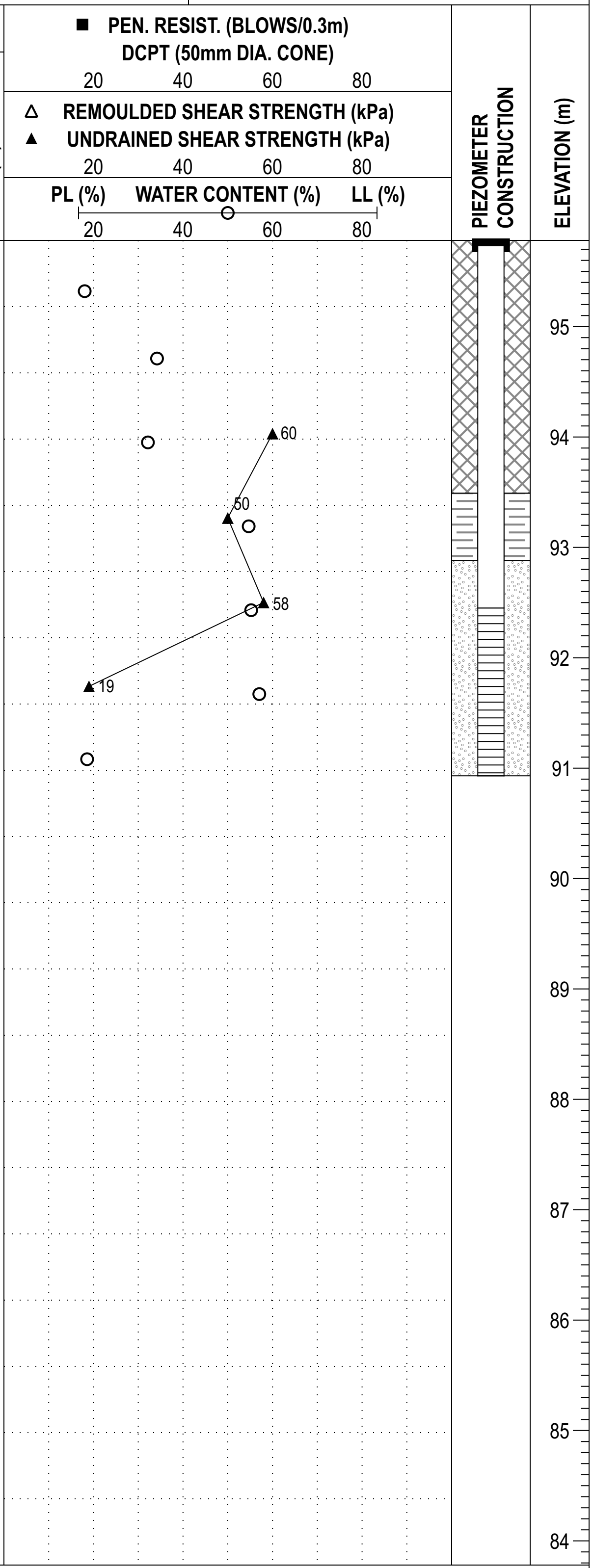
GLACIAL TILL: Grey silty sand, some clay, gravel, cobbles and boulders — 4.85m [90.93m]

End of Borehole

Practical refusal to augering at 4.85 m depth

(Piezometer blocked - February 18, 2025)

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			18
SS 2	92	3-1-2-2 3	34
SS 3	42	P	32
SS 4	100	P	55
SS 5	83	P	55
SS 6	92	P	57
SS 7	54	22-50-/-/ 50/0.1	19



DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349463.69 **NORTHING:** 5019155.73 **ELEVATION:** 97.17

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

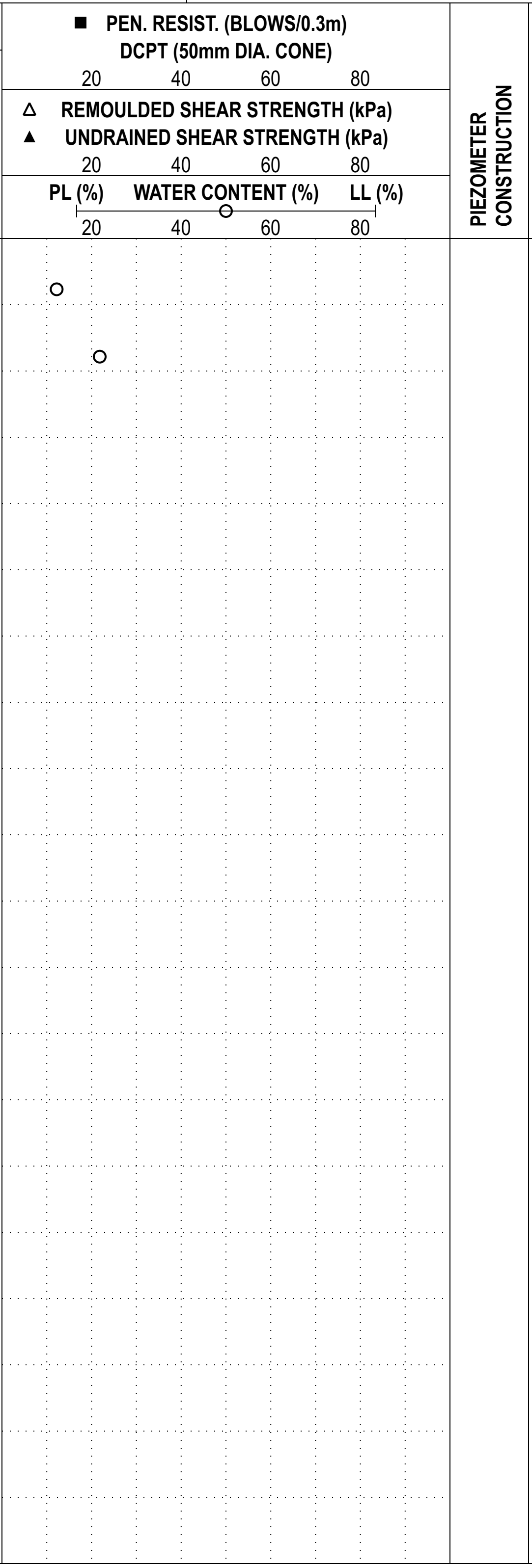
REMARKS:

DATE: February 7, 2025

FILE NO. : PG7418

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	
GROUND SURFACE		0					97.17
FILL: Brown silty clay, with sand, gravel and organics	▨		AU 1			12	97
	▨	1	SS 2	58	4-4-7-50 11	22	96
1.37m [95.80m]							
End of Borehole Practical refusal to augering at 1.37 m depth on inferred boulder		2					95
		3					94
		4					93
		5					92
		6					91
		7					90
		8					89
		9					88
		10					87
		11					86
		12					



P:/Autocad Drawings/Test Hole Data Files/PG74xx/PG7418/data.sqlite 2025-03-19, 11:53 Paterson_Template MA

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349461.57 **NORTHING:** 5019156.11 **ELEVATION:** 97.17

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 7, 2025

FILE NO. : PG7418

HOLE NO. : BH 5A-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty clay, with sand, gravel and organics

2.21m [94.96m]

Very stiff to stiff, brown **SILTY CLAY**

- Grey and firm by 5.3 m depth

8.31m [88.86m]

GLACIAL TILL: Grey silty clay, some sand, gravel, cobbles and boulders

9.02m [88.15m]

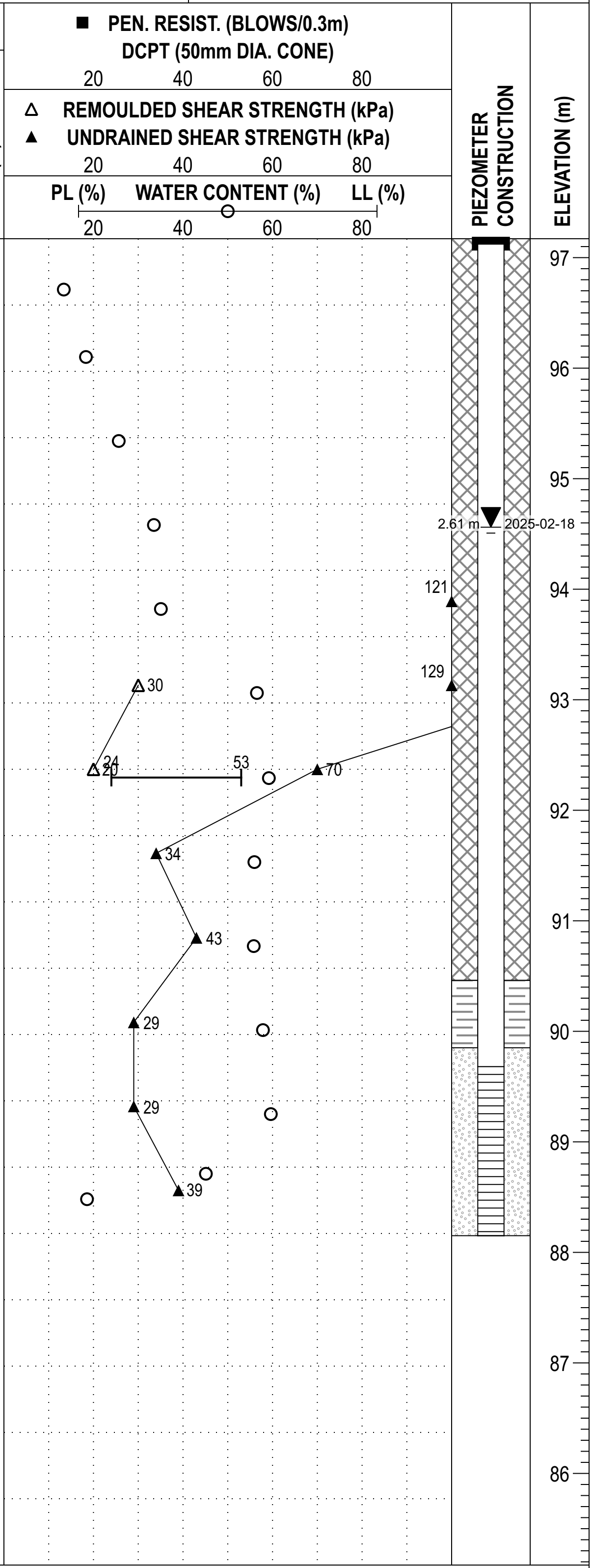
End of Borehole

Practical refusal to augering at 9.02 m depth

(GWL at 2.61 m depth - February 18, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			13
SS 2	8	P	18
SS 3		1-1-2-3 3	26
SS 4		3-2-1-2 3	34
SS 5	92	1-1-1-1 2	35
SS 6	100	P	57
SS 7	100	P	59
SS 8	100	P	56
SS 9	100	P	56
SS 10	100	P	58
SS 11	100	P	60
SS 12	83	P	45 19

DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12



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

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349398.84 **NORTHING:** 5019316.70 **ELEVATION:** 101.50

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

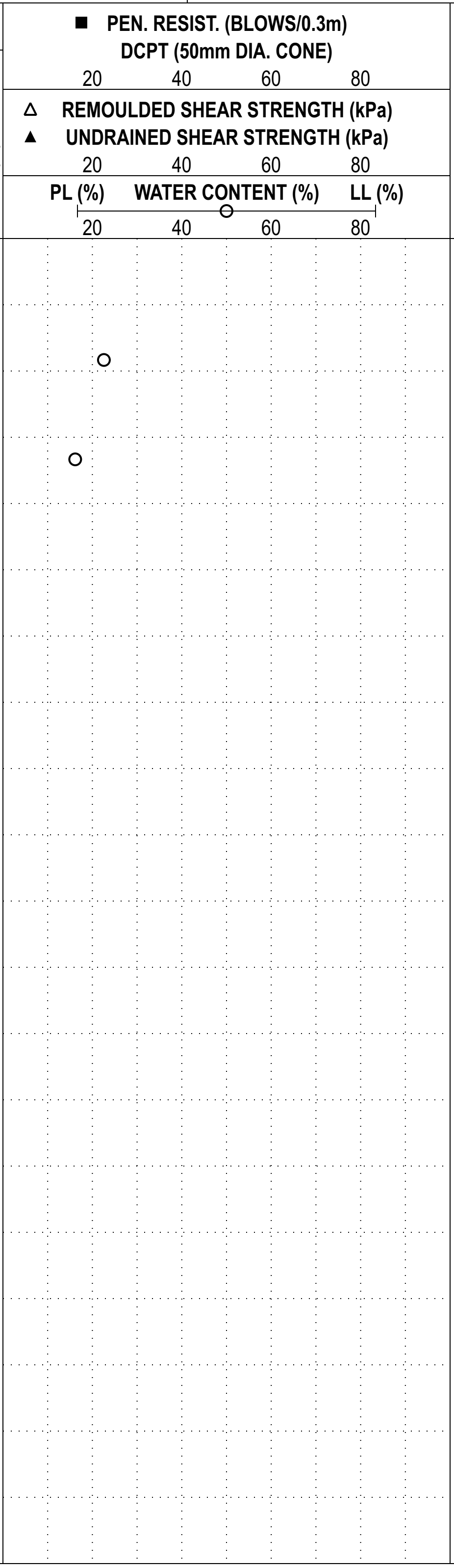
REMARKS:

DATE: February 5, 2025

FILE NO. : PG7418

HOLE NO. : TP 1-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE								
TOPSOIL 0.10m [101.40m]								
FILL: Brown silty clay, with sand and organics 0.50m [101.00m]								101
GLACIAL TILL: Brown silty clay, with sand, gravel, cobbles and boulders			G 1			23		
1.10m [100.40m]		1	G 2			16		
End of Test Pit Practical refusal to excavation at 1.10 m depth Test pit dry upon completion of excavation		2 3 4 5 6						100 99 98 97 96



P:/Autocad Drawings/Test Hole Data Files/PG74xx/PG7418/data.sqlite 2025-03-19, 12:17 Paterson_Template MA

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349373.44 **NORTHING:** 5019286.50 **ELEVATION:** 98.59

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

FILE NO. : **PG7418**

HOLE NO. : **TP 2-25**

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL

0.30m [98.29m]

Very stiff to stiff, brown **SILTY CLAY**

3.00m [95.59m]

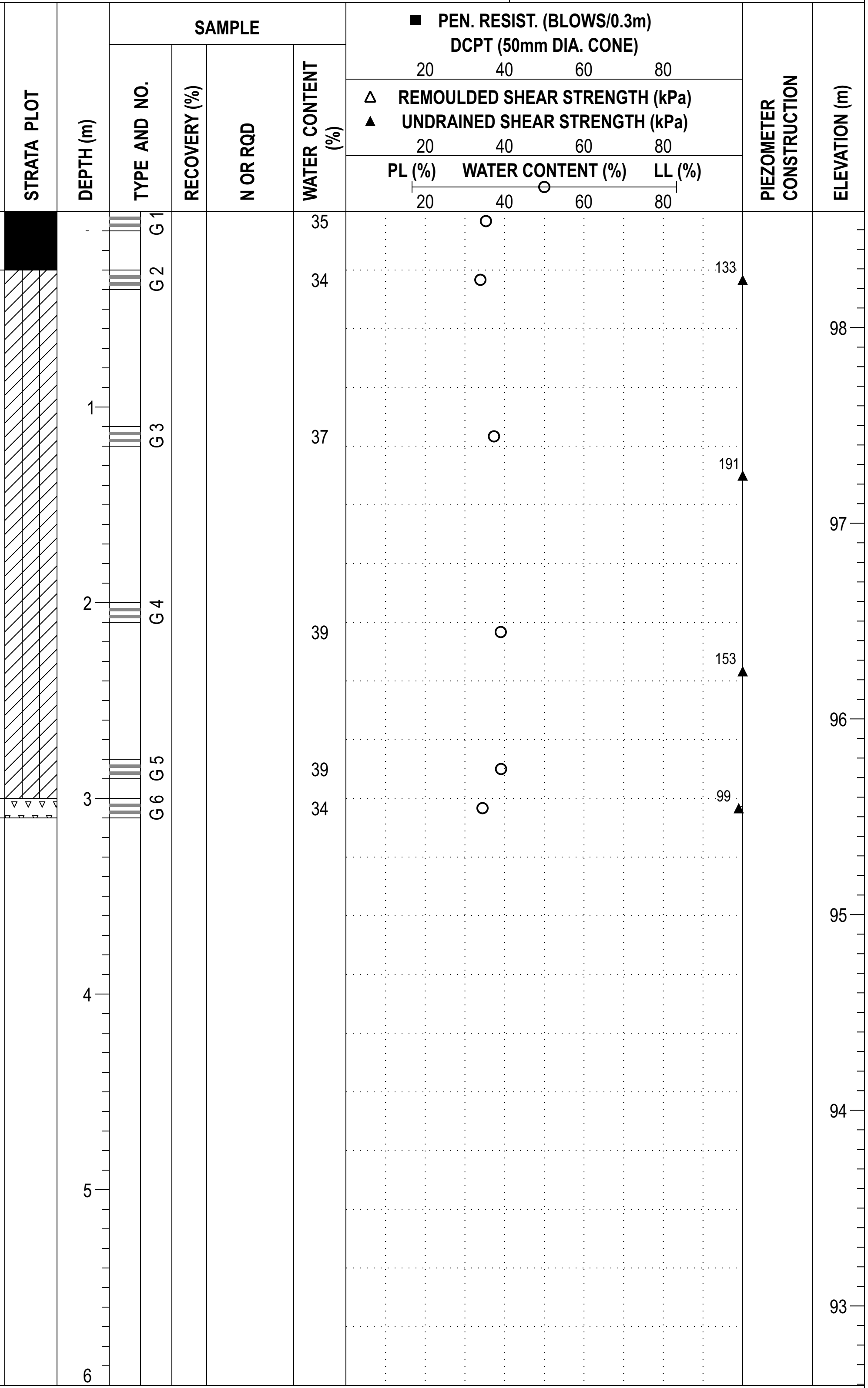
GLACIAL TILL: Brown silty clay, with gravel, cobbles and boulders, some sand

3.10m [95.49m]

End of Test Pit

Practical refusal to excavation at 3.10 m depth

Test pit dry upon completion of excavation



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349402.73 **NORTHING:** 5019288.48 **ELEVATION:** 100.19

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

FILE NO. : **PG7418**

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

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.10m [100.09m]

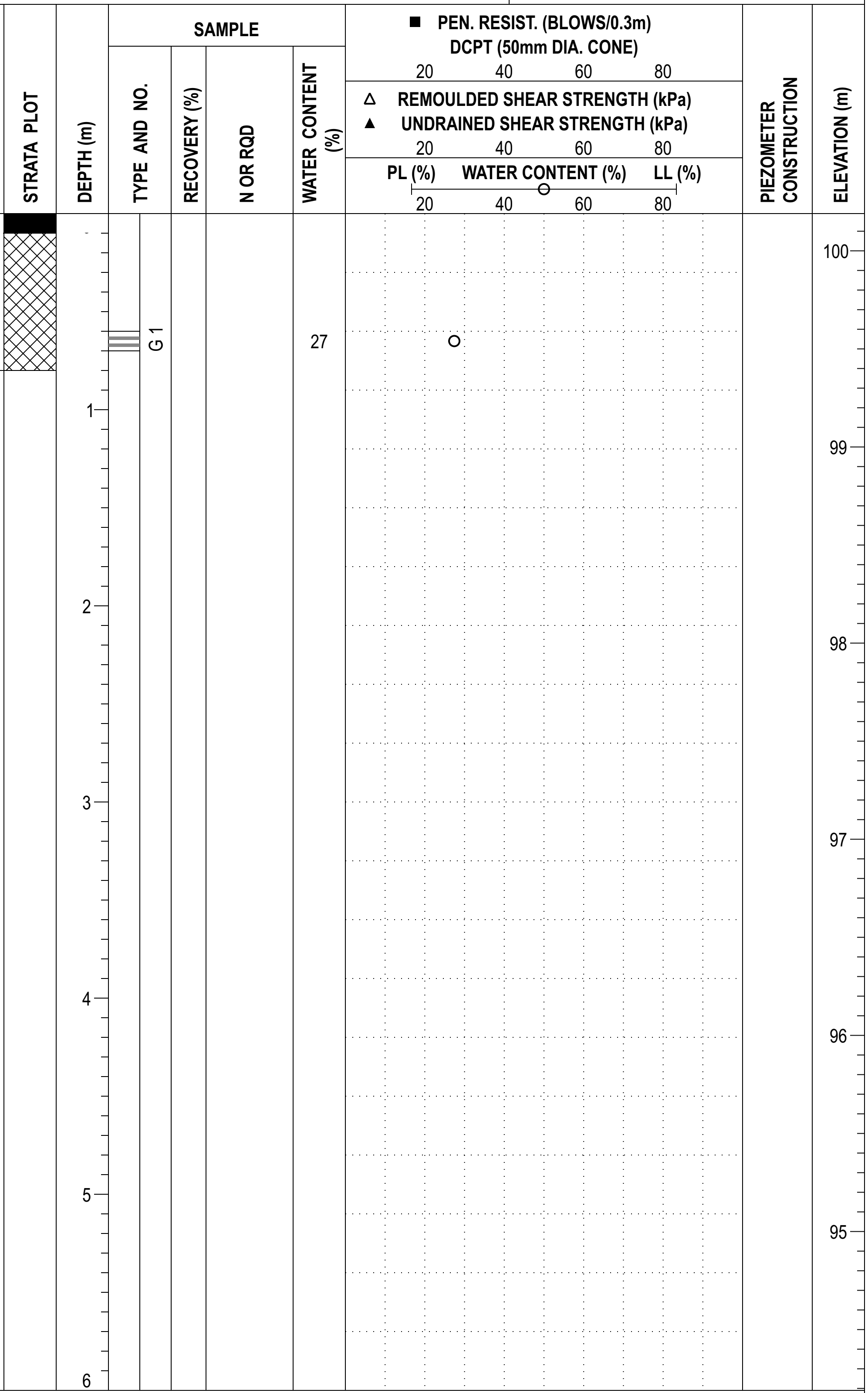
FILL: Brown silty sand, some construction debris and organics, trace clay

0.80m [99.39m]

End of Test Pit

Practical refusal to excavation at 0.80 m depth

Test pit dry upon completion of excavation



P:/Autocad Drawings/Test Hole Data Files/PG74xx/PG7418/data.sqlite 2025-03-19, 12:17 Paterson_Template MA

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349382.02 **NORTHING:** 5019258.92 **ELEVATION:** 97.63

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

FILE NO. : PG7418

HOLE NO. : TP 4-25

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.10m [97.53m]

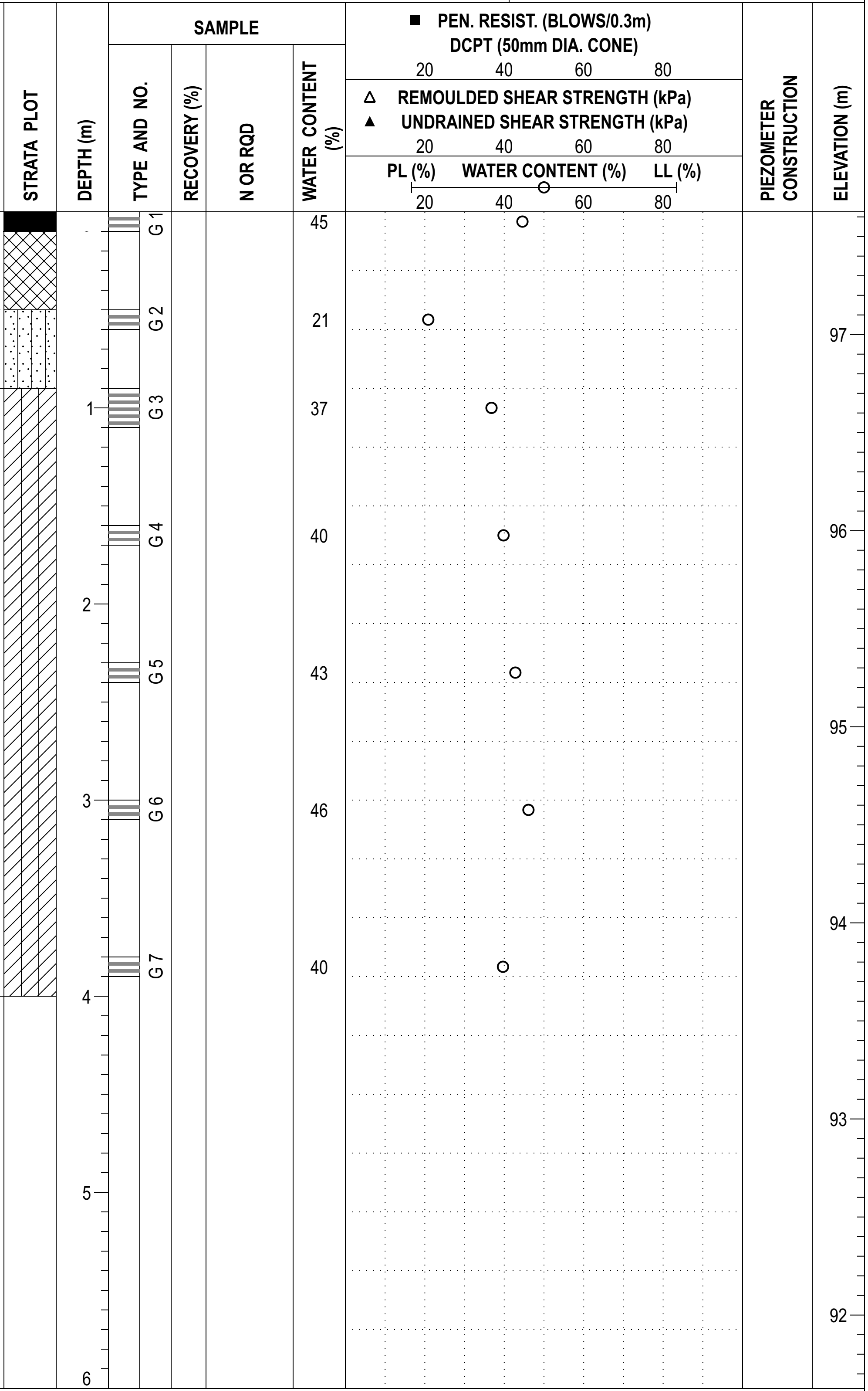
FILL: Brown silty clay, with sand, gravel and organics 0.50m [97.13m]

Compact, brown **SANDY SILT**, some organics, trace clay 0.90m [96.73m]

Brown **SILTY CLAY** 4.00m [93.63m]

End of Test Pit

Test pit dry upon completion of excavation



TYPE AND NO.	WATER CONTENT (%)
G 1	45
G 2	21
G 3	37
G 4	40
G 5	43
G 6	46
G 7	40

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349409.14 **NORTHING:** 5019258.59 **ELEVATION:** 98.58

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

FILE NO. : PG7418

HOLE NO. : TP 5-25

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.10m [98.48m]

FILL: Brown silty clay, with sand and organics 0.60m [97.98m]

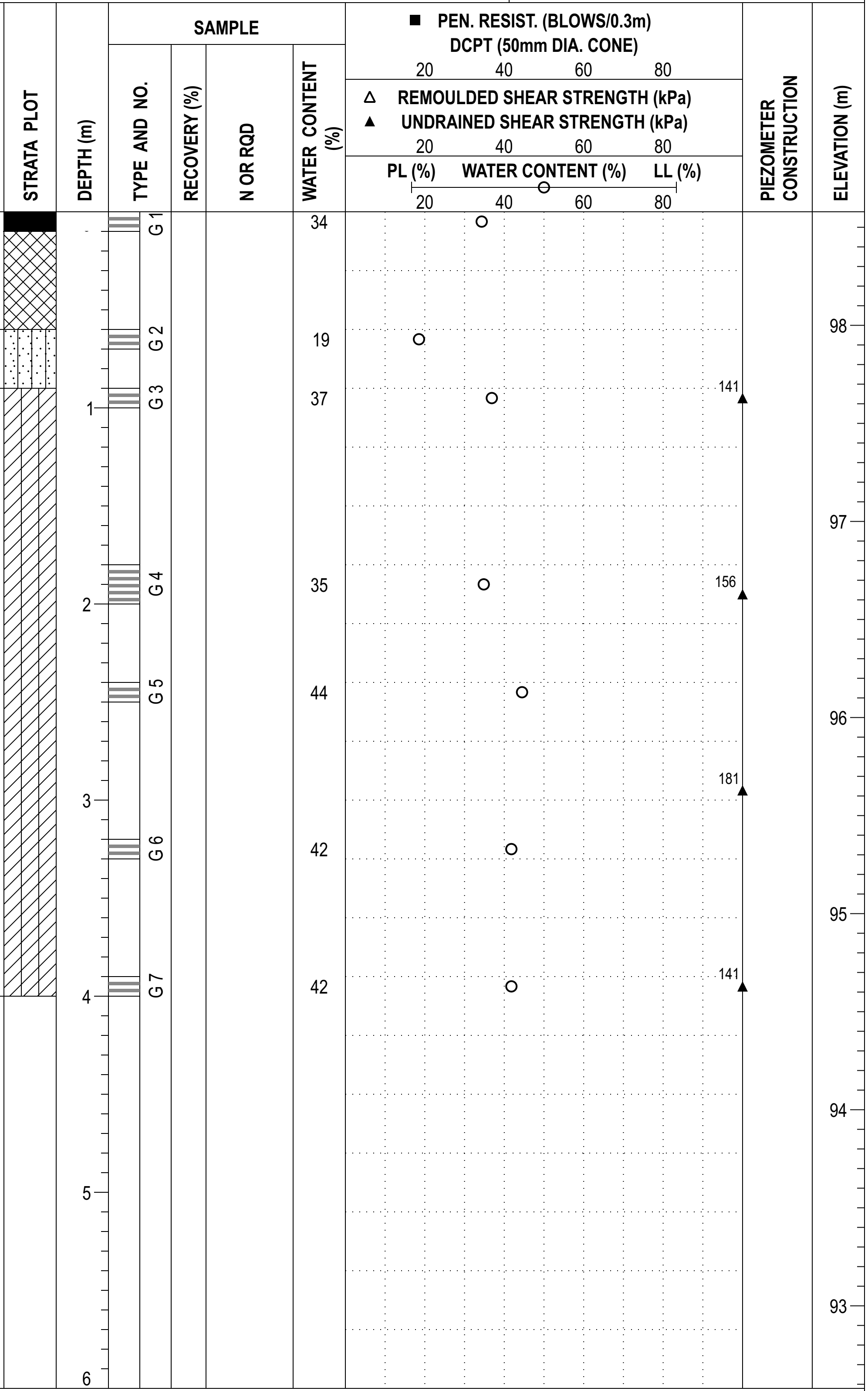
Compact, brown **SANDY SILT** 0.90m [97.68m]

Very stiff, brown **SILTY CLAY** 4.00m [94.58m]

End of Test Pit

Test pit dry upon completion of excavation

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
G 1			34
G 2			19
G 3			37
G 4			35
G 5			44
G 6			42
G 7			42



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349429.88 **NORTHING:** 5019227.88 **ELEVATION:** 98.53

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

FILE NO. : PG7418

HOLE NO. : TP 6-25

SAMPLE DESCRIPTION GROUND SURFACE	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	 RECOVERY (%)	 N OR RQD	 WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
TOPSOIL 0.20m [98.33m]	■	-	G 1			19	○		
End of Test Pit Practical refusal to excavation at 0.20 m depth Test pit dry upon completion of excavation									
		1							98
		2							97
		3							96
		4							95
		5							94
		6							93

P:/Autocad Drawings/Test Hole Data Files/PG74xx/PG7418/data.sqlite 2025-03-19, 12:17 Paterson_Template MA

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349377.83 **NORTHING:** 5019237.65 **ELEVATION:** 96.98

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

FILE NO. : PG7418

HOLE NO. : TP 7-25

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.10m [96.88m]

FILL: Brown silty sand, with organics, trace clay 0.60m [96.38m]

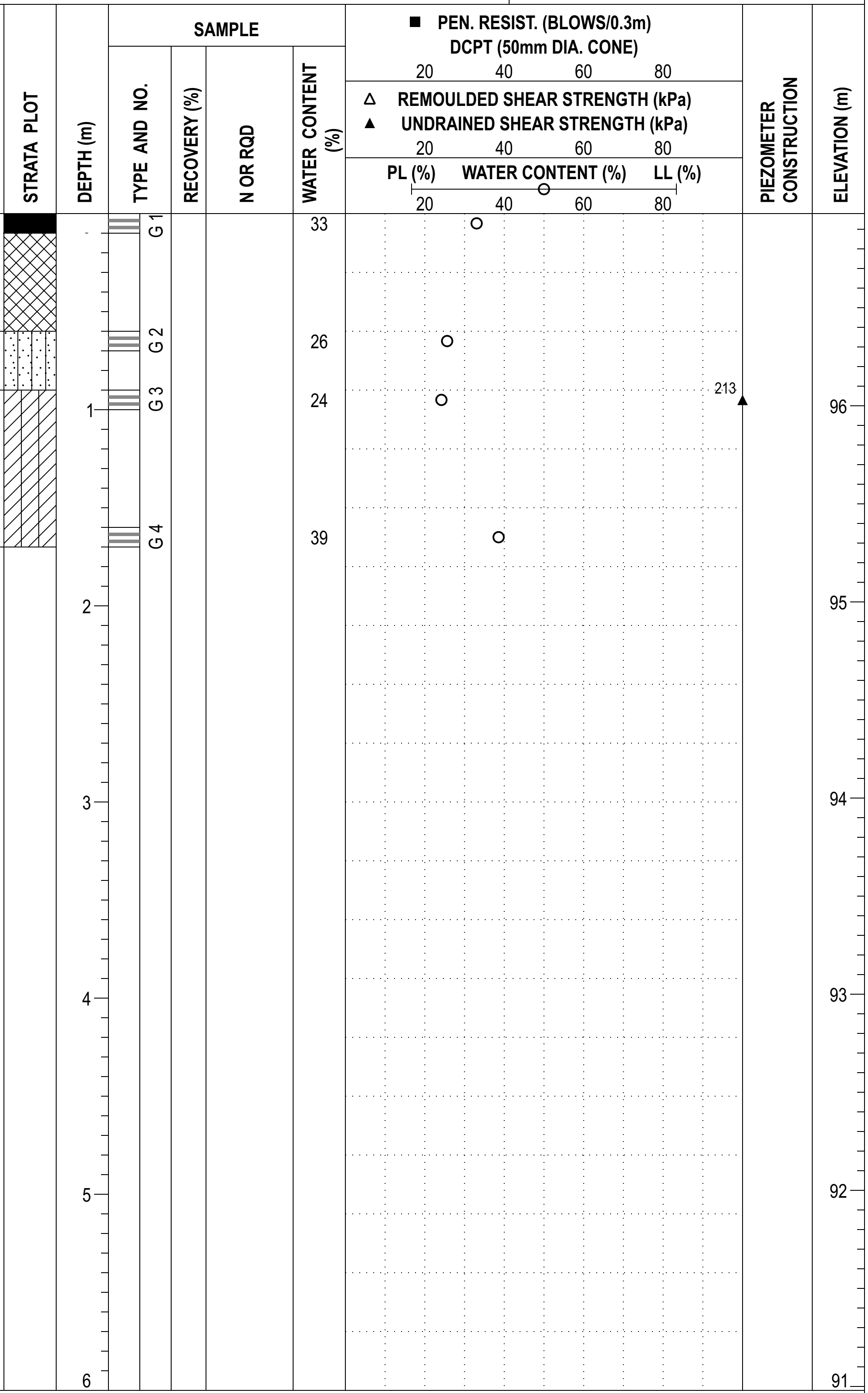
Compact, brown **SANDY SILT** 0.90m [96.08m]

Hard, brown **SILTY CLAY** 1.70m [95.28m]

End of Test Pit

Practical refusal to excavation at 1.70 m depth

Test pit dry upon completion of excavation



TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
G 1			33
G 2			26
G 3			24
G 4			39

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349446.52 **NORTHING:** 5019189.17 **ELEVATION:** 97.29

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

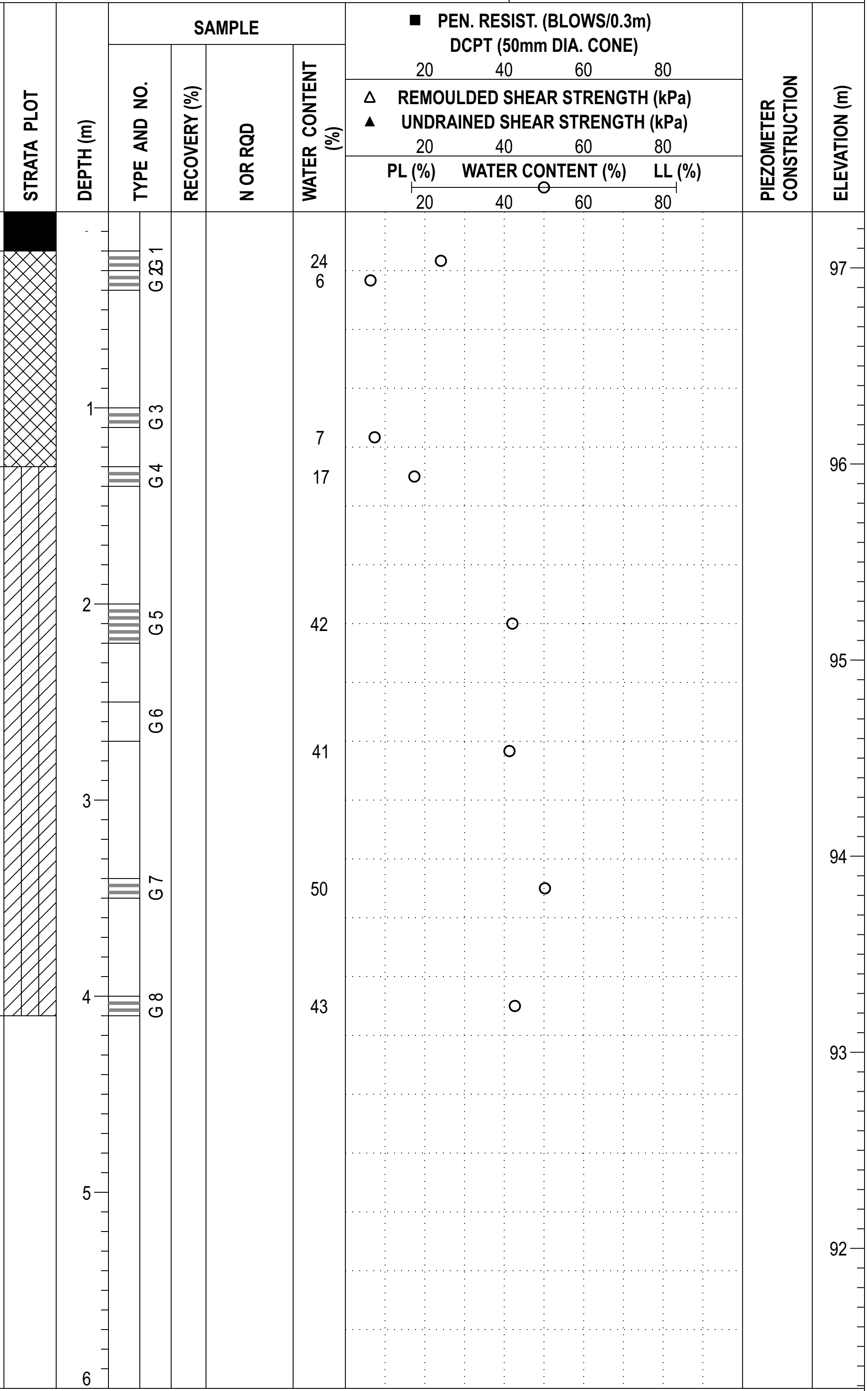
FILE NO. : PG7418

HOLE NO. : TP 8-25

SAMPLE DESCRIPTION	Depth / Elevation
GROUND SURFACE	
TOPSOIL	0.20m [97.09m]
FILL: Brown silty sand, with clay, trace gravel and organics	1.30m [95.99m]
Brown **SILTY CLAY**	4.10m [93.19m]

End of Test Pit

Groundwater infiltration observed at 3.0 m depth



TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
G 1			24
G 2			6
G 3			7
G 4			17
G 5			42
G 6			41
G 7			50
G 8			43

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349395.83 **NORTHING:** 5019336.46 **ELEVATION:** 101.53

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

FILE NO. : PG7418

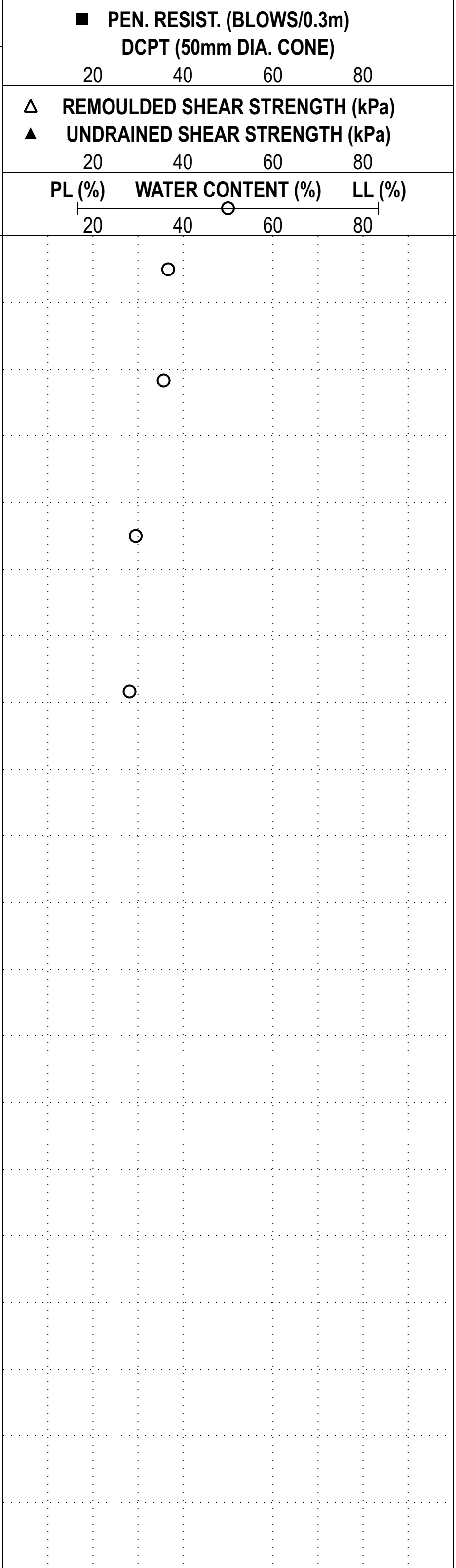
HOLE NO. : TP 9-25

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE					
TOPSOIL 0.10m [101.44m]		G 1			37
FILL: Brown silty clay, trace sand and organics 0.60m [100.94m]		G 2			36
Brown **SILTY CLAY** 2.00m [99.53m]	1	G 3			30
GLACIAL TILL: Brown silty clay, with sand, gravel, cobbles and boulders 2.30m [99.23m]	2	G 4			28

End of Test Pit

Practical refusal to excavation at 2.30 m depth

Test pit dry upon completion of excavation



PIEZOMETER CONSTRUCTION

DEPTH (m)	ELEVATION (m)
1	101
2	100
3	99
4	98
5	97
6	96

P:/Autocad Drawings/Test Hole Data Files/PG74xx/PG7418/data.sqlite 2025-03-19, 12:17 Paterson_Template MA

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349397.24 **NORTHING:** 5019351.95 **ELEVATION:** 101.99

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 5, 2025

FILE NO. : **PG7418**

HOLE NO. : **TP10-25**

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
TOPSOIL 0.15m [101.84m]	■	-							
End of Test Pit Practical refusal to excavation at 0.15 m depth Test pit dry upon completion of excavation									
		1							101
		2							100
		3							99
		4							98
		5							97
		6							96

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

475 Terry Fox Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 349466.33 **NORTHING:** 5019155.42 **ELEVATION:** 97.30

PROJECT: Proposed Residential Development

ADVANCED BY: Excavator

REMARKS:

DATE: February 12, 2025

FILE NO. : PG7418

HOLE NO. : TP11-25

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL

0.10m [97.20m]

FILL: Brown silty clay, with sand, gravel and organics

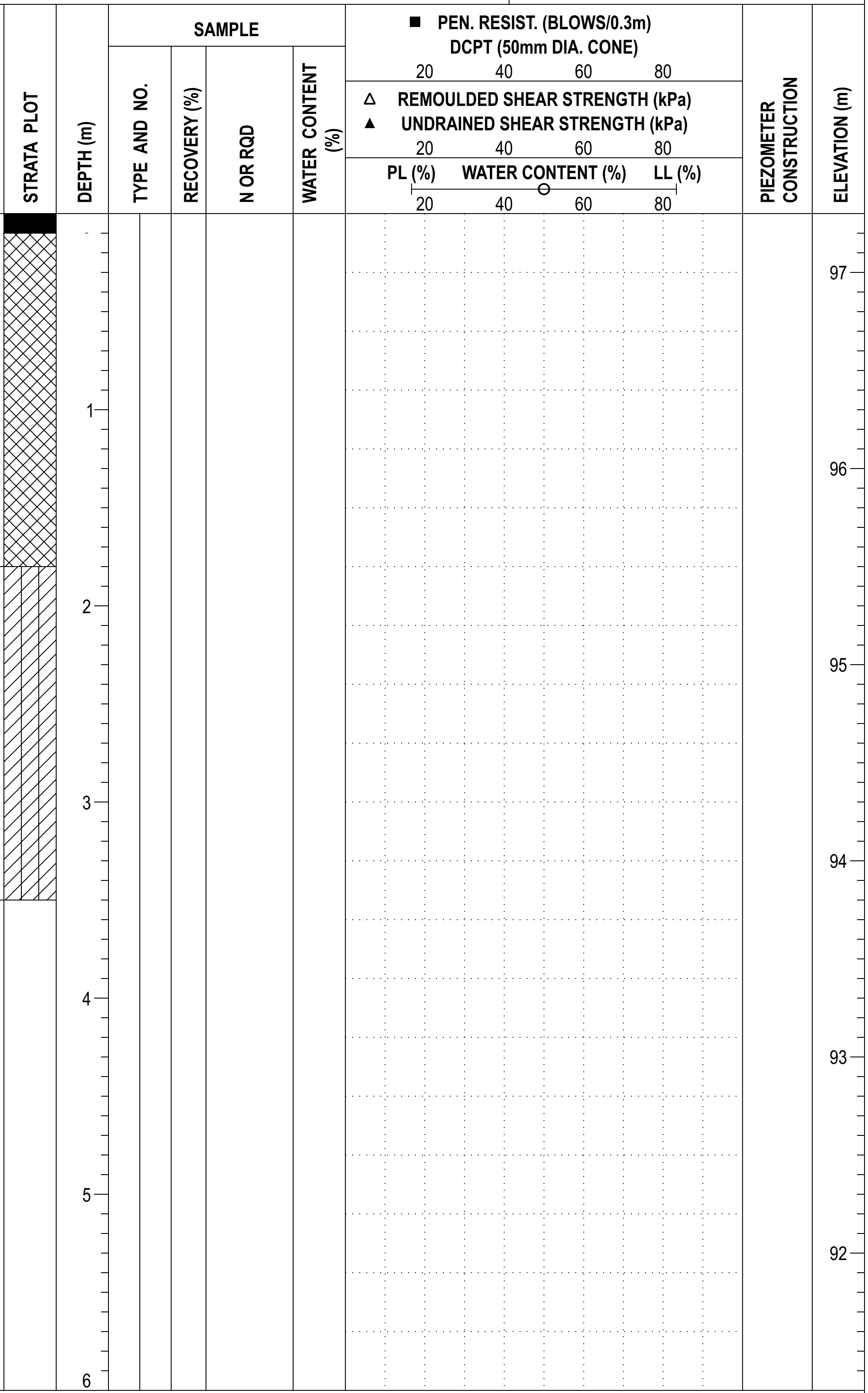
- Boulder at 1.4 m depth

1.80m [95.50m]

Brown **SILTY CLAY**

3.50m [93.80m]

End of Test Pit



P:/Autocad Drawings/Test Hole Data Files/PG74xx/PG7418/data.sqlite 2025-03-19, 12:17 Paterson_Template MA

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-use Development - 475 Terry Fox Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullvan Vollebekk Limited.

FILE NO. **PG3649**

REMARKS

HOLE NO. **BH 1**

BORINGS BY CME 55 Power Auger

DATE 27 October 2015

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.25
Very stiff, brown **SILTY CLAY** with sand	2.70
Compact, brown **SILTY SAND** with gravel, trace clay	4.85

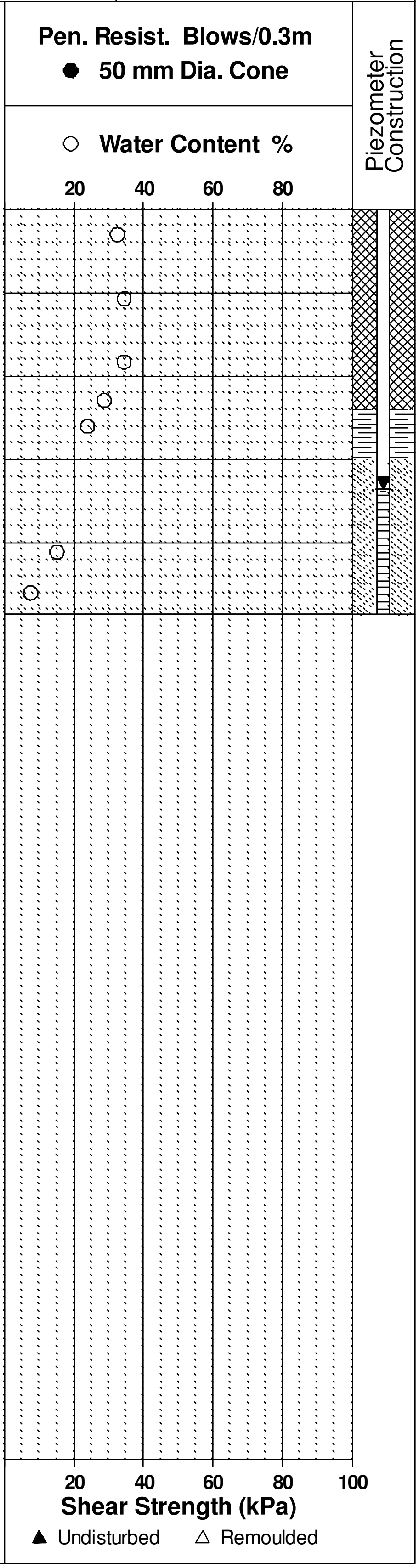
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	100	9
SS	2	100	16
SS	3	100	16
SS	4	100	12
SS	5	8	3
SS	6	75	28
SS	7	67	50+

DEPTH (m)	ELEV. (m)
0	99.26
1	98.26
2	97.26
3	96.26
4	95.26

End of Borehole

Practical refusal to augering at 4.85m depth

(GWL @ 3.35m-Nov. 12, 2015)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-use Development - 475 Terry Fox Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullvan Vollebekk Limited.

REMARKS

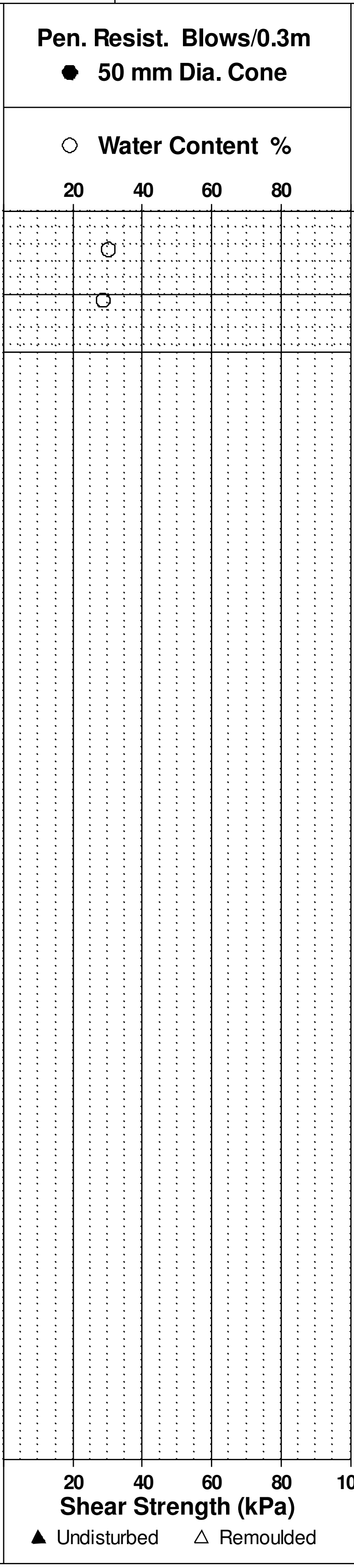
BORINGS BY CME 55 Power Auger

DATE 27 October 2015

FILE NO. PG3649

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	98.18	
TOPSOIL 0.30		AU	1					
Very stiff, brown **SILTY CLAY** with sand 1.37		SS	2		13	1	97.18	
BEDROCK: Fractured granite/gneiss 1.70		SS	3		50+			
End of Borehole Practical refusal to augering at 1.70m depth (BH dry upon completion)								



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-use Development - 475 Terry Fox Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullvan Vollebekk Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 27 October 2015

FILE NO. **PG3649**

HOLE NO. **BH 2A**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	98.18		
TOPSOIL 0.30	■								
Very stiff, brown **SILTY CLAY** with sand 1.37	▨					1	97.18		
Dense, brown **SILTY SAND** with gravel, trace clay 2.39		SS	1		50+	2	96.18		
End of Borehole Practical refusal to augering at 2.39m depth (BH dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-use Development - 475 Terry Fox Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullvan Vollebekk Limited.

REMARKS

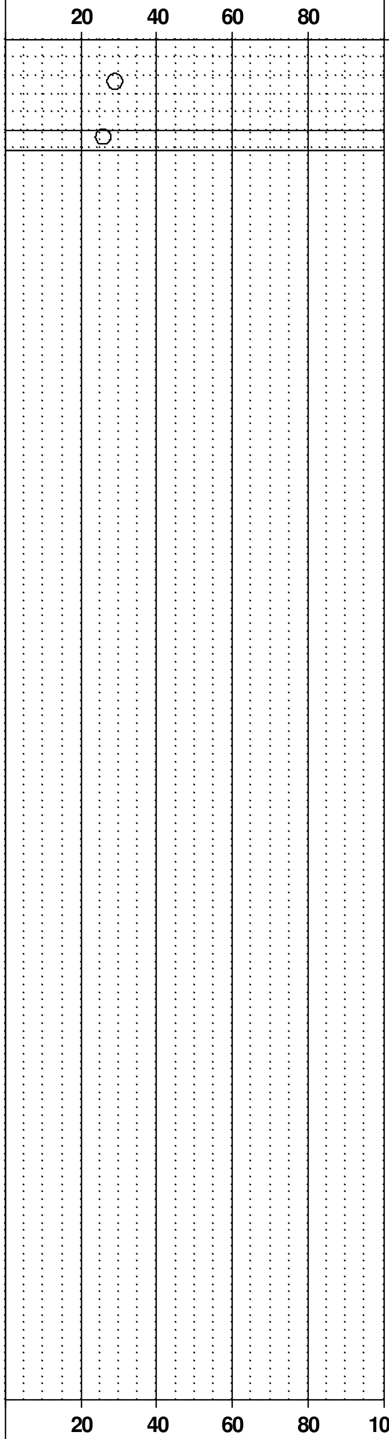
BORINGS BY CME 55 Power Auger

DATE 27 October 2015

FILE NO. PG3649

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.38
TOPSOIL 0.15							
Very stiff, brown **SILTY CLAY** with sand		AU	1				
1.22		SS	2	100	50+	1	96.38
End of Borehole Practical refusal to augering at 1.22m depth (BH dry upon completion)							



Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-use Development - 475 Terry Fox Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullvan Vollebekk Limited.

REMARKS

BORINGS BY CME 55 Power Auger

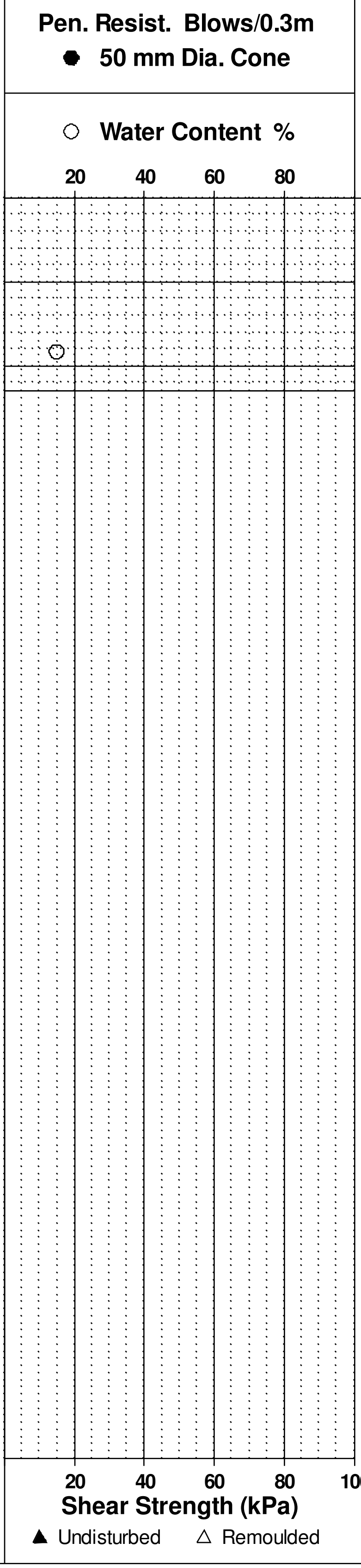
DATE 27 October 2015

FILE NO. PG3649

HOLE NO. BH 3A

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.15					
Very stiff, brown **SILTY CLAY** with sand 1.52					
Dense, brown **SILTY SAND** 2.29		SS	1	75	50+
End of Borehole Practical refusal to augering at 2.29m depth (BH dry upon completion)					

DEPTH (m)	ELEV. (m)
0	97.38
1	96.38
2	95.38



Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-use Development - 475 Terry Fox Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullvan Vollebekk Limited.

FILE NO. PG3649

REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger

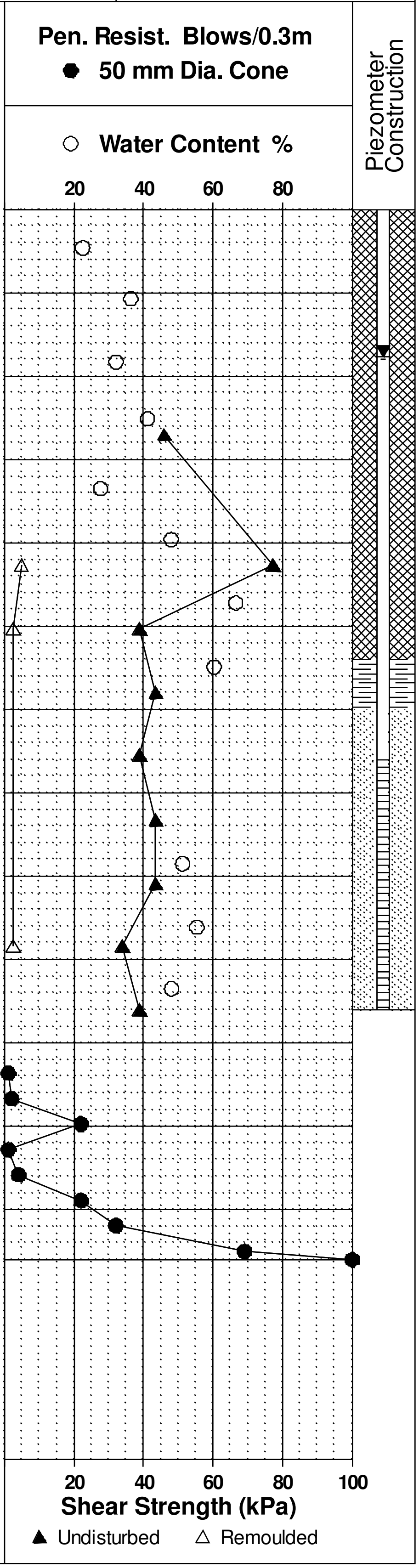
DATE 27 October 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.30						0	95.75
		AU	1				
		SS	2	100	11	1	94.75
		SS	3	100	3	2	93.75
		G	1				
Stiff to firm, brown **SILTY CLAY** with sand		SS	4	100	2	3	92.75
		G	2			4	91.75
		G	3			5	90.75
- grey by 5.0m depth		G	4				
		G	5			6	89.75
		G	6			7	88.75
		G	7			8	87.75
		G	8			9	86.75
9.60		G	9				
Dynamic Cone Penetration Test commenced at 9.60m depth						10	85.75
						11	84.75
						12	83.75
12.60							

End of Borehole

Practical DCPT refusal at 12.60m depth

(GWL @ 1.77m-Nov. 12, 2015)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-use Development - 475 Terry Fox Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullvan Vollebekk Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 27 October 2015

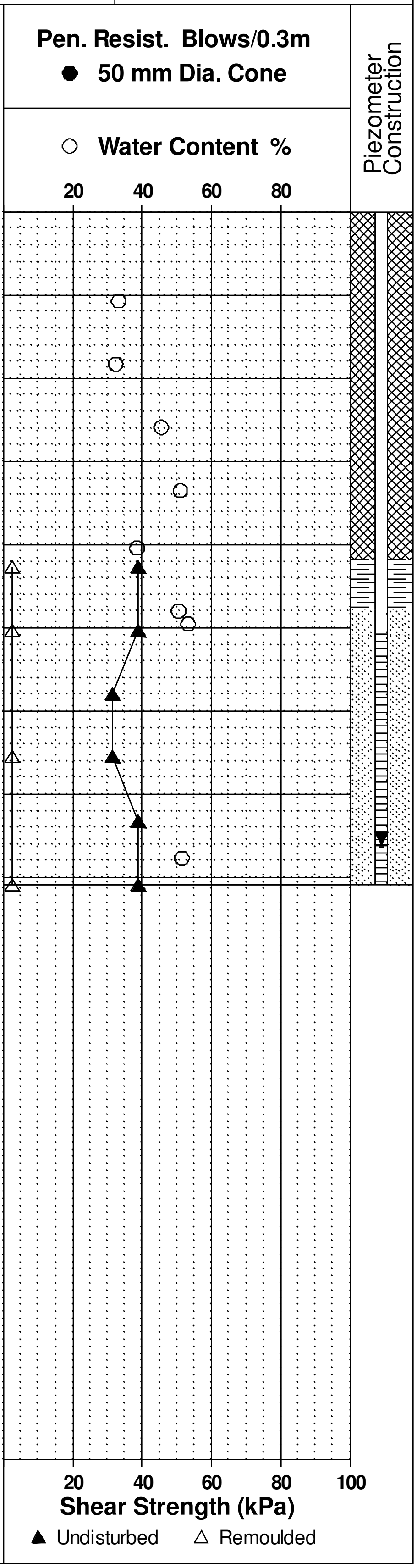
FILE NO. PG3649

HOLE NO. BH 5

SOIL DESCRIPTION		
GROUND SURFACE		
TOPSOIL	0.30	
Stiff to firm, brown **SILTY CLAY** with sand - grey by 4.3m depth	8.08	
End of Borehole (GWL @ 4.60m-Nov. 12, 2015)		

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	100	4
SS	2	100	2
SS	3	100	0
SS	4	100	1
G	1		
G	2		
G	3		
G	4		
G	5		
G	6		

DEPTH (m)	ELEV. (m)
0	95.15
1	94.15
2	93.15
3	92.15
4	91.15
5	90.15
6	89.15
7	88.15
8	87.15



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-use Development - 475 Terry Fox Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullvan Vollebekk Limited.

REMARKS

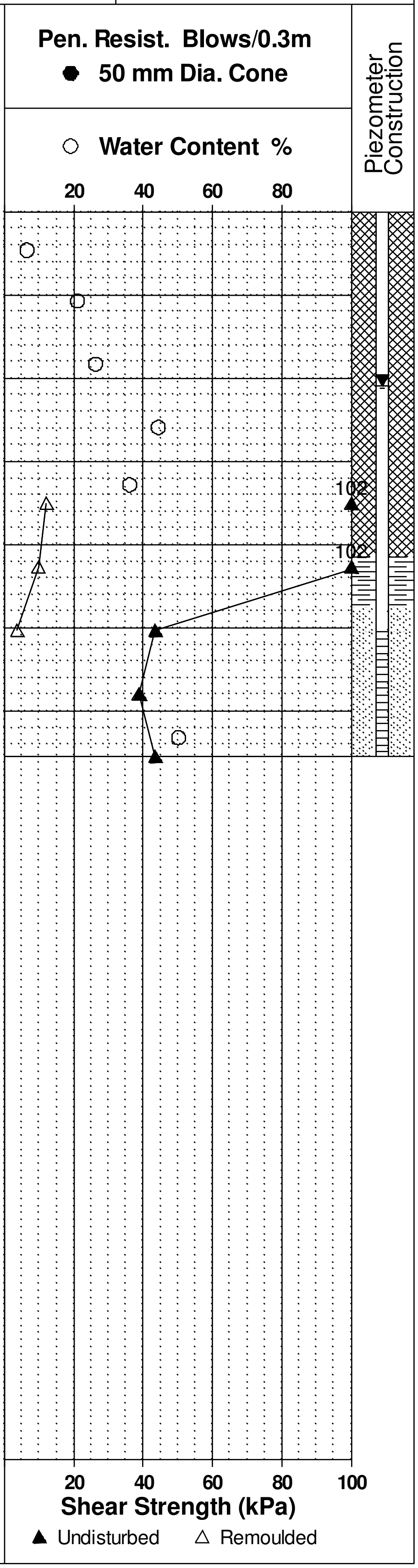
BORINGS BY CME 55 Power Auger

DATE 27 October 2015

FILE NO. PG3649

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.41
TOPSOIL 0.25							
Compact, brown **SAND** with gravel, some clay 1.50		AU	1				
		SS	2	58	13	1	96.41
Very stiff to stiff, brown **SILTY CLAY**		SS	3	100	19	2	95.41
		SS	4	100	3	3	94.41
		G	1				
		G	2			4	93.41
- firm and grey by 4.6m depth		G	3			5	92.41
		G	4			6	91.41
6.55		G	5				
End of Borehole							
(GWL @ 2.09m-Nov. 12, 2015)							



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)

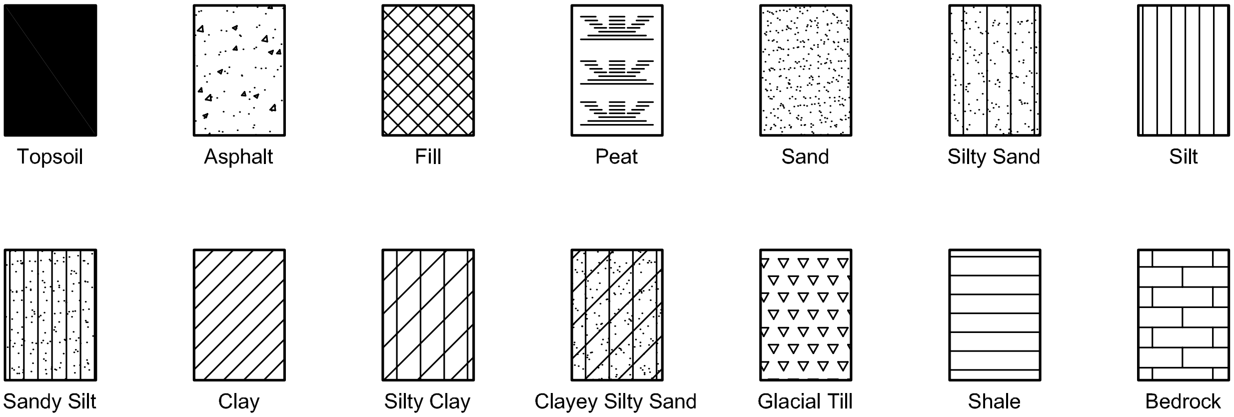
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

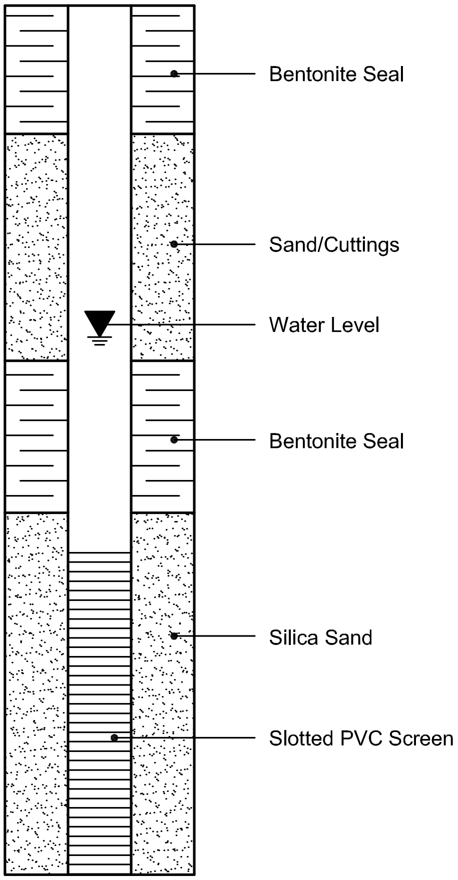
SYMBOLS AND TERMS (continued)

STRATA PLOT

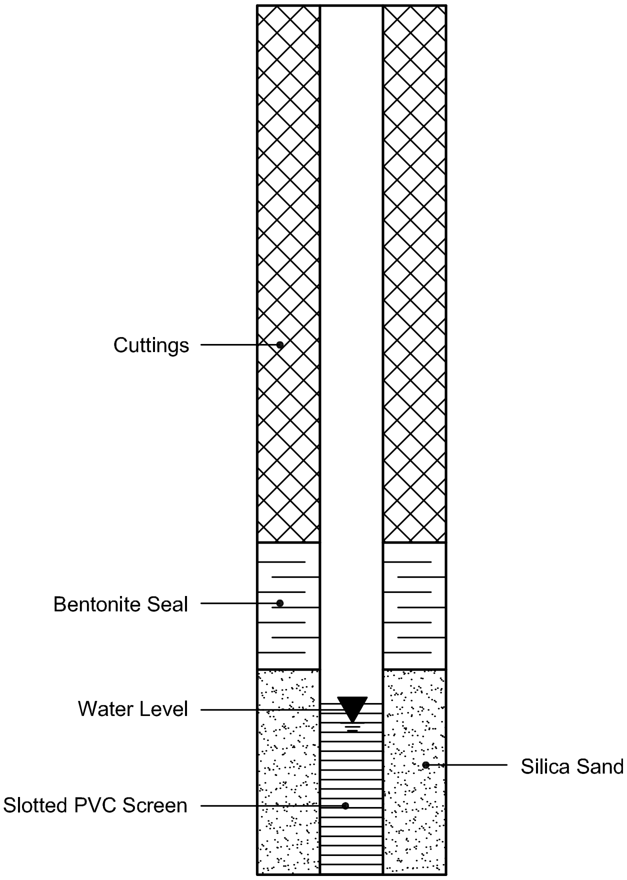


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Photograph of Rock Cores – BH2-25 – RC1

Photograph of Rock Core obtained from BH 2-25 from interval RC1

Rock Core interval ranged between 15’2” to 19’8”

Recovery (%) = 100

Rock Quality Designation (RQD) = 83

Photograph of Rock Cores – BH2-25 – RC2

Photograph of Rock Core obtained from BH 2-25 from interval RC2

Rock Core interval ranged between 19’8” to 22’6”

Recovery (%) = 100

Rock Quality Designation (RQD) = 58

PATERSON GROUP

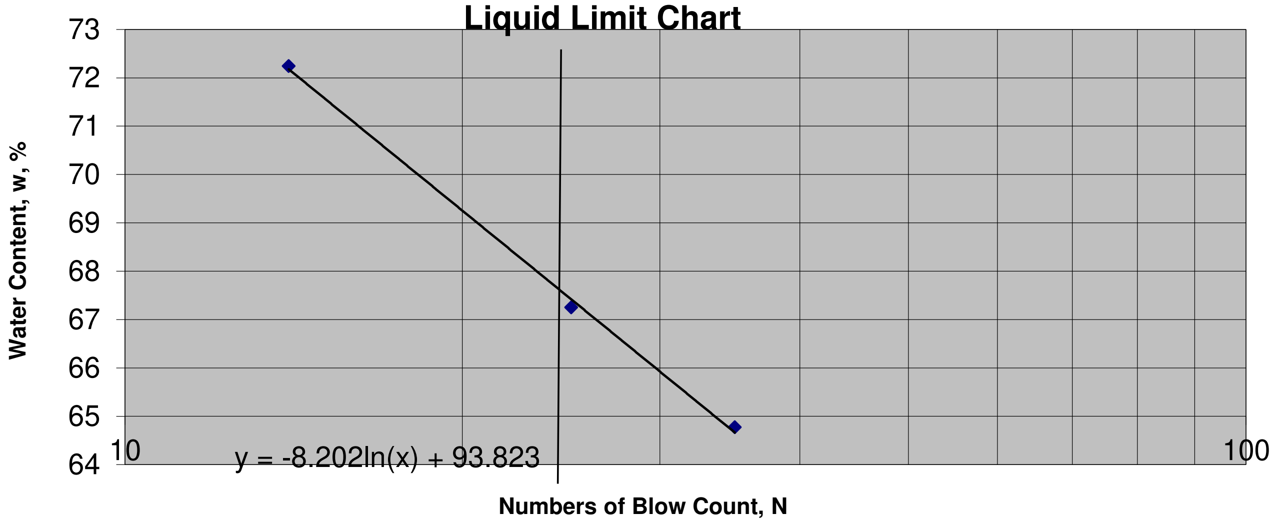
ATTERBERG LIMITS
LS-703/704

CLIENT:	Ironclad Developments	FILE NO.:	PG7418
PROJECT:	475 Terry Fox Drive	DATE SAMPLED:	-
LOCATION:	BH 2-25-SS4 @ 7'6"-9'6"	DATE REPORTED:	19-Feb

CAN NO.	**3**	**21**	**16**				
WT. OF CAN	8.71	8.68	8.77				
WT. OF SOIL & CAN	13.24	14.45	14.57				
WT. OF DRY SOIL & CAN	11.34	12.13	12.29				
WT. OF MOISTURE	1.9	2.32	2.28				
WT. OF DRY SOIL & CAN	2.63	3.45	3.52				
WATER CONTENT, w, %	**72.24**	**67.25**	**64.77**				
NO. OF BLOWS, N	14	25	35				

CAN NO.	**13**	**11**
WT. OF CAN	19.36	19.90
WT. OF SOIL & CAN	28.92	33.20
WT. OF DRY SOIL & CAN	26.59	30.05
WT. OF MOISTURE	2.33	3.15
WT. OF DRY SOIL & CAN	7.23	10.15
WATER CONTENT, w, %	**32.23**	**31.03**

RESULTS	
LIQUID LIMIT	**68**
PLASTIC LIMIT	**32**
PLASTICITY INDEX	**36**



TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

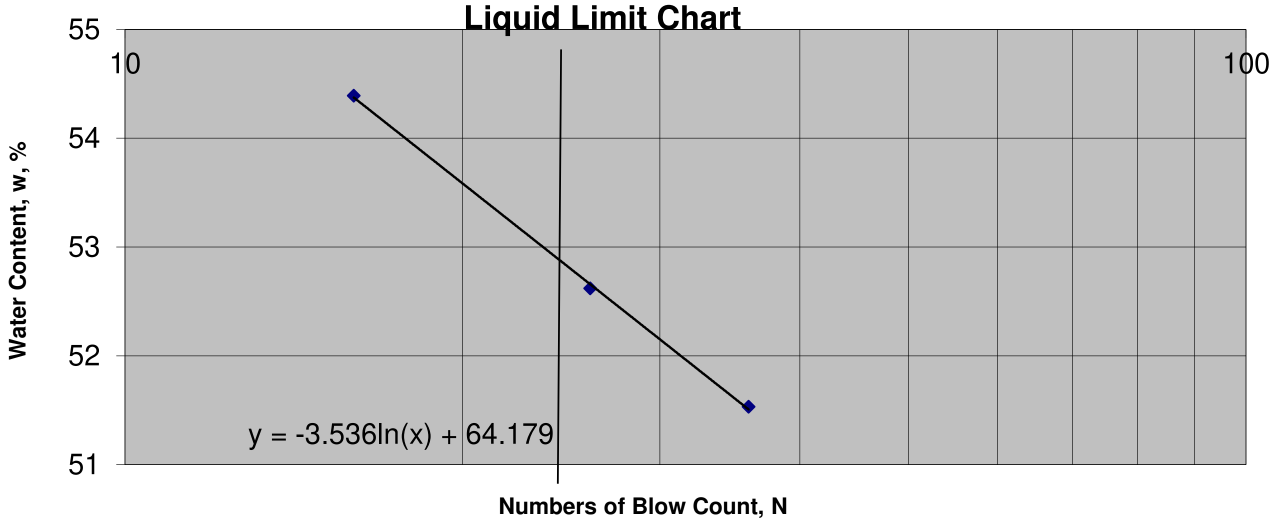
ATTERBERG LIMITS
LS-703/704

CLIENT:	Ironclad Developments	FILE NO.:	PG7418
PROJECT:	475 Terry Fox Drive	DATE SAMPLED:	-
LOCATION:	BH 5A-25-SS7 @ 15'-17'	DATE REPORTED:	19-Feb

CAN NO.	**4**	**11**	**12**				
WT. OF CAN	8.68	8.66	8.74				
WT. OF SOIL & CAN	17.99	20.32	19.62				
WT. OF DRY SOIL & CAN	14.71	16.30	15.92				
WT. OF MOISTURE	3.28	4.02	3.7				
WT. OF DRY SOIL & CAN	6.03	7.64	7.18				
WATER CONTENT, w, %	**54.39**	**52.62**	**51.53**				
NO. OF BLOWS, N	16	26	36				

CAN NO.	**2**	**18**
WT. OF CAN	19.93	20.02
WT. OF SOIL & CAN	28.43	28.10
WT. OF DRY SOIL & CAN	26.79	26.56
WT. OF MOISTURE	1.64	1.54
WT. OF DRY SOIL & CAN	6.86	6.54
WATER CONTENT, w, %	**23.91**	**23.55**

RESULTS	
LIQUID LIMIT	**53**
PLASTIC LIMIT	**24**
PLASTICITY INDEX	**29**



TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

Linear Shrinkage
ASTM D4943-02

CLIENT:	Ironclad Developments	DEPTH	7'6" - 9'6"	FILE NO.:	PG7418
PROJECT:	475 Terry Fox Drive, Ottawa	BH OR TP No:	BH 2-25 - SS4	DATE SAMPLED	-
LAB No:	58924	TESTED BY:	C.P	DATE RECEIVED	10-Feb-25
SAMPLED BY:	N.V.	DATE REPORTED:	19-Feb-25	DATE TESTED	11-Feb-25

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(7)	Calibration (Two Trials)	Tin NO.(A1)	
Tare	4.94	Tin	4.77	4.77
Soil Pat Wet + Tare	61.49	Tin + Grease	4.93	4.94
Soil Pat Wet	56.55	Glass	43.25	43.25
Soil Pat Dry + Tare	35.2	Tin + Glass + Water	85.62	85.61
Soil Pat Dry	30.26	Volume	37.44	37.42
Moisture	**86.88**	**Average Volume**	**37.43**	

Soil Pat + String	30.39
Soil Pat + Wax + String in Air	33.5
Soil Pat + Wax + String in Water	13.01
Volume Of Pat (Vdx)	20.49

RESULTS:

Shrinkage Limit	**19.35**
Shrinkage Ratio	**1.780**
Volumetric Shrinkage	**120.233**
Linear Shrinkage	**23.137**

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

PATERSON GROUP

CONCRETE CORE COMPRESSIVE STRENGTH CSA A23.2-14C

CLIENT:	**Ironclad**	FILE No.:	**PG7418**
PROJECT:	Lab Testing	REPORT No.:	1
SITE ADDRESS:	**475 Terry Fox Drive**	DATE REPT'D:	10-Feb-25
STRUCTURE TYPE & LOCATION:	BH 2-25 / 15'6" - 16'		

SAMPLE INFORAMTION			
LAB NO.:	**58929**		
SAMPLE NO.:	RC1		
LOCATION:	15'6" - 16'		
SAMPLE DATES			
DATE CAST	-		
DATE CORED	-		
DATE RECEIVED	10-Feb-25		
DATE TESTED	10-Feb-25		
SAMPLE DIMENSIONS			
AVERAGE DIAMETER (mm)	47.00		
HEIGHT (mm)	94.00		
WEIGHT (g)	460		
AREA (mm^2)	1735		
VOLUME (cm^3)	163		
UNIT WEIGHT (kg/m^3)	**2821**		
TEST RESULTS			
H / D RATIO	2.00		
CORRECTION FACTOR	1.000		
LOAD (lbs)	42000		
GROSS Mpa	107.7		
MPa CORRECTED	**107.7**		
FORM OF BREAK	**Type A**		
DIRECTION OF LOADING	PARALLEL		
CURING CONDITIONS	SITE→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→		

REMARKS

TECHNICAL PERSONNEL

TECHNICIAN:	**VERIFIED BY:**	**C. Beadow**	**APPROVED BY:**	**Joe Forsyth, P. Eng.**

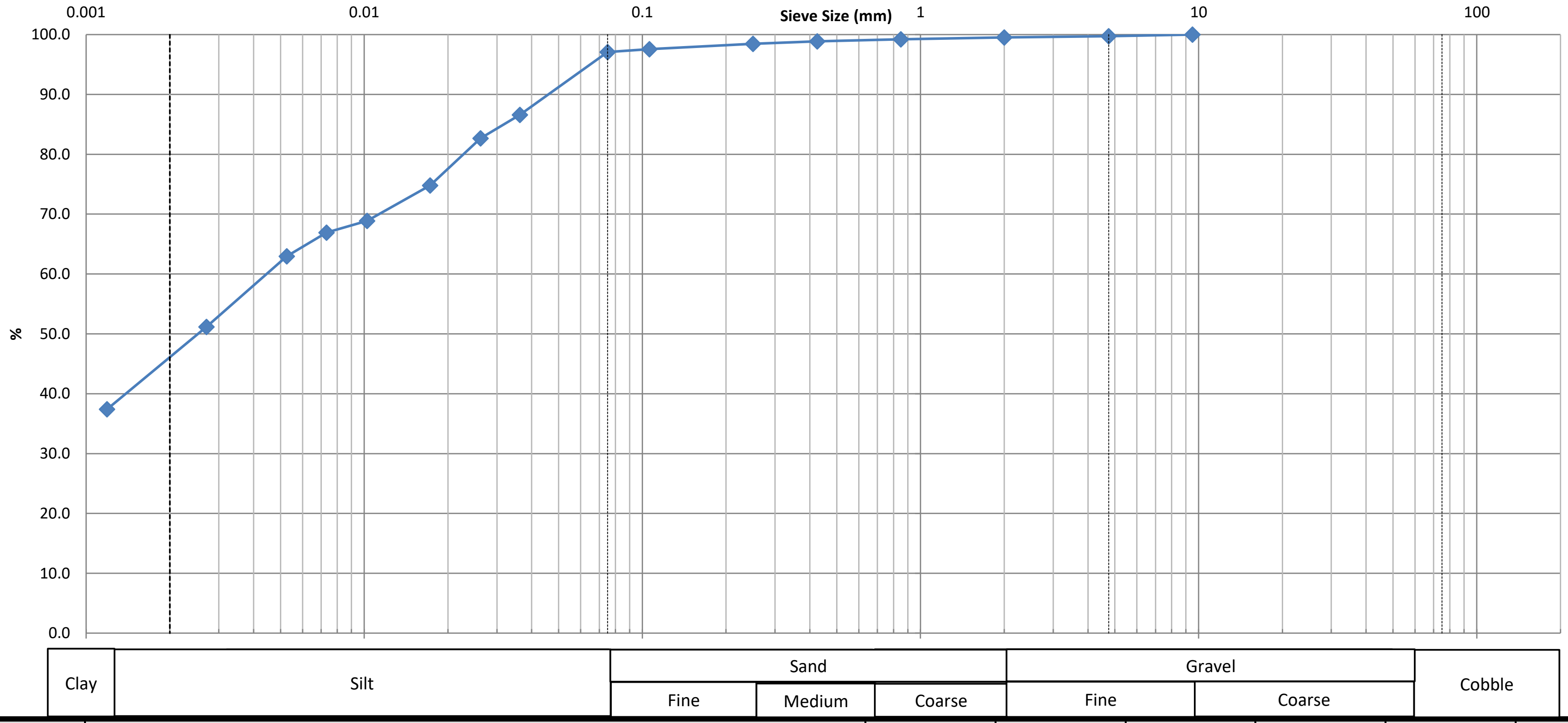
CERTIFIED LAB

John D. Paterson & Associates Ltd., 28 Concourse Gate, Nepean, ON

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Ironclad Developments	DEPTH:	15' - 17'	FILE NO:	PG7418
CONTRACT NO.:		BH OR TP No.:	BH 1-25 - SS5	LAB NO:	58925
PROJECT:	475 Terry Fox Drive, Ottawa			DATE RECEIVED:	10-Feb-25
				DATE TESTED:	11-Feb-25
DATE SAMPLED:	-			DATE REPORTED:	19-Feb-25
SAMPLED BY:	N.V.			TESTED BY:	D.K



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						59.4%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.3	2.6		50.6		46.5	

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Ironclad Developments	DEPTH:	15' - 17'	FILE NO.:	PG7418
PROJECT:	475 Terry Fox Drive, Ottawa	BH OR TP No.:	BH 1-25 - SS5	DATE SAMPLED:	-
LAB No. :	58925	TESTED BY:	D.K	DATE RECEIVED:	10-Feb-25
SAMPLED BY:	N.V.	DATE REPT'D:	19-Feb-25	DATE TESTED:	11-Feb-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
105.4	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	50.00	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	1.26	AIR DRY	105.40	105.40
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	105.40	105.40
		CORRECTED	1.000	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5	**0.0**	0.0	**100.0**
4.75	**0.3**	0.3	**99.7**
2.0	**0.5**	0.5	**99.5**
Pan	**104.9**		
0.850	**0.16**	0.8	**99.2**
0.425	**0.33**	1.1	**98.9**
0.250	**0.53**	1.5	**98.5**
0.106	**0.98**	2.4	**97.6**
0.075	**1.23**	2.9	**97.1**
Pan	**1.26**		

SIEVE CHECK	0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^oC)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	07:48	50.0	6.0	23.0	0.0363	87.0	**86.6**
2	07:49	48.0	6.0	23.0	0.0262	83.1	**82.7**
5	07:52	44.0	6.0	23.0	0.0173	75.1	**74.8**
15	08:02	41.0	6.0	23.0	0.0102	69.2	**68.9**
30	08:17	40.0	6.0	23.0	0.0073	67.2	**66.9**
60	08:47	38.0	6.0	23.0	0.0053	63.3	**63.0**
250	11:57	32.0	6.0	23.0	0.0027	51.4	**51.2**
1440	07:47	25.0	6.0	23.0	0.0012	37.6	**37.4**

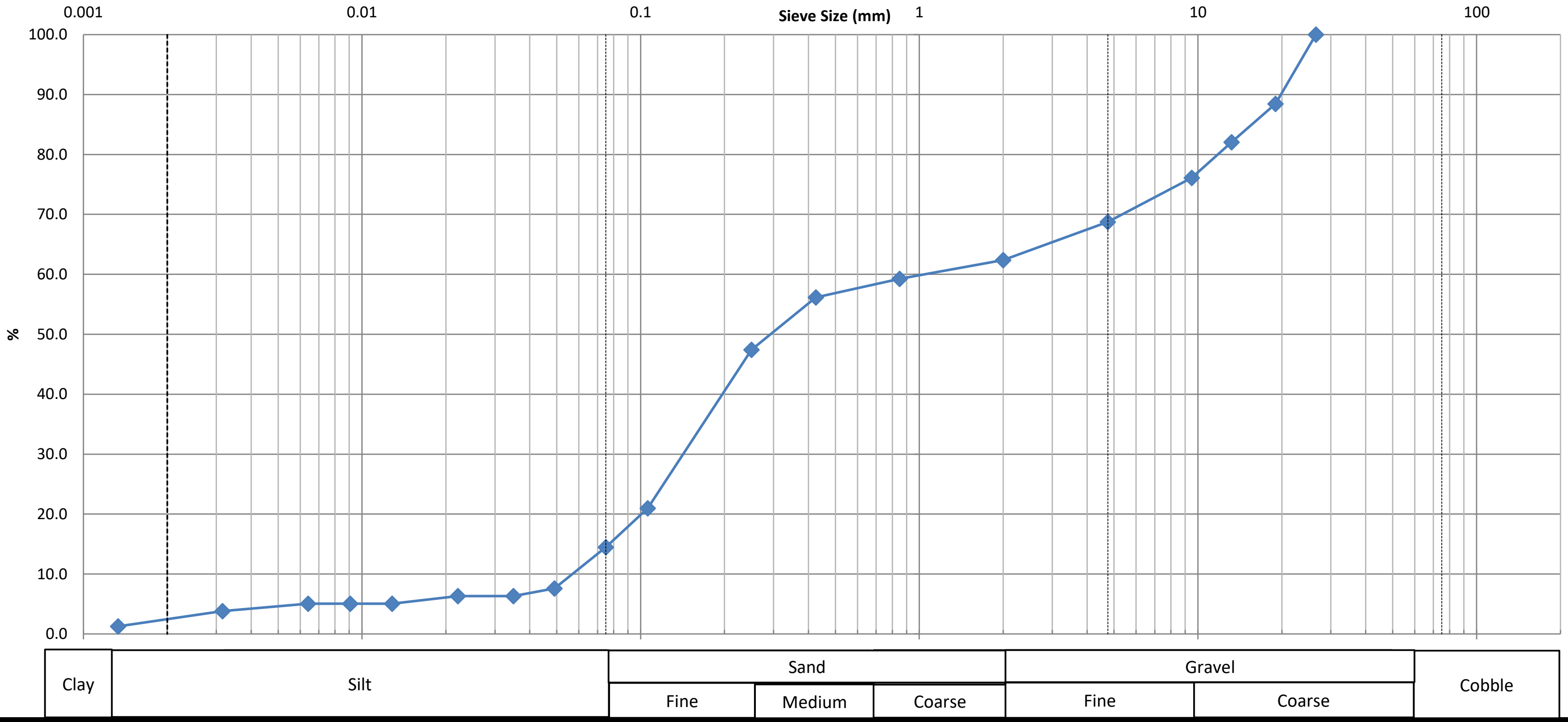
Moisture = 59.4%

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

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Ironclad Developments	DEPTH:	30' - 32'	FILE NO:	PG7418
CONTRACT NO.:		BH OR TP No.:	BH 1-25 SS8	LAB NO:	58928
PROJECT:	475 Terry Fox Drive, Ottawa			DATE RECEIVED:	10-Feb-25
				DATE TESTED:	11-Feb-25
DATE SAMPLED:	-			DATE REPORTED:	19-Feb-25
SAMPLED BY:	N.V.			TESTED BY:	D.K



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		12.7%					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				31.3	54.3	12.3	2.2

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Ironclad Developments	DEPTH:	30' - 32'	FILE NO.:	PG7418
PROJECT:	475 Terry Fox Drive, Ottawa	BH OR TP No.:	BH 1-25 SS8	DATE SAMPLED:	-
LAB No. :	58928	TESTED BY:	D.K	DATE RECEIVED:	10-Feb-25
SAMPLED BY:	N.V.	DATE REPT'D:	19-Feb-25	DATE TESTED:	11-Feb-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
517.9	2.700

INITIAL WEIGHT	50.00
WEIGHT CORRECTED	48.75
WT. AFTER WASH BACK SIEVE	40.00
SOLUTION CONCENTRATION	40 g/L

HYGROSCOPIC MOISTURE

TARE WEIGHT	0.00	ACTUAL WEIGHT
AIR DRY	530.00	530.00
OVEN DRY	516.70	516.70
CORRECTED	0.975	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5	**0.0**	0.0	**100.0**
19	**59.9**	11.6	**88.4**
13.2	**93.0**	17.9	**82.1**
9.5	**123.7**	23.9	**76.1**
4.75	**161.9**	31.3	**68.7**
2.0	**194.8**	37.6	**62.4**
Pan	**323.1**		
0.850	**2.52**	40.8	**59.2**
0.425	**4.99**	43.8	**56.2**
0.250	**12.00**	52.6	**47.4**
0.106	**33.20**	79.0	**21.0**
0.075	**38.40**	85.5	**14.5**
Pan	**40.00**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^oC)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	07:47	12.0	6.0	23.0	0.0491	12.2	**7.6**
2	07:48	11.0	6.0	23.0	0.0349	10.1	**6.3**
5	07:51	11.0	6.0	23.0	0.0221	10.1	**6.3**
15	08:01	10.0	6.0	23.0	0.0128	8.1	**5.1**
30	08:16	10.0	6.0	23.0	0.0091	8.1	**5.1**
60	08:46	10.0	6.0	23.0	0.0064	8.1	**5.1**
250	11:56	9.0	6.0	23.0	0.0032	6.1	**3.8**
1440	07:46	7.0	6.0	23.0	0.0013	2.0	**1.3**

Moisture = 12.7%

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

Certificate of Analysis

Report Date: 13-Feb-2025

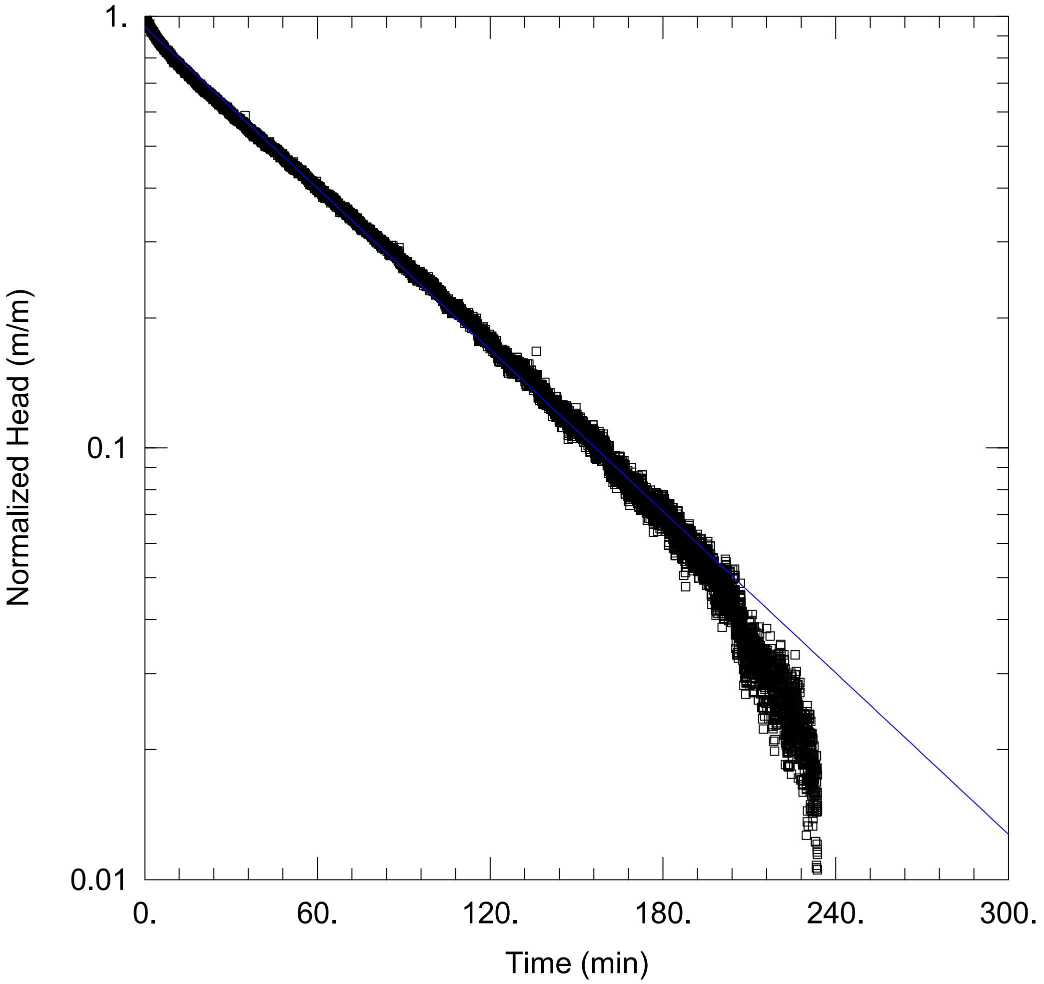
Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 10-Feb-2025

Client PO: 62347

Project Description: PG7418

	Client ID:	BH 5A-25 SS4	-	-	-		
	Sample Date:	07-Feb-25 16:00	-	-	-	-	-
	Sample ID:	2507065-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	59.4	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.25	-	-	-	-	-
Resistivity	0.1 Ohm.m	33.2	-	-	-	-	-
Anions							
Chloride	10 ug/g	58	-	-	-	-	-
Sulphate	10 ug/g	103	-	-	-	-	-



BH1-25 FALLING HEAD 1 OF 1

PROJECT INFORMATION

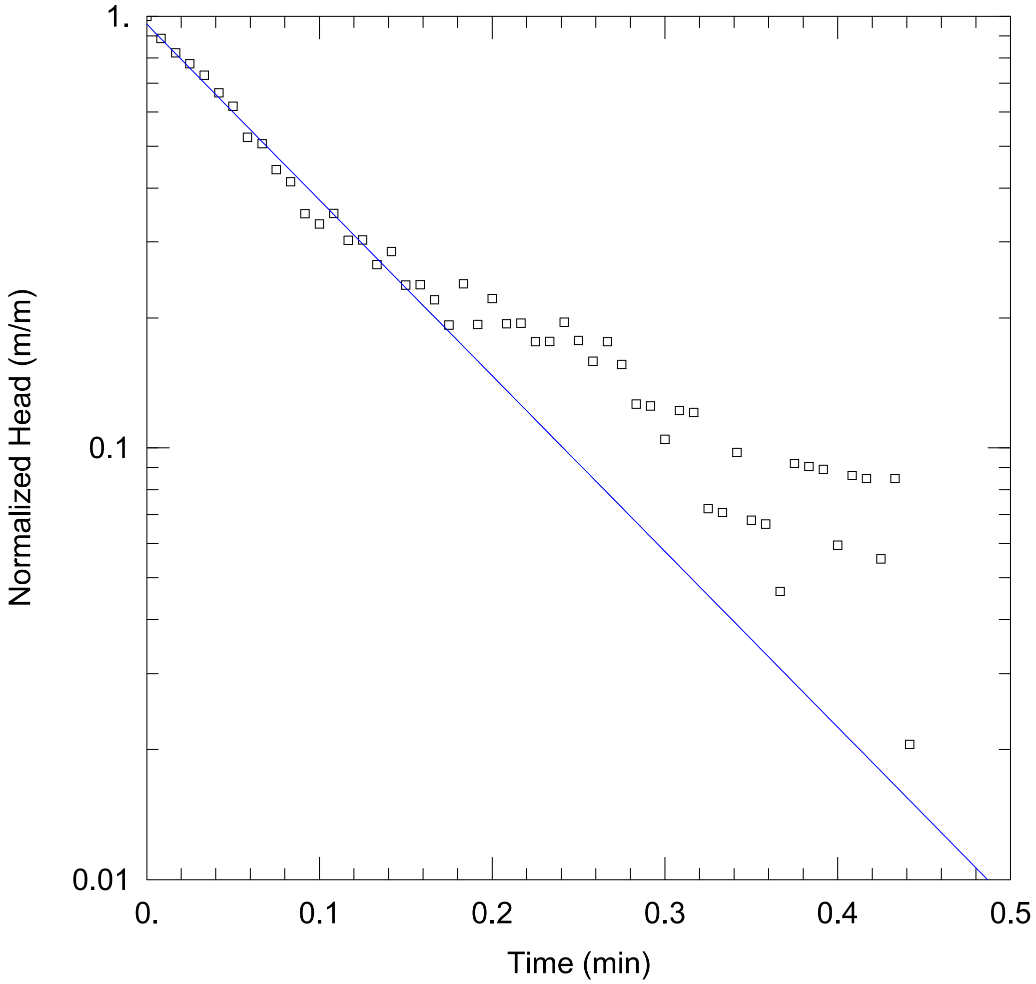
Company: Paterson Group
Client: Ironclad Developments Inc.
Project: PG7418
Location: 475 Terry Fox Drive
Test Well: BH1-25
Test Date: February 18, 2025

WELL DATA (BH1-25)

Initial Displacement: 0.47 m
Total Well Penetration Depth: 6.75 m
Casing Radius: 0.0254 m
Static Water Column Height: 6.75 m
Screen Length: 1.524 m
Well Radius: 0.1048 m

SOLUTION

Aquifer Model: Confined
Solution Method: Hvorslev
K = 2.29E-7 m/sec
y0 = 0.4436 m



BH2-25 FALLING HEAD 1 OF 1

PROJECT INFORMATION

Company: Paterson Group
Client: Ironclad Developments Inc.
Project: PG7418
Location: 475 Terry Fox Drive
Test Well: BH2-25
Test Date: February 18, 2025

WELL DATA (BH2-25)

Initial Displacement: 0.062 m
Total Well Penetration Depth: 4.765 m
Casing Radius: 0.01588 m

Static Water Column Height: 4.765 m
Screen Length: 1.524 m
Well Radius: 0.0381 m

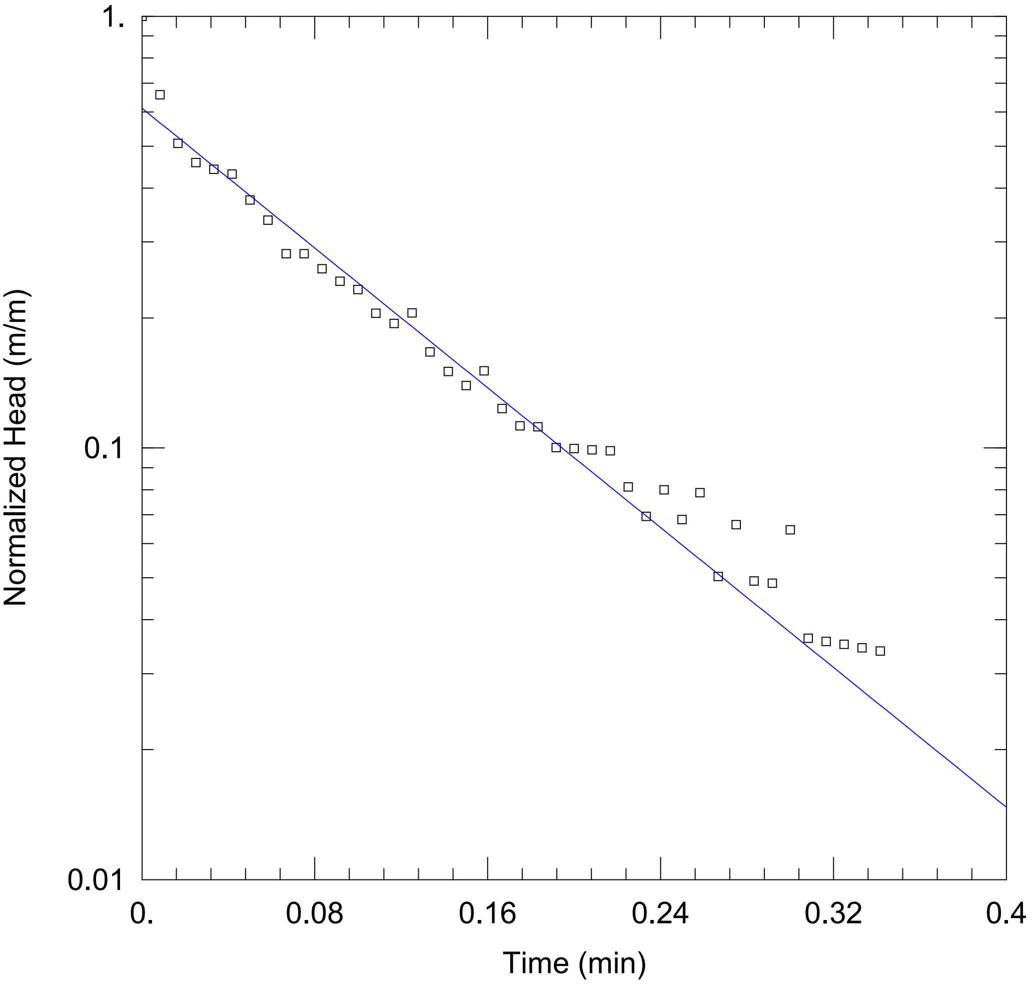
SOLUTION

Aquifer Model: Confined

K = 7.148E-5 m/sec

Solution Method: Hvorslev

y0 = 0.05941 m



BH2-25 RISING HEAD 1 OF 1

PROJECT INFORMATION

Company: Paterson Group
Client: Ironclad Developments Inc.
Project: PG7418
Location: 475 Terry Fox Drive
Test Well: BH2-25
Test Date: February 18, 2025

WELL DATA (BH2-25)

Initial Displacement: 0.105 m
Total Well Penetration Depth: 4.765 m
Casing Radius: 0.01588 m
Static Water Column Height: 4.765 m
Screen Length: 1.524 m
Well Radius: 0.0381 m

SOLUTION

Aquifer Model: Confined
Solution Method: Hvorslev
K = 7.109E-5 m/sec
y0 = 0.0643 m

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 1A & FIGURE 1B – GLOBAL STABLITY ANALYSIS RESULTS

FIGURE 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 – TYPICAL RETAINING WALL CROSS-SECTION

DRAWING PG7418-1 – TEST HOLE LOCATION PLAN

DRAWING PG7418-2 – PERMISSIBLE GRADE RAISE PLAN

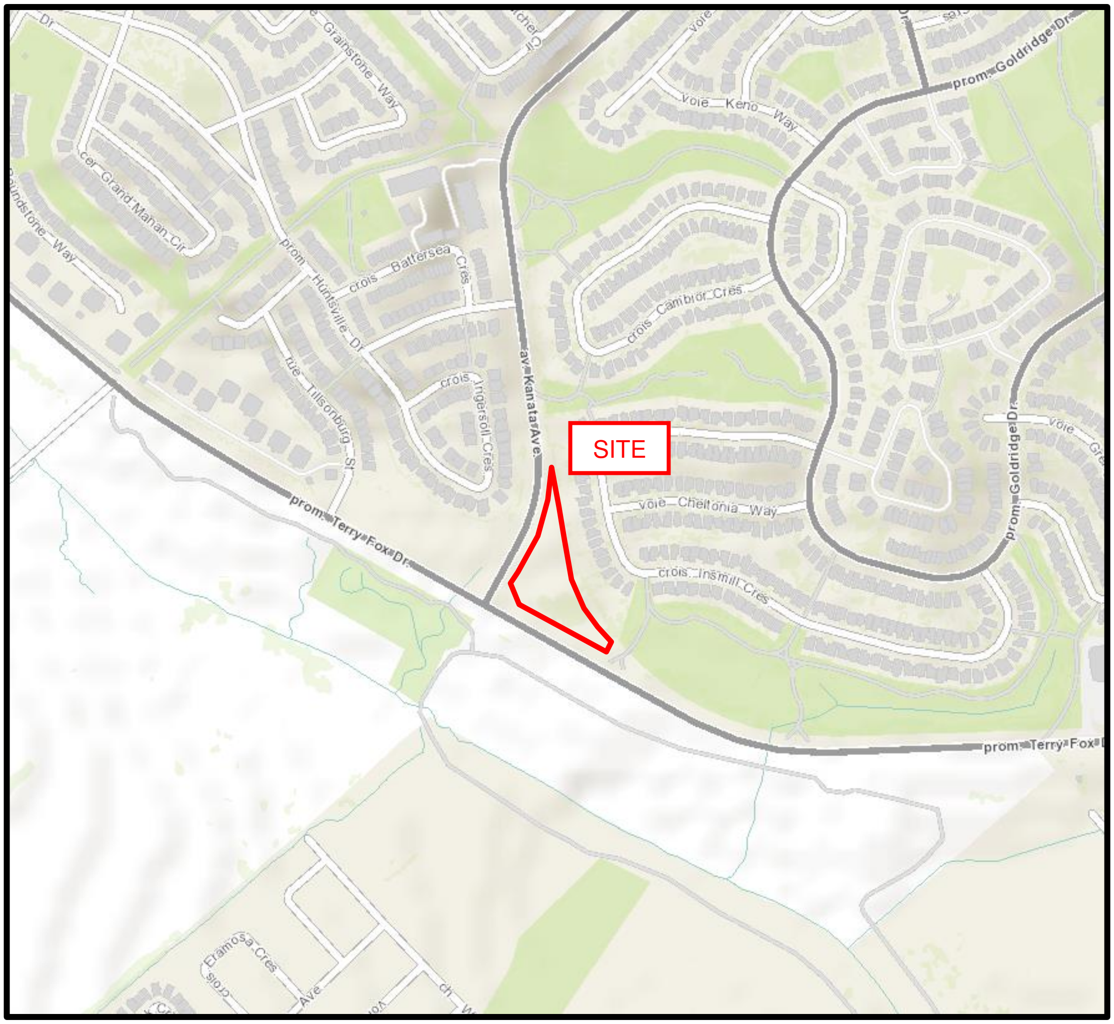
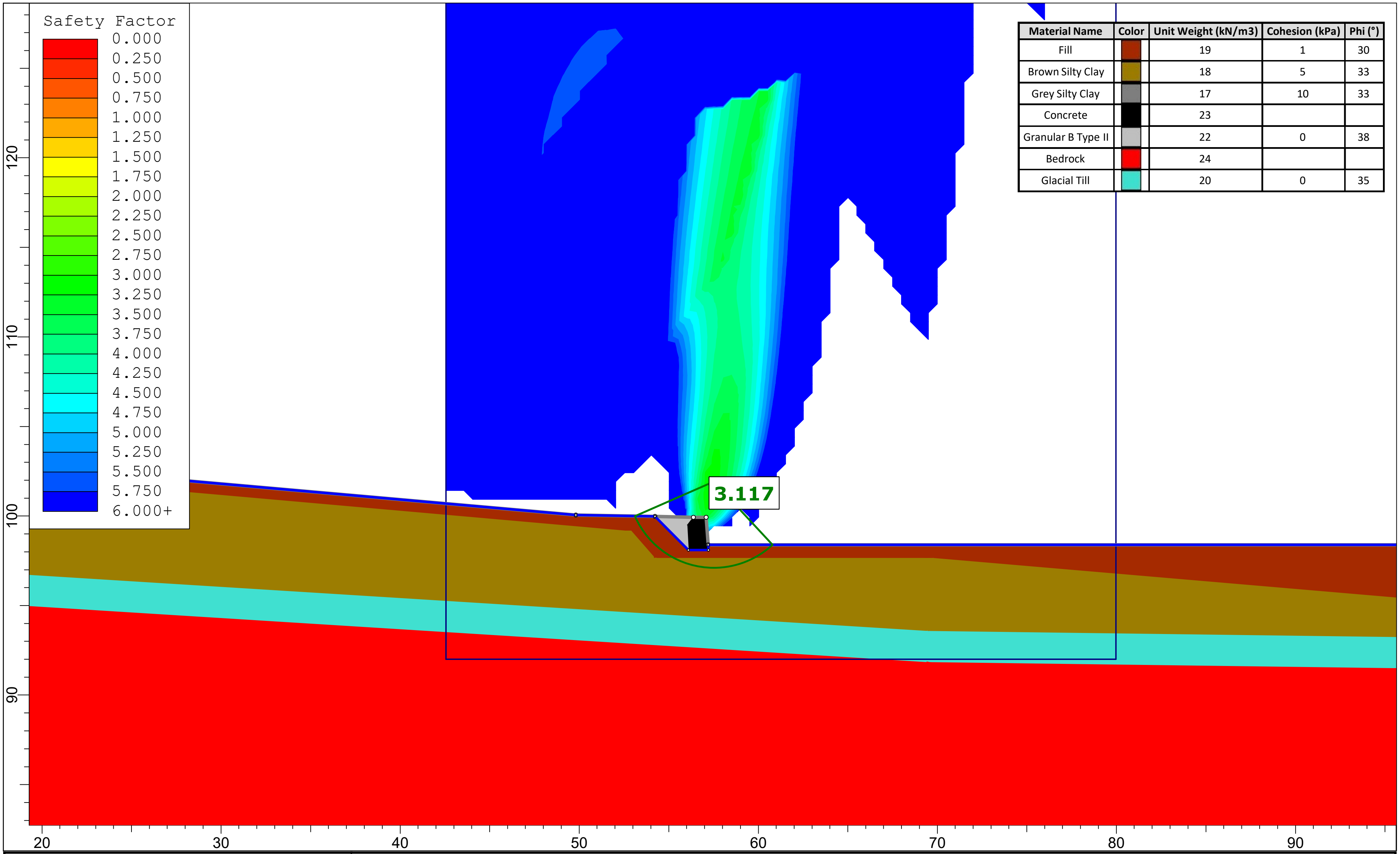


FIGURE 1

KEY PLAN

PATERSON GROUP



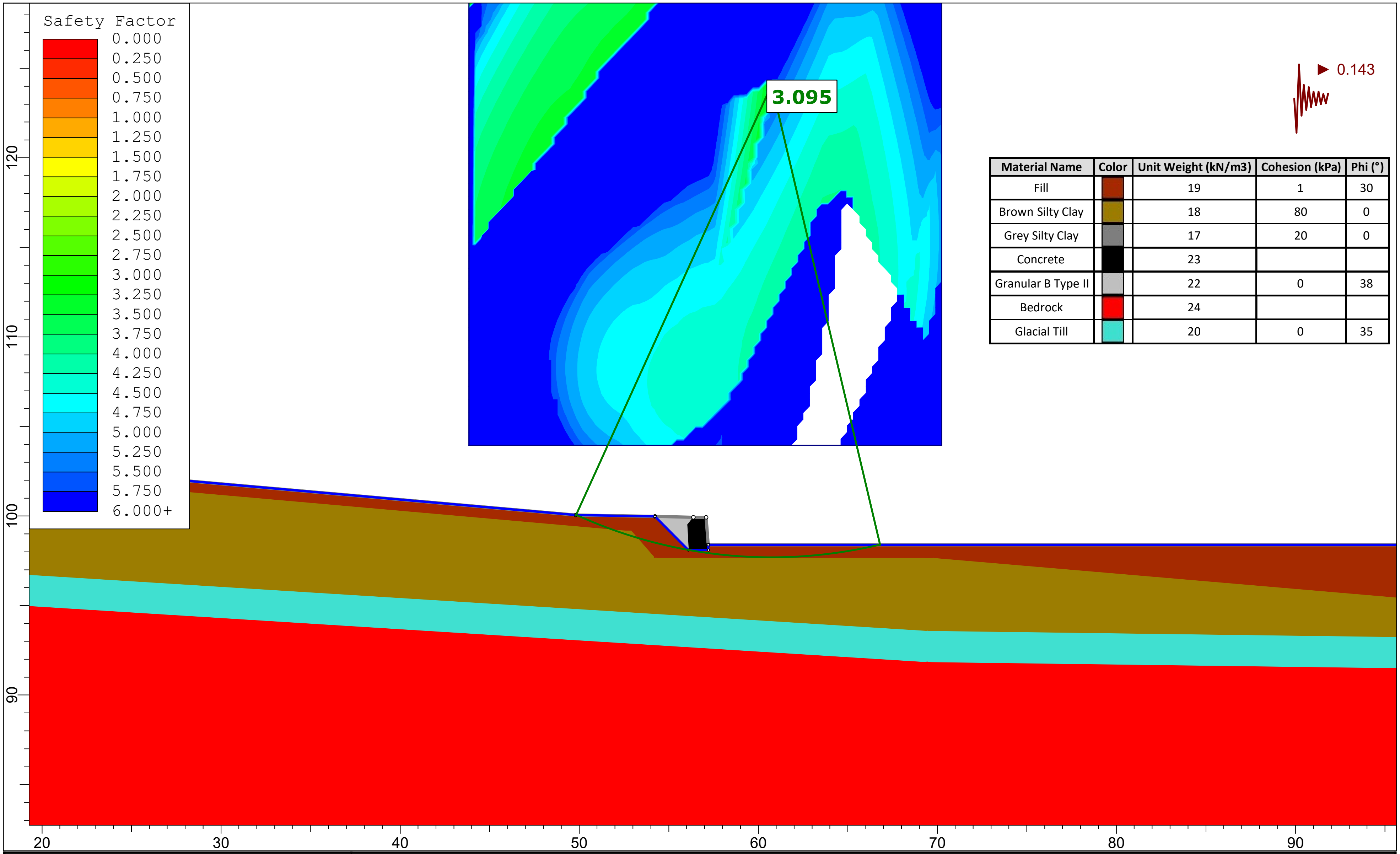
Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Fill	🟫	19	1	30
Brown Silty Clay	🟫	18	5	33
Grey Silty Clay	▨	17	10	33
Concrete	⬛	23		
Granular B Type II	⬜	22	0	38
Bedrock	🟥	24		
Glacial Till	🟦	20	0	35

PATERSON GROUP

SLIDEINTERPRET 9.031

Project	**Ironclad Developments Inc. Slope Stability Analysis 475 Terry Fox Drive, Ottawa, Ontario**
Figure No.	**Figure 1A - Section A - Proposed Conditions - Static Loading**

Drawn by:	NFRV	***Company:***	Paterson Group
Date:	2025-10-29	***File No.***	PG7418



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Fill	🟫	19	1	30
Brown Silty Clay	🟨	18	80	0
Grey Silty Clay	▨	17	20	0
Concrete	⬛	23		
Granular B Type II	⬜	22	0	38
Bedrock	🟥	24		
Glacial Till	🟦	20	0	35

PATERSON GROUP

SLIDEINTERPRET 9.031

Project	**Ironclad Developments Inc.** **Slope Stability Analysis** **475 Terry Fox Drive, Ottawa, Ontario**
Figure No.	**Figure 1B - Section A - Proposed Conditions - Seismic Loading**

Drawn by:	NFRV	***Company:***	Paterson Group
Date:	2025-10-29	***File No.***	PG7418

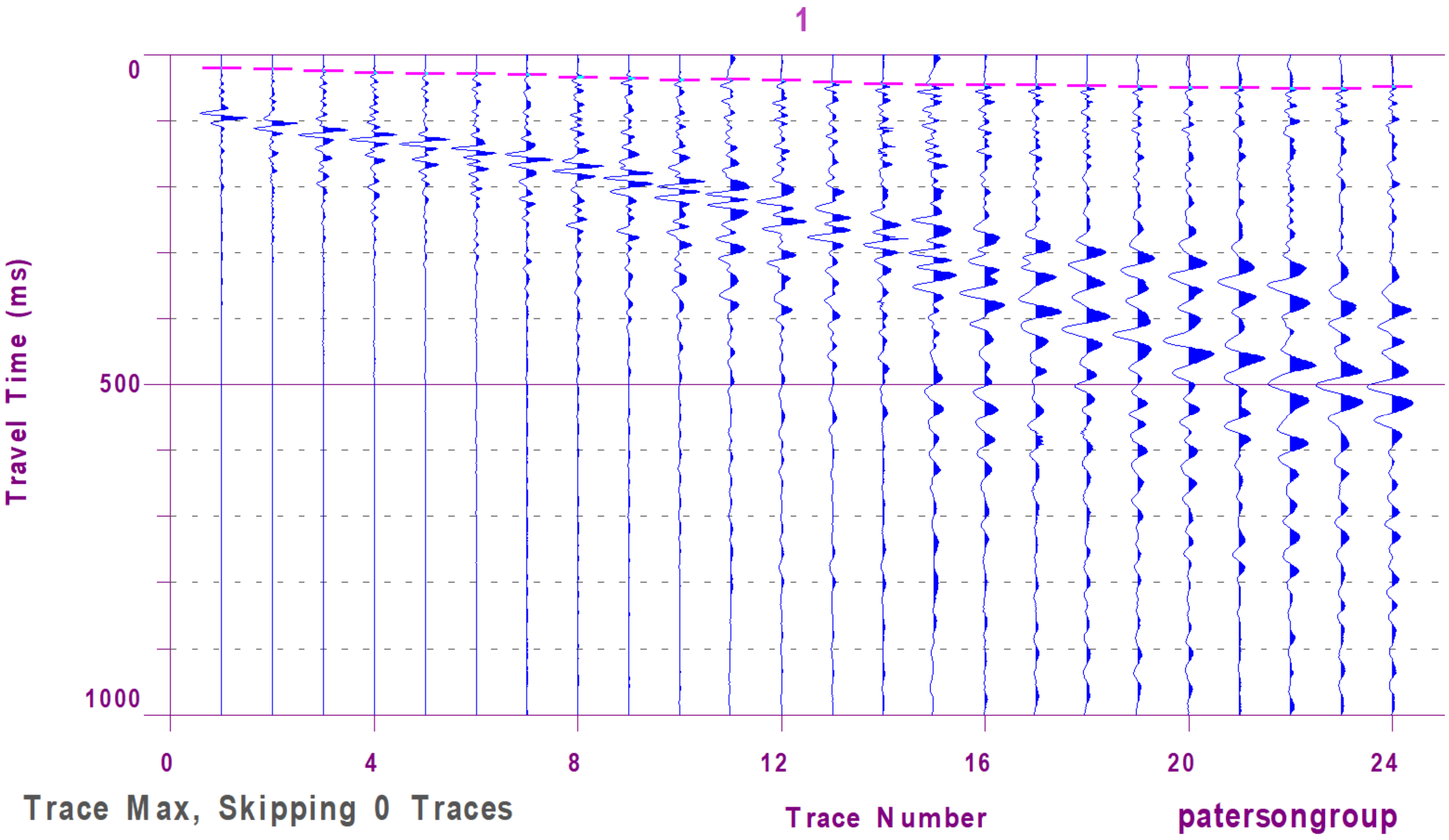


Figure 2 – Shear Wave Velocity Profile at Shot Location -15 m

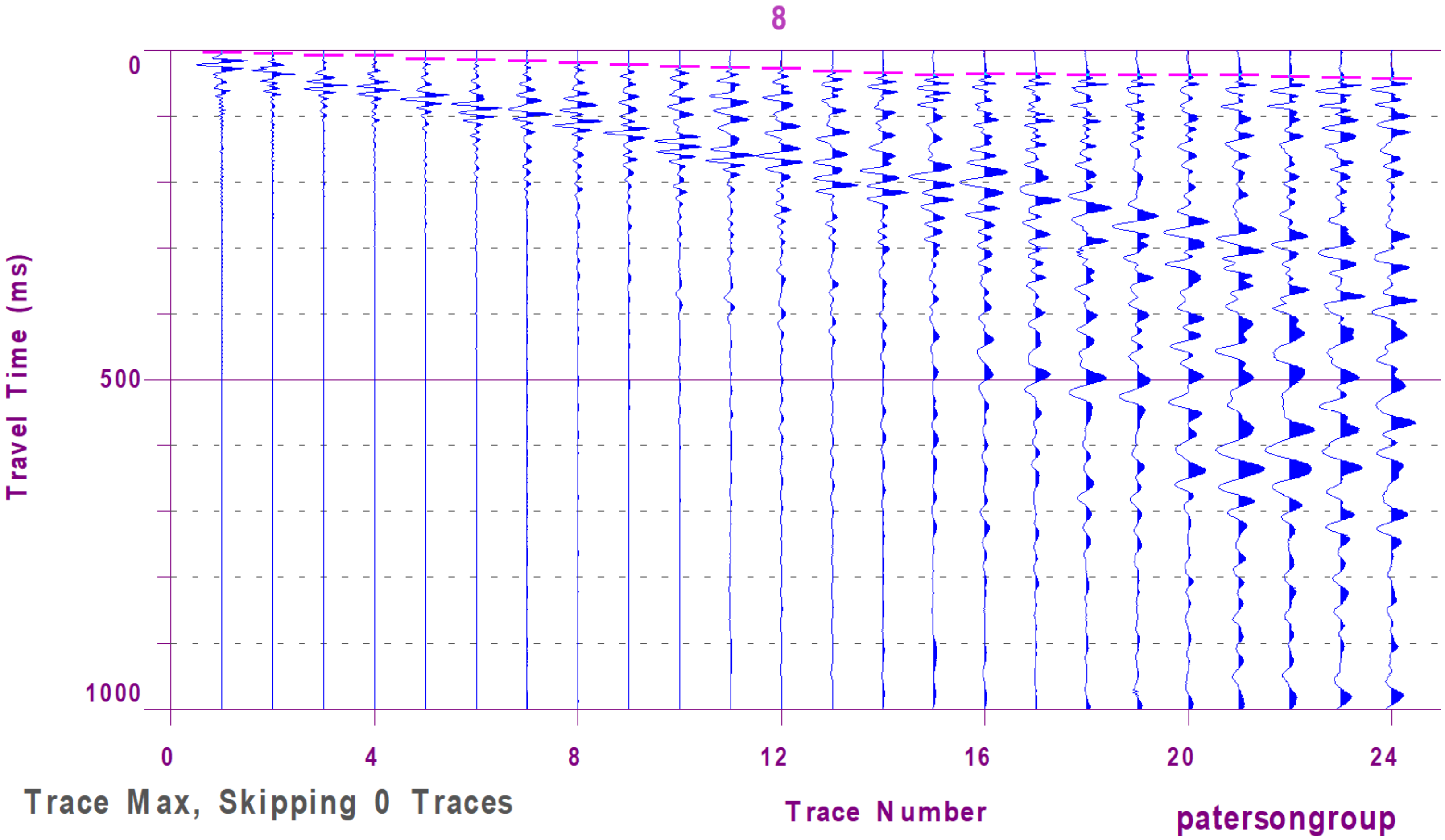
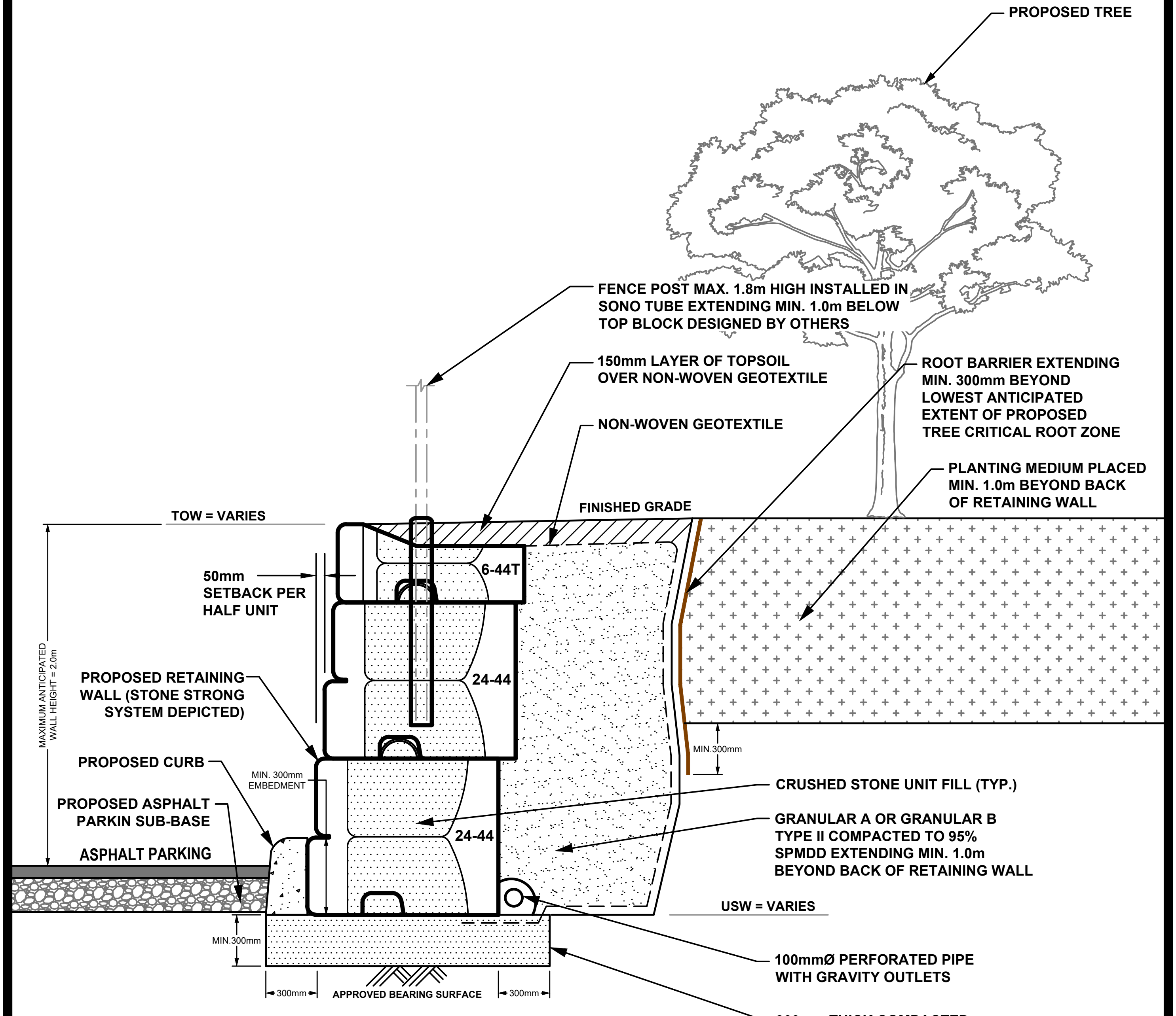


Figure 3 – Shear Wave Velocity Profile at Shot Location -3 m

PATERSON GROUP



PATERSON GROUP

IRONCLAD DEVELOPMENTS INC.

PROPOSED RESIDENTIAL DEVELOPMENT
475 TERRY FOX DRIVE
OTTAWA, ONTARIO

Title:

TYPICAL RETAINING WALL CROSS-SECTION

Date: **10/2025**

Report No.: **PG7418**

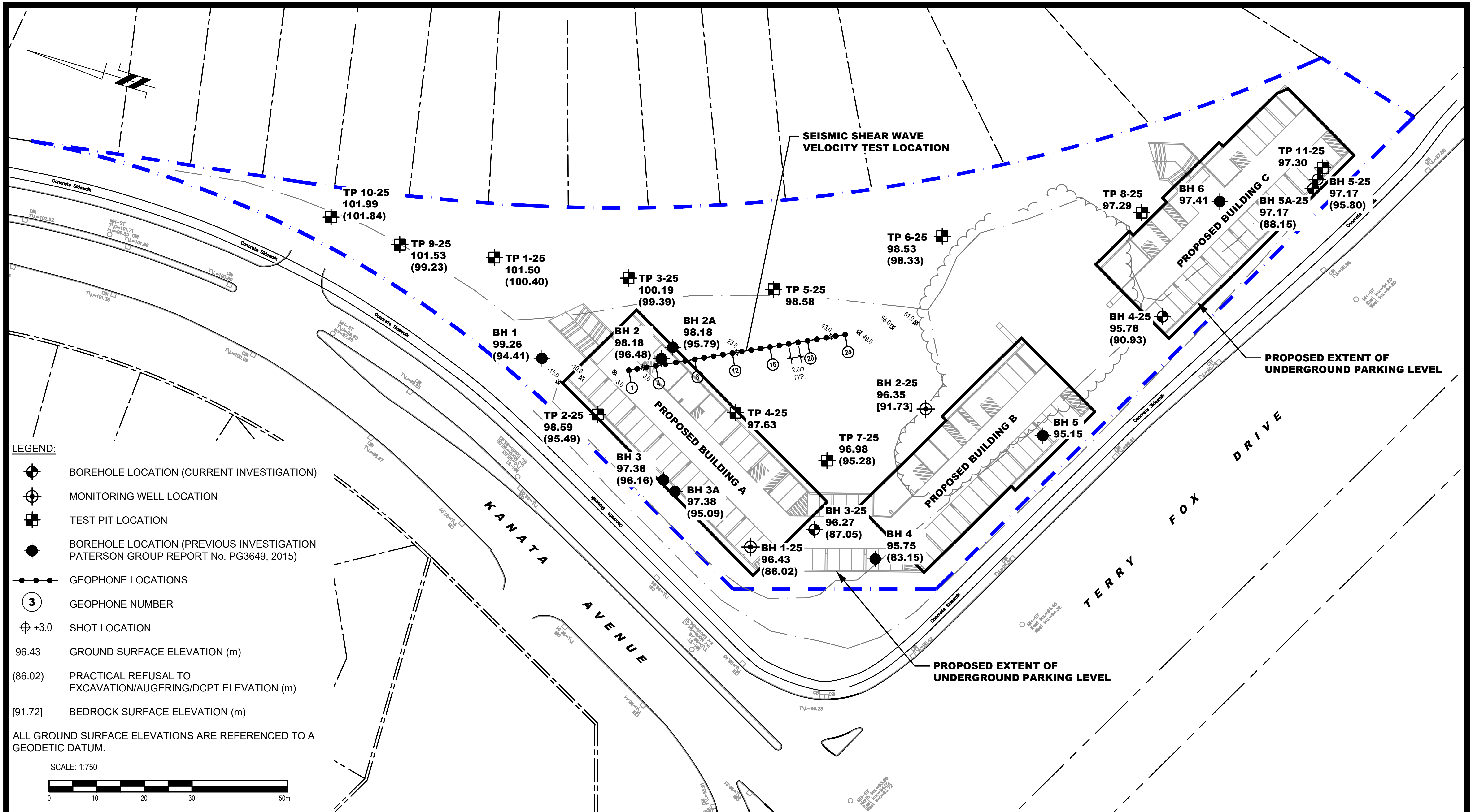
Scale: **1:35**

Drawing No.: **FIGURE 4**

Drawn by: **NFRV**

Checked by: **DP**

p:\autocad drawings\geotechnical\pg74xx\pg7418\pg7418 typ ret section.dwg



LEGEND:

⊕ BOREHOLE LOCATION (CURRENT INVESTIGATION)

⊕ MONITORING WELL LOCATION

▨ TEST PIT LOCATION

● BOREHOLE LOCATION (PREVIOUS INVESTIGATION PATERSON GROUP REPORT No. PG3649, 2015)

●-●-● GEOPHONE LOCATIONS

③ GEOPHONE NUMBER

⊕ +3.0 SHOT LOCATION

96.43 GROUND SURFACE ELEVATION (m)

(86.02) PRACTICAL REFUSAL TO EXCAVATION/AUGERING/DCPT ELEVATION (m)

[91.72] BEDROCK SURFACE ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:750

0 10 20 30 50m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

IRONCLAD DEVELOPMENTS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY DEVELPOMENT
475 TERRY FOX DRIVE

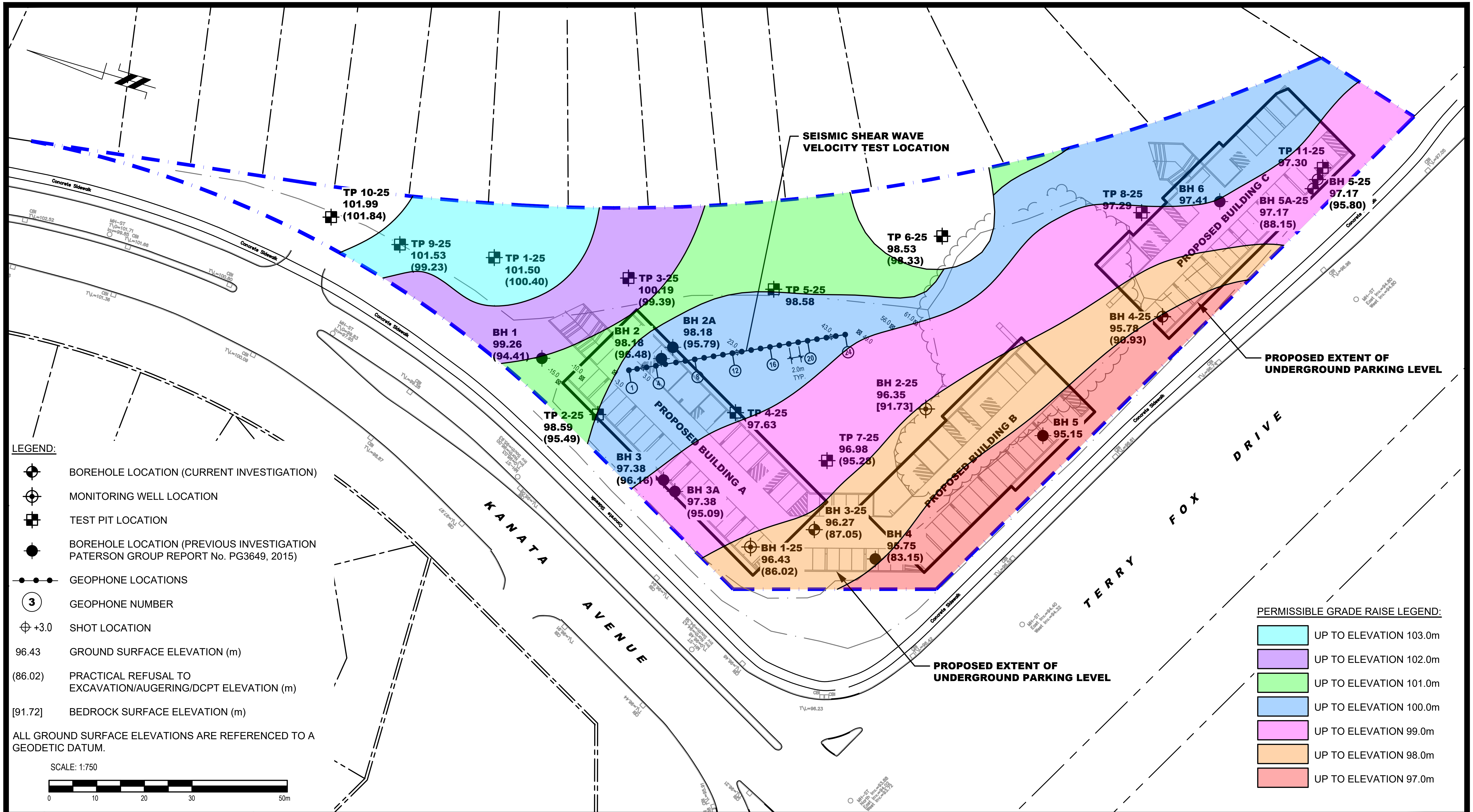
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 02/2025
Drawn by: NFRV
Report No.: PG7418-1
Checked by: NFRV
Dwg. No.: PG7418-1
Approved by: DP
Revision No.:

p:\autocad drawings\geotechnical\pg74xx\pg7418\pg7418-1 thlp.dwg



LEGEND:

⊕	BOREHOLE LOCATION (CURRENT INVESTIGATION)
⊕	MONITORING WELL LOCATION
▣	TEST PIT LOCATION
●	BOREHOLE LOCATION (PREVIOUS INVESTIGATION PATERSON GROUP REPORT No. PG3649, 2015)
●—●—●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⊕ +3.0	SHOT LOCATION
96.43	GROUND SURFACE ELEVATION (m)
(86.02)	PRACTICAL REFUSAL TO EXCAVATION/AUGERING/DCPT ELEVATION (m)
[91.72]	BEDROCK SURFACE ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:750

0 10 20 30 50m

PERMISSIBLE GRADE RAISE LEGEND:

🟦	UP TO ELEVATION 103.0m
🟪	UP TO ELEVATION 102.0m
🟩	UP TO ELEVATION 101.0m
🟦	UP TO ELEVATION 100.0m
🟪	UP TO ELEVATION 99.0m
🟧	UP TO ELEVATION 98.0m
🟥	UP TO ELEVATION 97.0m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

IRONCLAD DEVELOPMENTS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
475 TERRY FOX DRIVE

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:750
Date: 02/2025
Drawn by: NFRV
Report No.: PG7418-1
Checked by: NFRV
Dwg. No.: PG7418-2
Approved by: DP
Revision No.:

p:\autocad drawings\geotechnical\pg74xx\pg7418\pg7418-1 thlp.dwg