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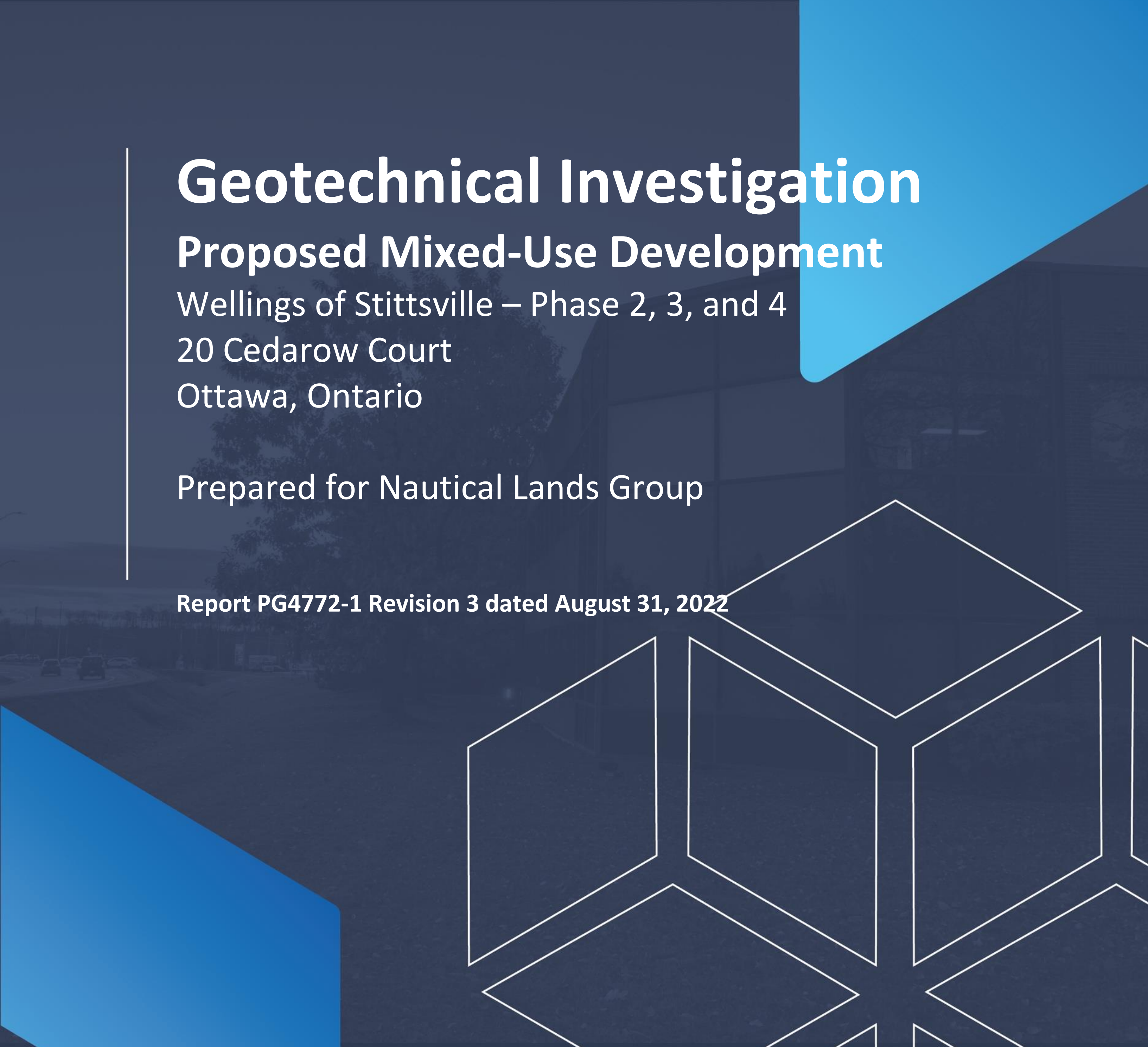

Geotechnical Investigation

Proposed Mixed-Use Development

Wellings of Stittsville – Phase 2, 3, and 4
20 Cedarow Court
Ottawa, Ontario

Prepared for Nautical Lands Group

Report PG4772-1 Revision 3 dated August 31, 2022

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Nautical Lands Group to conduct a geotechnical investigation for the proposed mixed-use development to be located at 20 Cedarow Court, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

☐ Determine the subsurface conditions by means of boreholes.

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

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

2.0 Proposed Development

Based on the available drawings, it is our understanding that the proposed development will consist of four, six (6) storey mixed-use buildings with a shared underground parking level occupying the majority of the footprint of the subject site. The buildings are understood to include retail, office space and residential units. Associate at-grade parking areas, access lanes, amenity and landscaped areas are also anticipated as a part of the development. It is also anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out from January 14, 2019 to January 18, 2019. At that time, 29 boreholes were drilled to a maximum depth of 4 m below existing grade.

A supplemental field investigation was conducted on February 2, 2022. At that time, 3 boreholes were advanced to the bedrock surface and cored to a maximum depth of 3.2 m into the bedrock surface.

The borehole locations were distributed in a manner to provide general coverage of the proposed development. The locations of the boreholes are shown in Drawing PG4772-1 – Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from a 50 mm diameter split-spoon or the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are presented as SS and AU, respectively, on the Soil Profile and Test Data sheets.

Standard Penetration Tests (SPT) were conducted and 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 sample 300 mm into the soil after the initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength tests were conducted in cohesive soil with a field vane apparatus.

Rock core samples were recovered from boreholes BH 1-22, BH 2-22 and BH 3-22 drilled during the supplemental investigation using a core barrel and diamond drilling techniques.

The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quantity Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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

Groundwater

Flexible polyethylene standpipes were installed in the majority of the boreholes to permit groundwater results subsequent to the sampling program completion. Monitoring wells were installed in BH 4, BH 9, BH 15, BH 22, and BH 27 to provide general site coverage as part of our hydrogeological study. The groundwater observations are discussed in Subsections 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All rock core samples from the supplemental investigation will be stored in the laboratory for a period of one month after issuance of this report at which time the samples will be discarded unless otherwise directed.

3.2 Field Survey

The borehole locations were selected by Paterson taking into consideration site features. The ground surface at the test pit locations were located and surveyed by Annis, O'Sullivan, Vollebekk LTD. It is understood that the ground surface elevations at the borehole locations were referenced to a geodetic datum. The locations and ground surface elevation at the boreholes are presented on Drawing PG4772-1 – 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 logs.

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 for sulphate attacks against subsurface concrete structures. The sample was tested to determine the concentration of sulphate and chloride, and the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and grassed covered with a tree-line located along the west boundary line of Cedarow Court. The ground surface across the site is relatively flat and approximately 1 m lower than adjacent properties and Hazeldean Road. Poole Creek ravine runs along the western border of the subject site approximately 3 m below the subject site.

The subject site is bordered by an active construction site for Phase 1 of the Wellings of Stittsville development along the north, Hazeldean Road along the east, and commercial building at the edge of Cedarow Court along the south.

4.2 Subsurface Profile

Overburden

The subsurface profile at the borehole locations consists of topsoil overlying a hard to very stiff silty clay crust followed by a grey, very stiff to stiff silty clay layer. Glacial till was encountered below the silty clay layer consisting of compact silty sand to sandy silt with clay, gravel, cobbles and boulders. A deposit of very stiff to hard clayey silt was encountered below the topsoil in BH 17, BH 18, BH 24, BH 25, BH 26, and BH 27. Practical refusal to augering on inferred bedrock was encountered in all borehole depths ranging between 1.6 to 4.0 m. Specific details to the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets provided in Appendix 1.

Bedrock

Bedrock was cored in 3 boreholes to a maximum depth of 3.2 m below the bedrock surface. The bedrock in borehole BH 1-22 was observed to have an RQD value of 100%. This is indicative of a fair to excellent quality bedrock. The average RQD value in boreholes BH 2-21 was generally between 82 and 100% which is an indicative of good to excellent quality bedrock. The upper portion of the bedrock in borehole BH 3-21 had an RQD value of 64%, indicative of fair quality bedrock whereas the remainder of the bedrock was found to be in good to excellent quality. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at borehole location.

Based on available geological mapping, the subject site consists of interbedded dolostone and limestone of the Gull River formation and an approximate drift thickness of 2 to 15 m.

4.3 Groundwater

The measured groundwater levels at the borehole locations are presented in Table 1. Groundwater readings recorded in flexible piezometers could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater level can also be estimated based on observations of the recovered soil samples, such as the moisture level, soil consistency and colouring. Based on these observations, the long-term groundwater level is anticipated at a depth ranging between 2.5 to 3.5 m below existing grade. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

Table 1 – Groundwater Readings Summary

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	104.37	DRY	n/a	January 29, 2019
BH 2	103.59	3.05	100.54	January 29, 2019
BH 3	103.55	1.81	101.74	January 29, 2019
BH 4	103.28	3.05	100.23	January 29, 2019
BH 5	103.45	3.05	100.40	January 29, 2019
BH 6	103.49	3.04	100.45	January 29, 2019
BH 7	103.41	DRY	n/a	January 29, 2019
BH 8	103.46	DRY	n/a	January 29, 2019
BH 9	103.42	3.17	100.25	January 29, 2019
BH 10	103.31	2.18	101.13	January 29, 2019
BH 11	103.44	DRY	n/a	January 29, 2019
BH 12	103.58	DRY	n/a	January 29, 2019
BH 13	103.55	DRY	n/a	January 29, 2019
BH 14	104.18	DRY	n/a	January 29, 2019
BH 15	103.65	2.92	100.73	January 29, 2019
BH 16	103.66	DRY	n/a	January 29, 2019
BH 17	104.19	DRY	n/a	January 29, 2019
BH 18	104.15	DRY	n/a	January 29, 2019
BH 19	103.78	DRY	n/a	January 29, 2019
BH 20	103.59	DRY	n/a	January 29, 2019
BH 21	103.58	DRY	n/a	January 29, 2019
BH 22	103.65	DRY	n/a	January 29, 2019
BH 23	103.87	2.62	101.25	January 29, 2019
BH 24	104.04	2.55	101.49	January 29, 2019
BH 25	104.07	1.68	102.39	January 29, 2019
BH 26	104.30	DRY	n/a	January 29, 2019
BH 27	103.97	DRY	n/a	January 29, 2019
BH 28	103.78	DRY	n/a	January 29, 2019
BH 29	103.71	DRY	n/a	January 29, 2019

Note: The ground surface elevation at the borehole locations was provided by Annis, O’Sullivan, Vollebekk Ltd.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed structures will be founded on conventional shallow foundations placed on an undisturbed, hard to very stiff silty clay, compact to dense glacial till and/or clean, surface sounded bedrock bearing surface. Alternatively, conventional shallow footings can be placed over a near vertical, zero entry, concrete in-filled trenches extending to a clean, surface sounded bedrock bearing surface.

Permissible grades raise restriction areas are also required due to the silty clay deposit. A permissible grade raise restriction of **2 m** is recommended for areas where settlement sensitive structures are founded over the silty clay deposit.

Depending on the extent of the underground parking garage and potential grade raise, the bedrock may be encountered during excavation and construction. All contractors should be prepared for bedrock removal within the subject site.

Prior to considering blasting operations, if required, 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 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/claims related to the blasting operations.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

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.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or 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 proximity of the blasting operations should be completed 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/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing 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 excavated almost vertical side walls. A minimum 1 m horizontal ledge should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

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 equipments 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.

Fill Placement

Fill placed for grading beneath the structure(s) or other settlement sensitive areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The engineered fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill materials should be spread in thin lifts and at a minimum compacted by the tracks of the spreading equipment to minimize voids. If the non-specified backfill is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values (Shallow Foundation)

Footings for proposed buildings can be designed with the following bearing resistance values presented in Table 2.

Table 2 – Bearing Resistance Values

Bearing Surface	Bearing resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Very stiff to hard silty clay	150	250
Compact to dense glacial till	200	300
Weathered Limestone Bedrock	-	1500
Clean, Surface Sounded Limestone Bedrock	-	2000
Lean Concrete In-filled Trenches	-	2000

Note – Strip footings, up to 3 m wide, and pad footings, up to 8 m wide, placed over an undisturbed, silty clay bearing surface can be designed using the above noted bearing resistance values. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The above-noted bearing resistance values at SLS for soil bearing surfaces 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 values provided herein will be subjected to negligible potential post-construction total and differential settlements.

The bearing resistance values are provided on the assumption that the footings are placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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.

Lean Concrete Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (15 MPa 28-day compressive strength). 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 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.

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 a 1.5 m depth). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, 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. Also, 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 0.5 m. 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.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an engineered fill, stiff silty clay or glacial till above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

Permissible Grade Raise Restriction

Based on the current borehole information, a **permissible grade raise restriction of 2 m** is recommended for the proposed buildings and settlement sensitive structures were founded over a silty clay deposit. A post-development groundwater lowering of 0.5 m was assumed for our calculations.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as Class C for the foundations considered at this site. However, a higher site class, such as Class A or B can be provided if a site specific shear wave velocity test is completed to confirm the seismic site classification. The soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

The basement area for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the proposed structure's basement walls. 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 dry unit weight of 20 kN/m^3.

The foundation wall is anticipated to be provided with a perimeter drainage system; therefore, the retained soils should be considered drained. For the undrained conditions, the applicable effective unit weight of the retained soil can be designed with13 kN/m^3. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

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

Static Conditions

The static horizontal earth pressure (po) could be calculated with 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 with a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above formula for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure should only be applicable for static analyses and not be calculated 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 Conditions

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}) could be calculated using $0.375 \cdot ac \cdot \gamma \cdot H2/g$ 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 0.32g according to OBC 2012. The vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions could be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions presented above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car only parking areas and access lanes, if required.

Table 3 - Recommended Flexible Pavement Structure – At-Grade 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 – In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

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

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be sub-excavated and replaced with OPSS Granular Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundational Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed structures. The composite drainage system (such as Miradrain G100N, Delta Drain 6000 or an approved equivalent) is recommended to extend to the footing level. Sleeves, 150 mm diameter, at 3 m centres are recommended to be placed in the footing or at the foundation wall/footing interface for blind sided pours to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage is recommend to control water infiltration for the proposed structures. For design purposes, Paterson recommends 150 mm diameter PVC, corrugated, perforated pipes be placed at 3 to 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Adverse Effects of Dewatering on Adjacent Properties

Due to the low permeability of the subsoils profile, any dewatering will be considered relatively minor as a result of the proposed construction. Therefore, adverse effects to the surrounding buildings or properties are not expected with respect to any groundwater lowering.

Foundational 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 where frost heave sensitive structures, such as a concrete sidewalk, will be placed. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material may be used for this purpose. A composite drainage system, such as Delta Drain 6000, Miradrain G100 or an approved equivalent, should be placed against the foundation wall to promote drainage toward the perimeter drainage pipe.

6.2 Protection of Footings Against Frost Action

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

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

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

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

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

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 excavation depths, 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 designer 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 sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 6 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m_3	20
Effective 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 are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock 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 and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of a 150 mm layer of OPSS Granular A crushed stone should be placed for pipe bedding for sewer and water pipes for a soil subgrade. The bedding thickness should be increased to 300 mm for areas where the subgrade consists of bedrock. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the SPMDD.

The site excavated material may be placed above cover material if the excavation operations are completed in dry weather conditions and the site excavated material is approved by the geotechnical consultant. All cobbles greater than 200 mm in the longest dimension should be removed prior to the site materials being reused.

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 differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD. Within the frost zone (1.8 m below finished grade), non-frost susceptible materials should be used when backfilling trenches below the original bedrock level.

Clay seals are recommended for the subject site. The seals should be a minimum of 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries, roadway intersections and at a maximum distance of every 50 m in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-term Groundwater Control

Any groundwater encountered along the buildings' perimeter or sub-slab drainage system will be directed to the proposed buildings' cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the expected long-term groundwater flow should be low (i.e. less than 25,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

Impacts on Neighbouring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. It should be noted that the neighbouring multi-storey buildings are expected to be founded over the bedrock surface and would not be affected by the short-term groundwater lowering during construction. The water table is located within the glacial till layer and/or bedrock surface. Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures or City infrastructures.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. 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 in the contract documents should be provided to protect the excavation walls from freezing, if applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means. The excavation base should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

6.7 Corrosion Potential and Sulphate

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

6.8 Limit of Hazard Lands

Field Observations

Paterson conducted a site visit on January 13, 2019 to review the slope located along the west boundary of the subject site, assess the current slope conditions and confirm the grades provided in the existing topographic mapping. A section of Poole Creek is located within the west portion of the site and shown in Drawing PG4772-1 - Test Hole Location Plan.

Three (3) slope cross-sections were reviewed in the field as the worst case scenarios. The cross section locations are presented on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2. Generally, the riverbanks along both sides of Poole Creek are currently well vegetated and were observed in an acceptable condition. Poole Creek was observed within a 20 to 40 m wide flood plain. The slope along the east side of Poole Creek ranged in height between 3 and 5 m with an inclination ranging between 2.3H:1V and 3.3H:1V. The upper slope was observed to be well vegetated with little to no signs of active surficial erosion.

Slope Stability Analysis

Limit of Hazard Lands
The slope condition was reviewed based on available topographic mapping along the east side slopes of Poole Creek within the west portion of the subject development. A total of 3 slope cross-sections were assessed as the worst-case scenarios. The cross-section locations are presented on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2.

A slope stability assessment was carried out to determine the required stable slope allowance setback from the top of slope based on a factor of safety of 1.5. A toe erosion and 6 m erosion access allowances were also included in the determination of limits of hazard lands and are discussed below. The proposed limit of hazard lands (as shown on Drawing PG4772-1 - Test Hole Location Plan) includes:

☐ a geotechnical slope stability allowance with a factor of safety of 1.5
☐ a toe erosion allowance
☐ a 6 m erosion access allowance and top of slope

Slope Stability Analysis

The analysis of the stability of the slope sections 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 than 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.

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16G was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The cross-sections were analysed taking into account a groundwater level at ground surface, which represents a worse-case scenario that can be reasonably expected to occur in cohesive soils. The stability analysis assumes full saturation of the soil with groundwater flow parallel to the slope face. Subsoil conditions at the cross-sections were inferred based on the findings at borehole locations along the top of slope and general knowledge of the area's geology.

Stable Slope Allowance

The results of the stability analysis for static conditions at Sections A through C are presented in Figures 2A to 4A in Appendix 2. All the reviewed slope sections along the subject creek were noted to be shaped to at least a 2.3H:1V. Based on the soil conditions observed and the results of the slope stability analysis, the slope stability factor of safety was calculated to be 1.5 or greater for all the slope sections which indicates that a stable slope allowance is not required for the subject slope.

The results of the analyses including seismic loading are shown in Figures 2B to 4B for the slope sections. The results indicate that the factor of safety for the sections are greater than 1.1.

It should be noted that the existing vegetation on the slope face should not be removed as it contributes to the stability of the slope and reduces erosion. If the existing vegetation needs to be removed, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed and/or topped with an erosion control blanket be which can be placed across the exposed slope face.

Toe Erosion and Erosion Access Allowance

The toe erosion allowance for the valley corridor wall slope was based on the cohesive nature of the top layers of the subsoils, the observed current erosional activities and the width and location of the current watercourse. It should be noted that if the flood plain is measured to be greater than 20 m, no toe erosion will be required. Therefore, based on the above factors, no toe erosion allowance is considered for the subject slope.

An erosion access allowance of 6 m is required from the top of slope to ensure access is provided should future maintenance to the slope face is required. The limit of hazard lands, which includes these allowances, is indicated on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2.

Proposed Conditions

An analysis was conducted following a review of the proposed grade raise and development. It is understood that storm water storage tanks are proposed on the north portion of the site. The proposed conditions are presented in Figure 2C, 3C and 4C in Appendix 1. Following a review of the proposed conditions, the slope will not be impacted by the proposed development.

6.9 Landscaping Considerations

Tree Planting Restrictions

According to the City of Ottawa Guidelines for tree planting, where a sensitive silty clay deposit is present within the vicinity of the site, tree planting restrictions should be determined. However, for this site, based on the founding medium of the underground parking level which will occupy the majority of the site, tree planting restrictions are not required from a geotechnical perspective.

6.10 Storm water storage tanks

Based on existing servicing drawings, it is understood that storm water storage tanks are proposed along the north portion of the site. The tanks are approximately 3.6 m wide, 3.6 m deep and 9.1 m long. The tanks are expected to be fully buried with an invert level of approximately 99.7 m with a soil cover of approximately 1 m above the top of the tanks. Frost protection will not be required due to the founding depth.

Due to the depth of the storm tanks and the estimated depth of the groundwater table, the tanks should be waterproofed up to 1 m above the estimated groundwater table. It should be noted that the fill placed around and above the water tanks will provide sufficient resistance to the expected buoyancy forces resulting from the long-term groundwater table. Therefore, the proposed water storage tanks are considered acceptable from a geotechnical perspective.

Due to the expected founding depth, the tanks should be installed on OPSS Granular A or Granular B Type II extending to the bedrock surface and compacted to a minimum 98% of the material's SPMDD.

Reference should be made to subsection 6.8 for slope stability analysis and limit of hazard lands setback for the development. The setback from the top of slope and the tanks is sufficient, therefore, the slope will not be negatively impacted by the proposed storm water storage tanks.

7.0 Recommendations

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

☐ Review detail grading plan(s) from a geotechnical perspective.

☐ Review groundwater conditions at the time of construction to determine if waterproofing is required.

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

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

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

A geotechnical investigation is a limited sampling of a site. Should any conditions encountered during construction differ from the borehole locations, Paterson requests immediate notification to permit reassessment of the recommendations provided herein.

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

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 Nautical Lands Group or their agent(s) is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

O.C

Owen Canton, E.I.T

LICENSED PROFESSIONAL ENGINEER
Aug. 31, 2022
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. About-Seido, P.Eng.

Report Distribution:

❑ Nautical Lands Group (3 copies)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4772

REMARKS

HOLE NO. BH1-22

BORINGS BY CME 55 Power Auger

DATE February 2, 2022

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.32
						1	102.32
OVERBURDEN						2	101.32
						3	100.32
3.73							
						4	99.32
		RC	1	100	100		
						5	98.32
BEDROCK							
						6	97.32
		RC	2	100	100		
6.99							
End of Borehole							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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Ottawa, Ontario

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FILE NO. **PG4772**

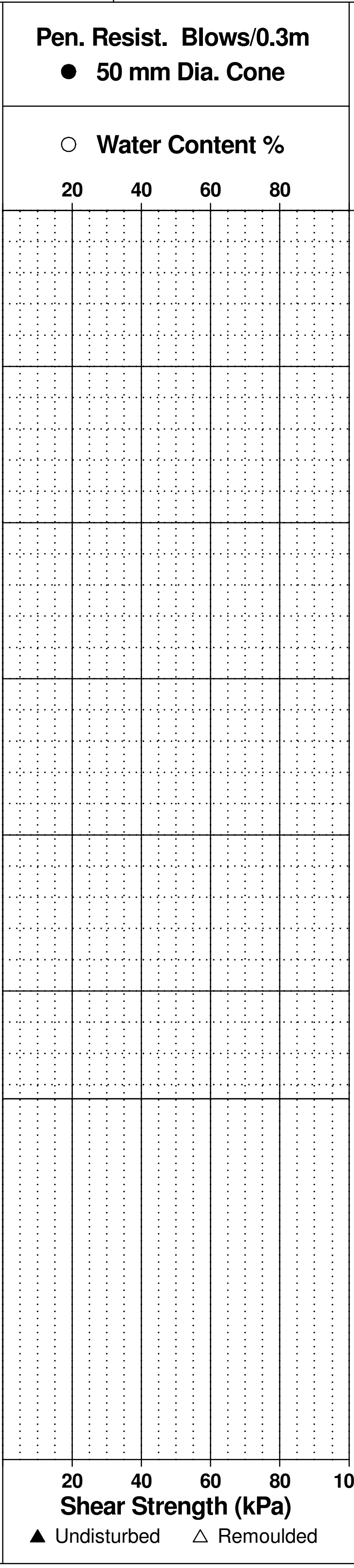
REMARKS

HOLE NO. **BH2-22**

BORINGS BY CME 55 Power Auger

DATE February 2, 2022

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	103.34	
						1	102.34	
OVERBURDEN						2	101.34	
						3	100.34	
3.51								
BEDROCK	▤	RC	1	100	100	4	99.34	
	▤	RC	2	97	82	5	98.34	
5.72								
End of Borehole								



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4772

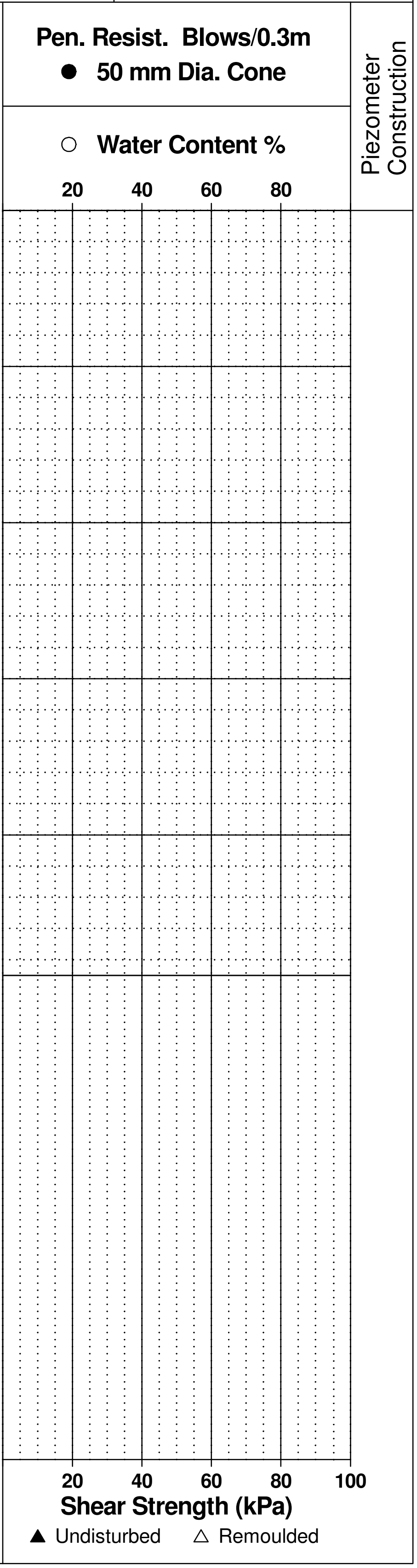
REMARKS

HOLE NO. BH3-22

BORINGS BY CME 55 Power Auger

DATE February 2, 2022

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.81
OVERBURDEN					1	102.81
2.29					2	101.81
BEDROCK	RC	1	100	64	3	100.81
	RC	2	100	95	4	99.81
	RC	3	97	97		
4.90 End of Borehole						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

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FILE NO. **PG4772**

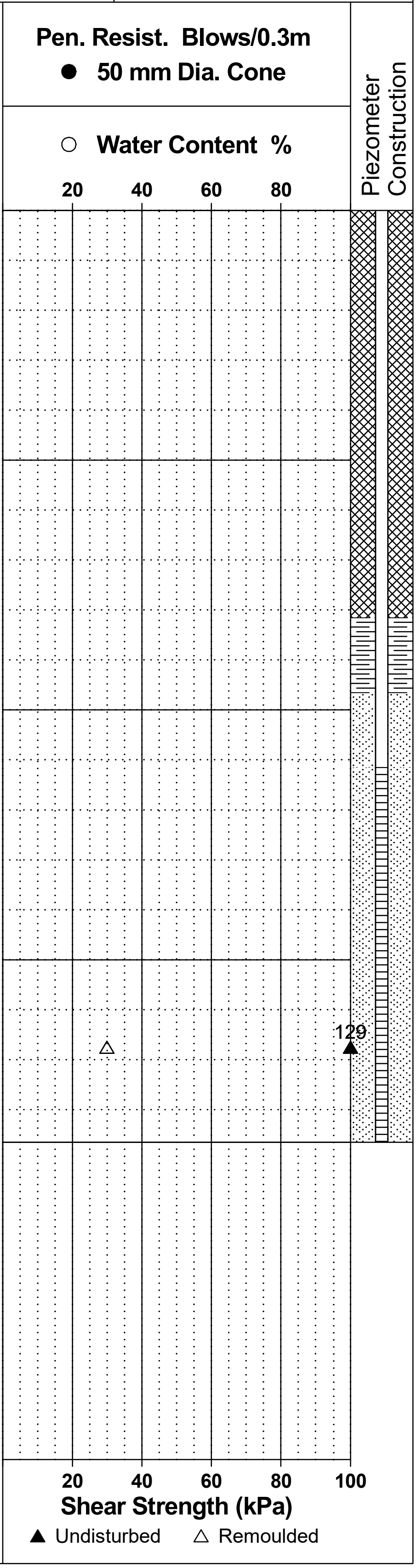
REMARKS

HOLE NO. **BH 1**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.37
FILL: Compact brown silty sand, some gravel	AU	1				
	SS	2	38	15	1	103.37
1.52						
Very stiff, brown **SILTY CLAY,** trace gravel	SS	3	42	7	2	102.37
	SS	4	58	4		
					3	101.37
3.73						
End of Borehole Practical refusal to augering at 3.73m depth (BH dry - Jan 29/19)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

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FILE NO. **PG4772**

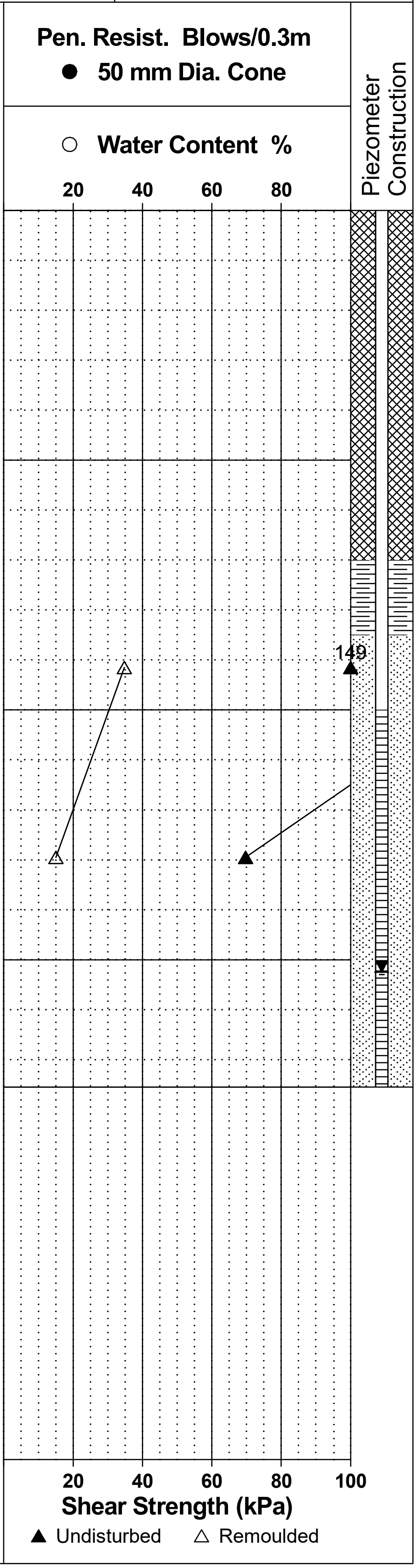
REMARKS

HOLE NO. **BH 2**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.59
FILL: Brown silty sand, some gravel		AU	1				
0.66							
		SS	2	33	4	1	102.59
Very stiff to stiff, brown **SILTY CLAY**							
						2	101.59
- grey and trace gravel by 3.0m depth						3	100.59
		SS	3		50+		
3.51							
End of Borehole Practical refusal to augering at 3.51m depth (GWL @ 3.05m depth - Jan 29/19)							



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SOIL PROFILE AND TEST DATA

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FILE NO. **PG4772**

REMARKS

HOLE NO. **BH 3**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

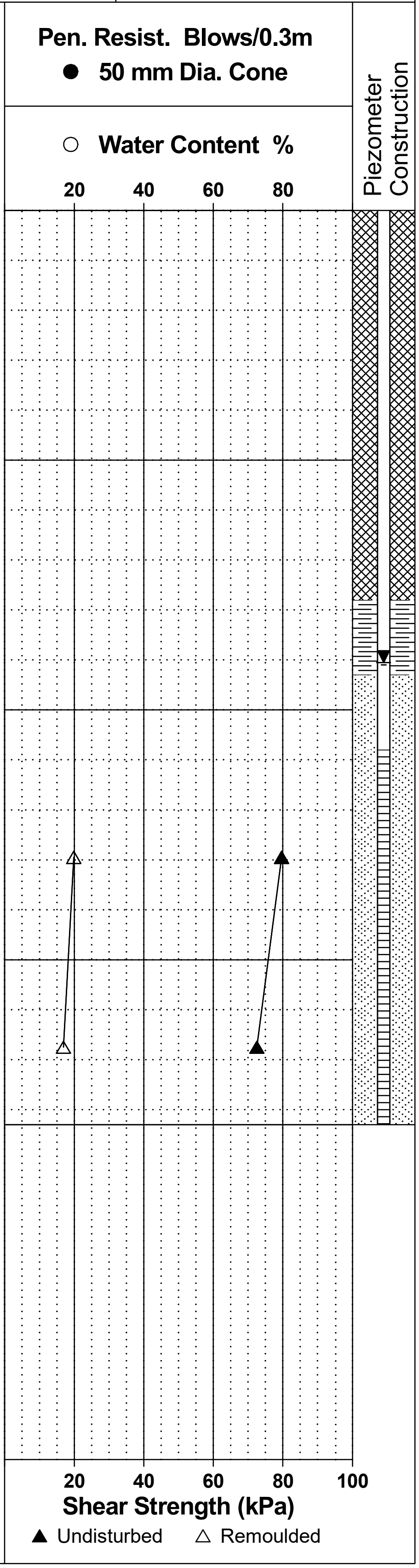
SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.33	AU	1		
Very stiff to stiff, brown **SILTY CLAY**	SS	2	21	7
	SS	3	62	7
- grey by 2.3m depth				
3.66				

DEPTH (m)	ELEV. (m)
0	103.55
1	102.55
2	101.55
3	100.55

End of Borehole

Practical refusal to augering at 3.66m depth

(GWL @ 1.81m depth - Jan 29/19)



SOIL PROFILE AND TEST DATA

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FILE NO. **PG4772**

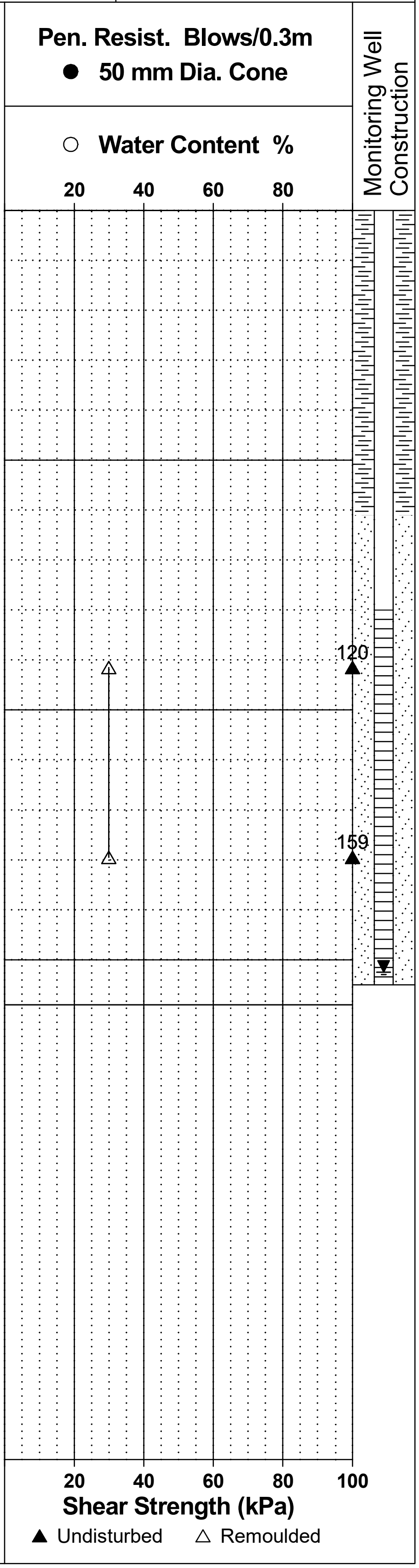
REMARKS

HOLE NO. **BH 4**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.28
TOPSOIL 0.30	AU	1				
Very stiff, brown **SILTY CLAY**	SS	2	25	6	1	102.28
					2	101.28
- grey by 2.4m depth - trace sand and gravel by 3.0m depth					3	100.28
3.18	SS	3	100	50+		
End of Borehole Practical refusal to augering at 3.18m depth (GWL @ 3.05m depth - Jan 29/19)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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FILE NO. **PG4772**

REMARKS

HOLE NO. **BH 5**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

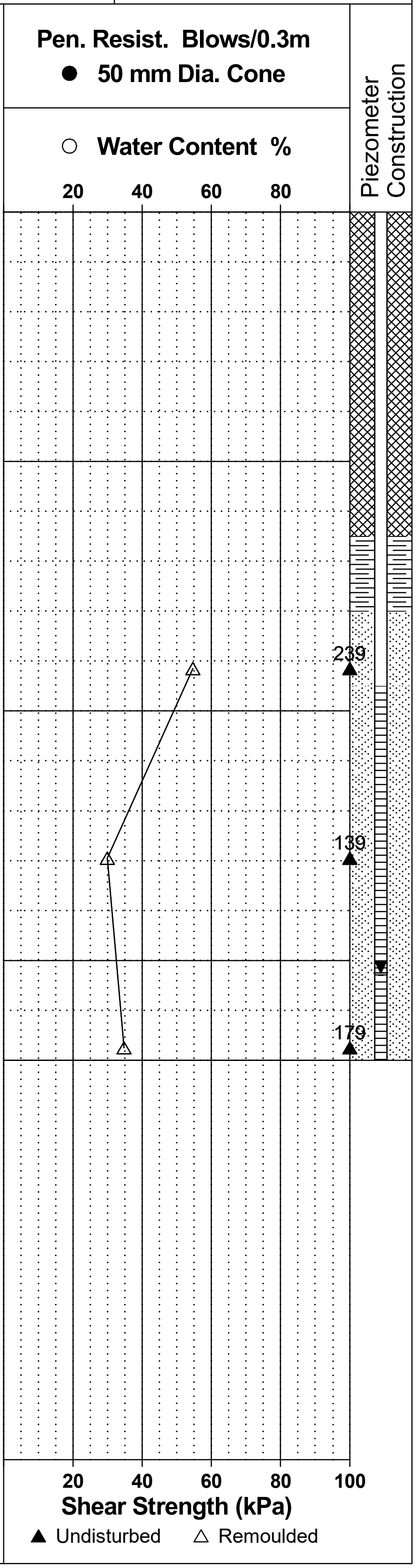
SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.36	AU	1		
Hard to very stiff, brown **SILTY CLAY**	SS	2	38	6
- grey by 2.1m depth				
3.40				

End of Borehole

Practical refusal to augering at 3.40m depth

(GWL @ 3.05m depth - Jan 29/19)

DEPTH (m)	ELEV. (m)
0	103.45
1	102.45
2	101.45
3	100.45



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

FILE NO. **PG4772**

HOLE NO. **BH 6**

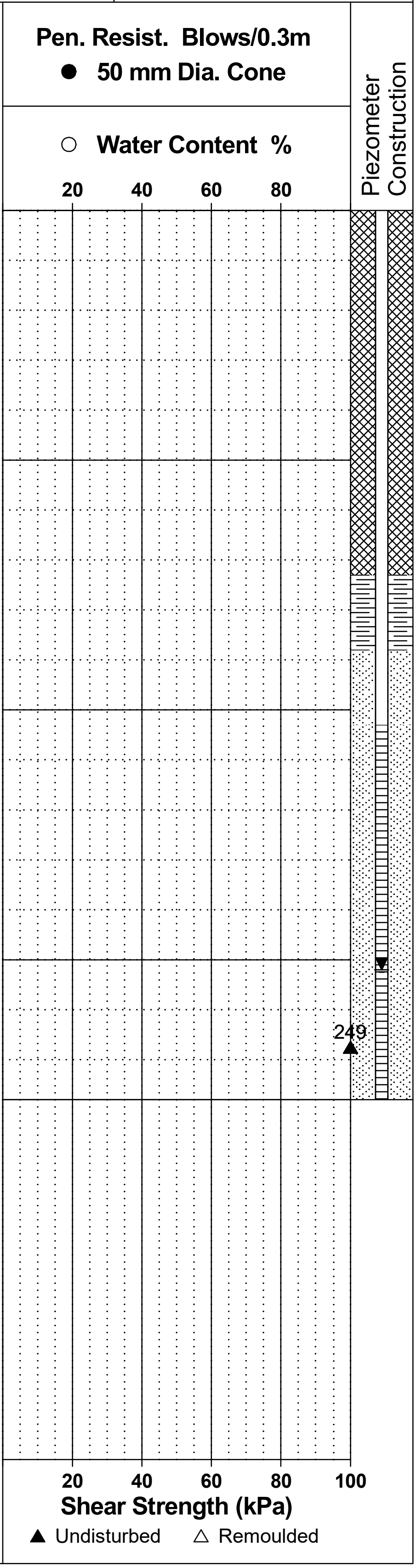
SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL (to 0.38)	AU	1		
Very stiff, brown **SILTY CLAY**	SS	2	58	8
	SS	3	71	9
- grey by 2.0m depth	SS	4	100	5
(to 3.56)				

DEPTH (m)	ELEV. (m)
0	103.49
1	102.49
2	101.49
3	100.49

End of Borehole

Practical refusal to augering at 3.56m depth

(GWL @ 3.04m depth - Jan 29/19)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

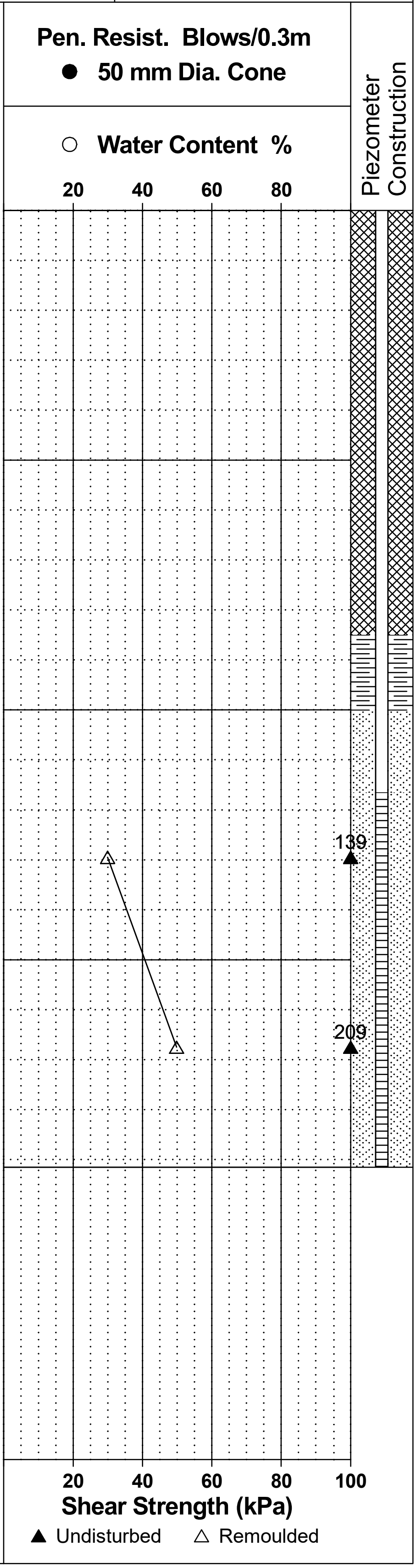
DATE 2019 January 14

FILE NO. PG4772

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.43		AU	1		
Very stiff to hard, brown **SILTY CLAY**		SS	2	58	7
- grey by 1.8m depth 3.83		SS	3	92	6
End of Borehole Practical refusal to augering at 3.83m depth (BH dry - Jan 29/19)					

DEPTH (m)	ELEV. (m)
0	103.41
1	102.41
2	101.41
3	100.41



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

FILE NO. PG4772

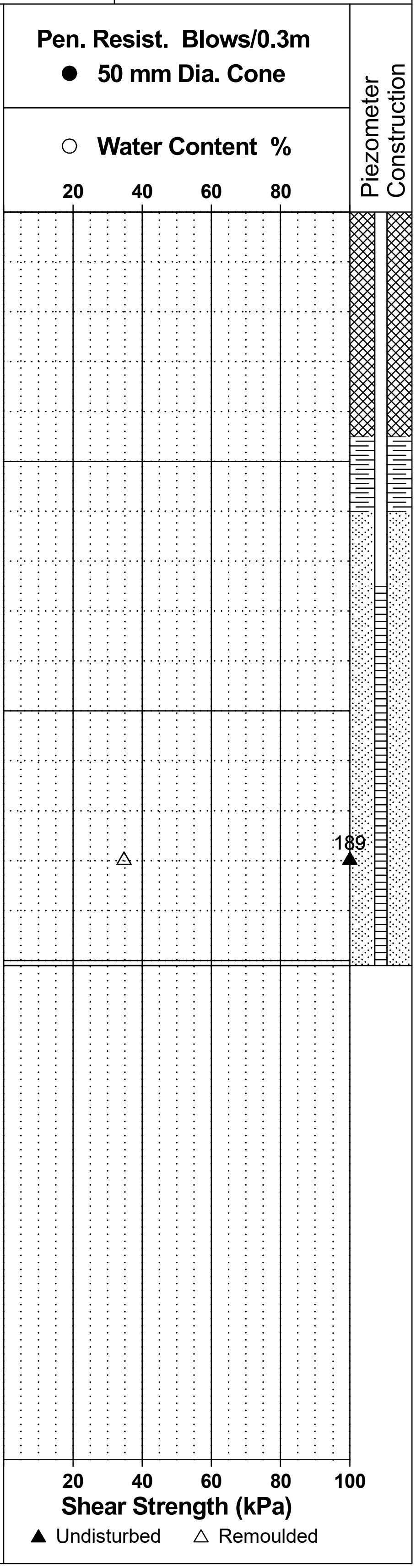
HOLE NO. BH 8

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.46
TOPSOIL (to 0.41)	AU	1				
Very stiff, brown **SILTY CLAY**	SS	2	67	7	1	102.46
	SS	3	92	6		
- grey by 2.0m depth					2	101.46
3.02					3	100.46

End of Borehole

Practical refusal to augering at 3.02m depth

(BH Dry - Jan 29/19)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

FILE NO. PG4772

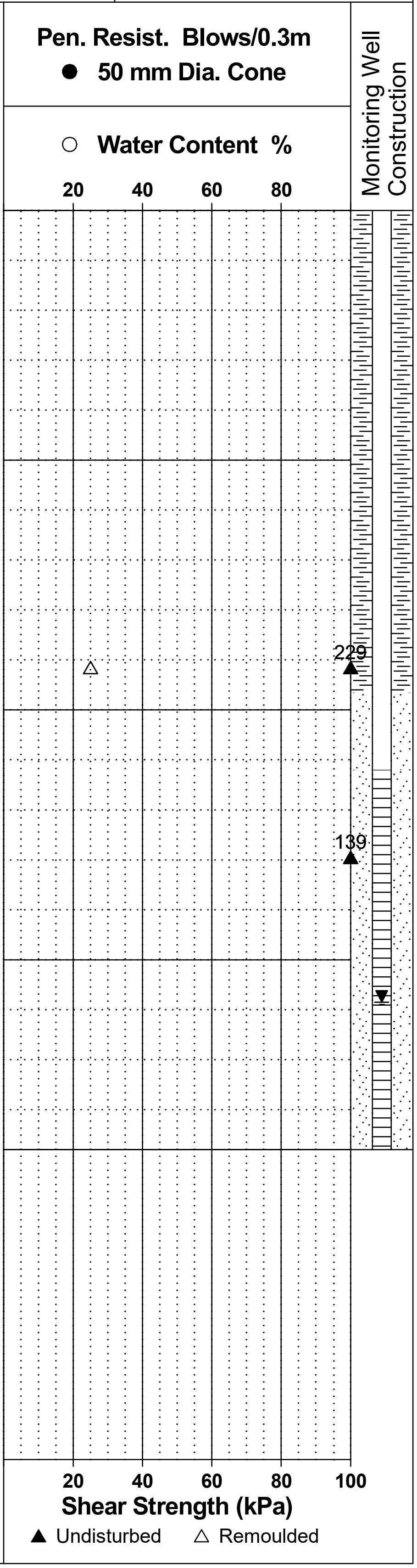
HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.42
TOPSOIL 0.38		AU	1				
Hard to very stiff, brown **SILTY CLAY**		SS	2	71	4	1	102.42
						2	101.42
						3	100.42
3.76		SS	3	71	14		

End of Borehole

Practical refusal to augering at 3.76m depth

(GWL @ 3.17 m depth - Jan 29/19)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4772

REMARKS

HOLE NO. BH10

BORINGS BY CME 55 Power Auger

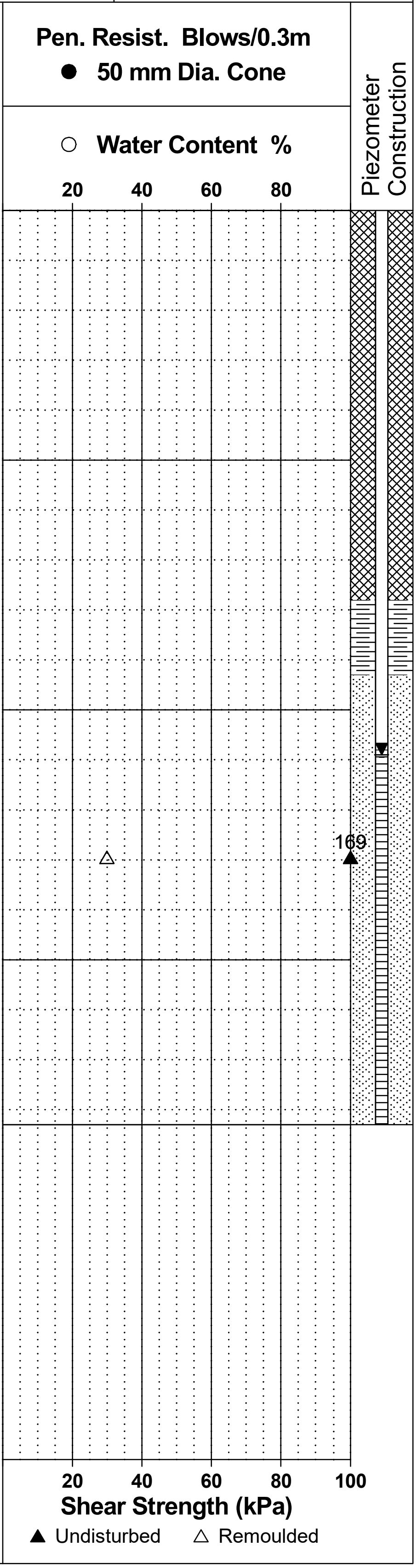
DATE 2019 January 15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.31
TOPSOIL		AU	1				
0.41							
Very stiff, brown **SILTY CLAY**		SS	2	67	9	1	102.31
		SS	3	75	6		
- grey by 2.1m depth						2	101.31
3.05						3	100.31
GLACIAL TILL: Compact, brown sandy silt, trace clay and gravel, occasional cobbles and boulders		SS	4	83	19		
3.66							

End of Borehole

Practical refusal to augering at 3.66m depth

(GWL @ 2.18m depth - Jan 29/19)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

FILE NO. PG4772

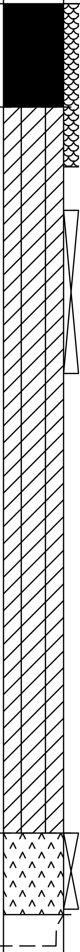
HOLE NO. BH11

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.38
Very stiff, brown **SILTY CLAY**	3.05
GLACIAL TILL: Very dense brown to grey sandy silt, trace clay and gravel, occasional cobbles and boulders	3.35

End of Borehole

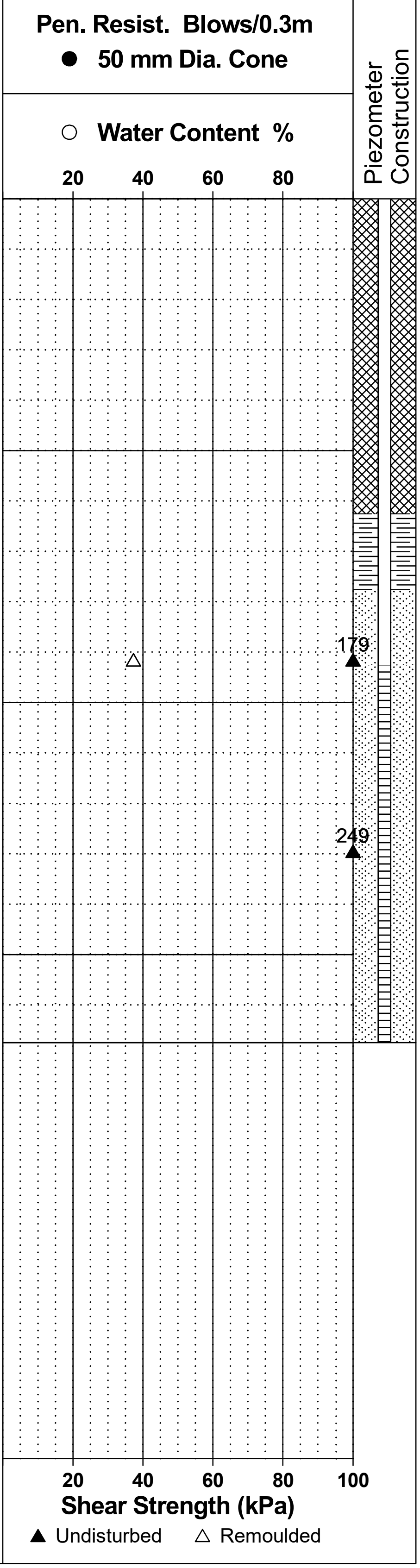
Practical refusal to augering at 3.35m depth

(BH Dry - Jan 29/19)



SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	71	4
SS	3	100	50+

DEPTH (m)	ELEV. (m)
0	103.44
1	102.44
2	101.44
3	100.44



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

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REMARKS

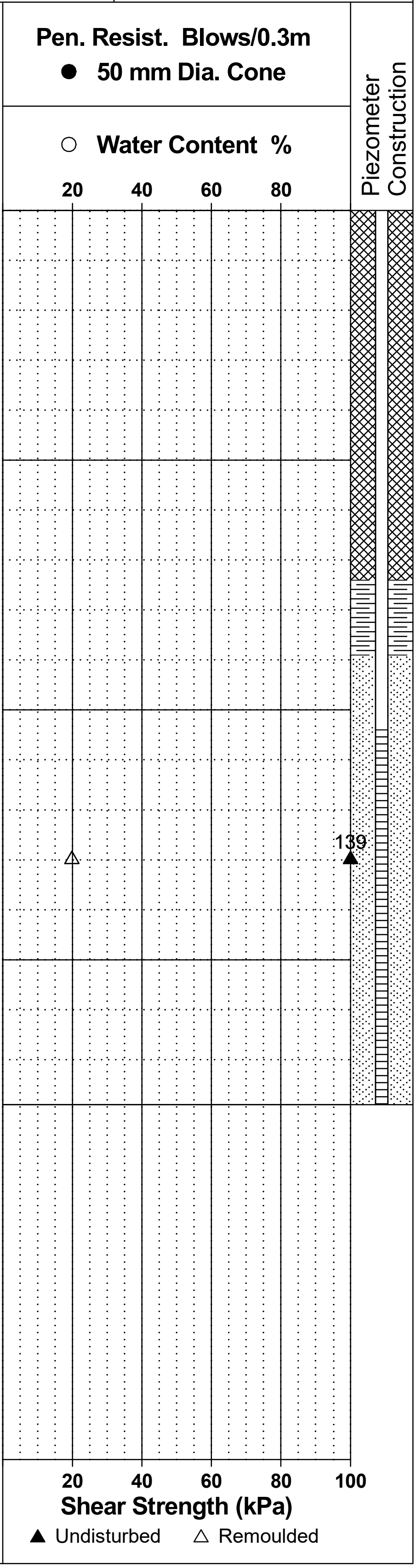
BORINGS BY CME 55 Power Auger

DATE 2019 January 15

FILE NO. PG4772

HOLE NO. BH12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.58
TOPSOIL 0.38		AU	1				
Very stiff, brown **SILTY CLAY**		SS	2	88	6	1	102.58
		SS	3	96	5	2	101.58
3.05						3	100.58
GLACIAL TILL: Compact, brown to grey clayey silt, some sand, trace gravel, occasional cobbles and boulders 3.58		SS	4	90	11		
End of Borehole Practical refusal to augering at 3.58m depth (BH Dry - Jan 29/19)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. **PG4772**

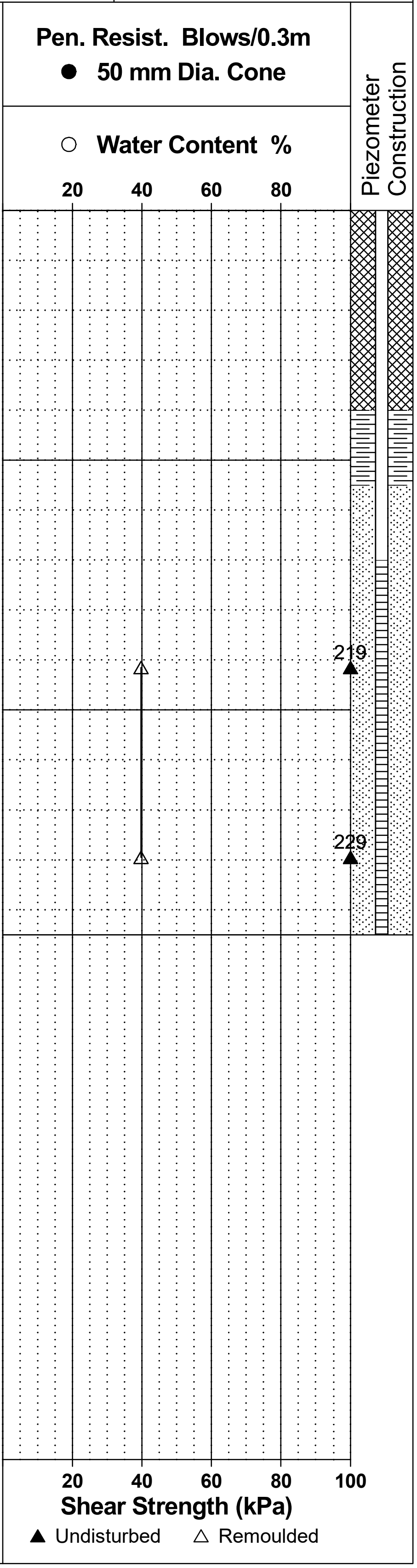
REMARKS

HOLE NO. **BH13**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.55
TOPSOIL 0.36		AU	1				
Hard, brown **SILTY CLAY**		SS	2	88	4	1	102.55
						2	101.55
2.90							
End of Borehole Practical refusal to augering at 2.90m depth (BH Dry - Jan 29/19)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
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FILE NO. **PG4772**

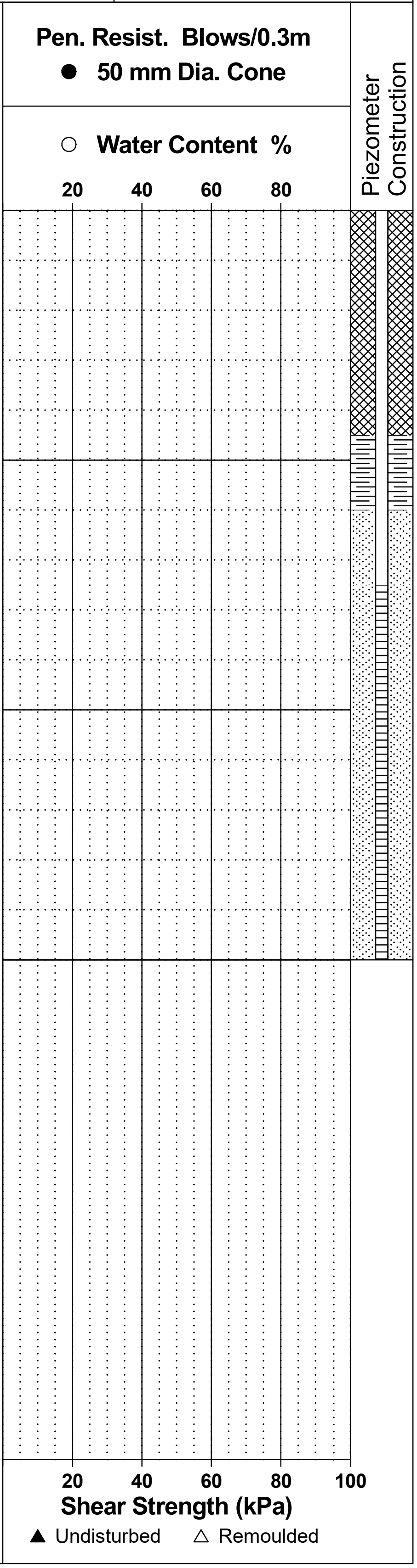
REMARKS

HOLE NO. **BH14**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.18
TOPSOIL	AU	1				
0.41						
Very stiff, brown **SILTY CLAY**	SS	2	67	7	1	103.18
	SS	3	96	6		
- grey by 2.0m depth					2	102.18
2.29						
GLACIAL TILL: Grey silty clay, trace sand and gravel, occasional cobbles and boulders						
3.00					3	101.18
End of Borehole Practical refusal to augering at 3.00m depth (BH Dry - Jan 29/19)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

FILE NO. PG4772

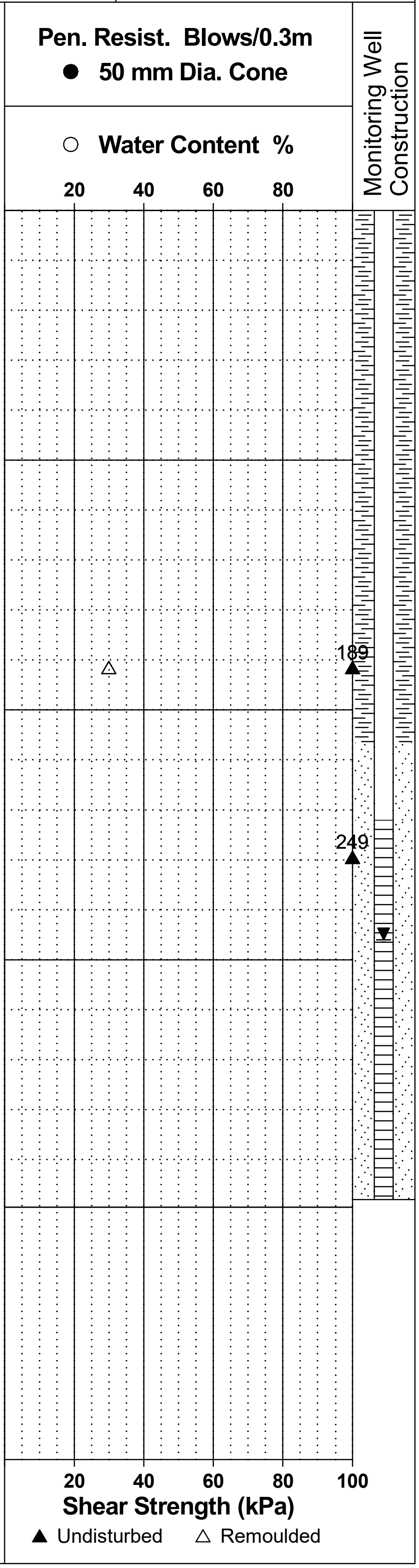
HOLE NO. BH15

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.65
TOPSOIL (to 0.36)	AU	1				
Very stiff, brown **SILTY CLAY** (0.36 to 2.29)	SS	2	71	6	1	102.65
					2	101.65
Hard, brown **CLAYEY SILT** (2.29 to 3.05)					3	100.65
GLACIAL TILL: Compact to very dense, grey clayey silt, some sand, trace gravel, occasional cobbles and boulders (3.05 to 3.99)	SS	3	79	24		
	SS	4	100	50+		

End of Borehole

Practical refusal to augering at 3.99m depth

(GWL @ 2.92m depth - Jan 29/19)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH16**

BORINGS BY CME 55 Power Auger

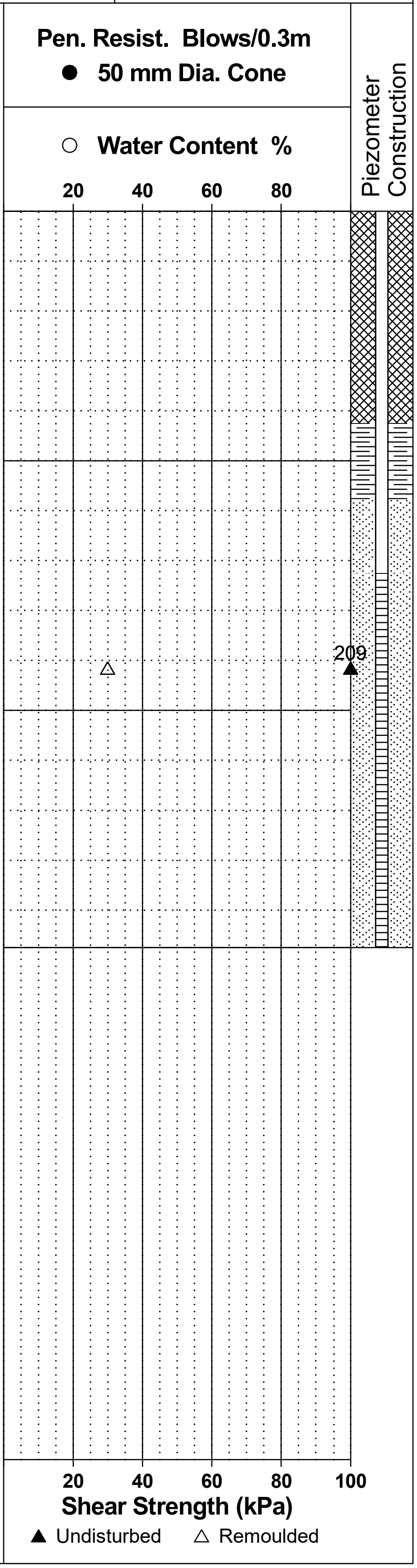
DATE 2019 January 15

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.66
TOPSOIL	AU	1				
0.33						
Hard, brown **SILTY CLAY**	SS	2	75	4	1	102.66
					2	101.66
2.29						
GLACIAL TILL: Dense, brown to grey clayey silt, some sand, gravel, cobbles and boulders	SS	3	46	31		
2.95						

End of Borehole

Practical refusal to augering at 2.95m depth

(BH Dry - Jan 29/19)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

HOLE NO. BH17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.19
TOPSOIL 0.38		AU	1				
Very stiff to hard, brown **CLAYEY SILT**		SS	2	79	7	1	103.19
- grey by 1.8m depth 2.23		SS	3	100	55	2	102.19

End of Borehole

Practical refusal to augering at 2.23m depth

(BH Dry - Jan 29/19)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

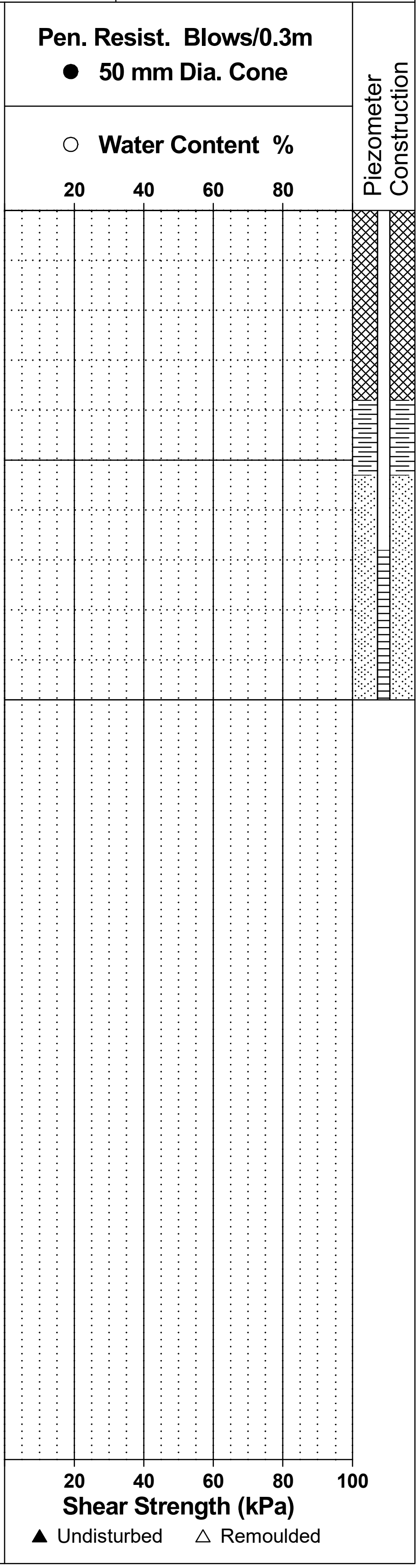
HOLE NO. BH18

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.15
TOPSOIL (to 0.33)	AU	1				
Hard, brown **CLAYEY SILT**	SS	2	88	11	1	103.15
- grey by 1.8m depth (to 1.96)	SS	3	88	50+		

End of Borehole

Practical refusal to augering at 1.96m depth

(BH Dry - Jan 29/19)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

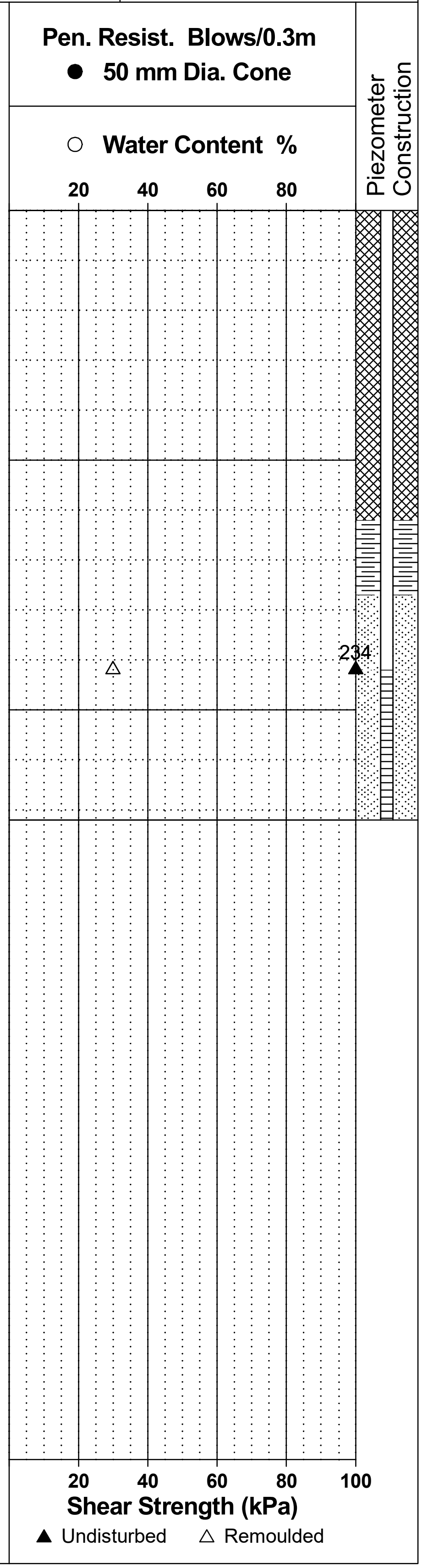
HOLE NO. BH19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.78
TOPSOIL 0.36		AU	1				
Hard, brown to grey **SILTY CLAY**		SS	2	88	3	1	102.78
						2	101.78
2.44		SS	3	100	50+		

End of Borehole

Practical refusal to augering at 2.44m depth

(BH Dry - Jan 29/19)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH20**

BORINGS BY CME 55 Power Auger

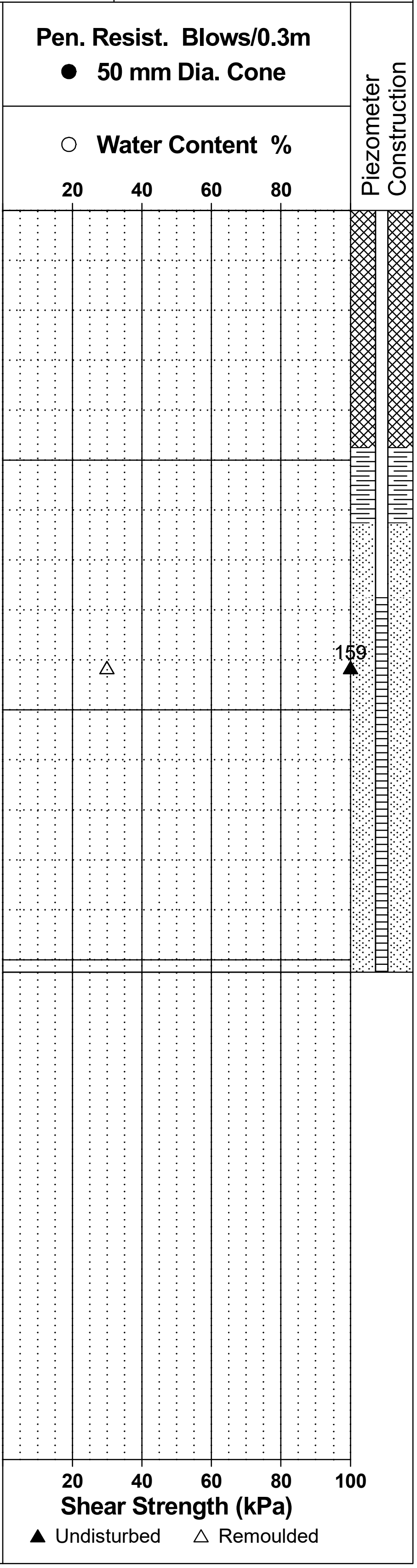
DATE 2019 January 16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.59
TOPSOIL 0.33	AU	1				
Very stiff, brown **SILTY CLAY**	SS	2	83	4	1	102.59
- grey by 1.8m depth 2.30					2	101.59
Loose, grey **CLAYEY SILT,** trace sand and gravel 3.05	SS	3	83	9	3	100.59

End of Borehole

Practical refusal to augering at 3.05m depth

(BH Dry - Jan 29/19)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4772

REMARKS

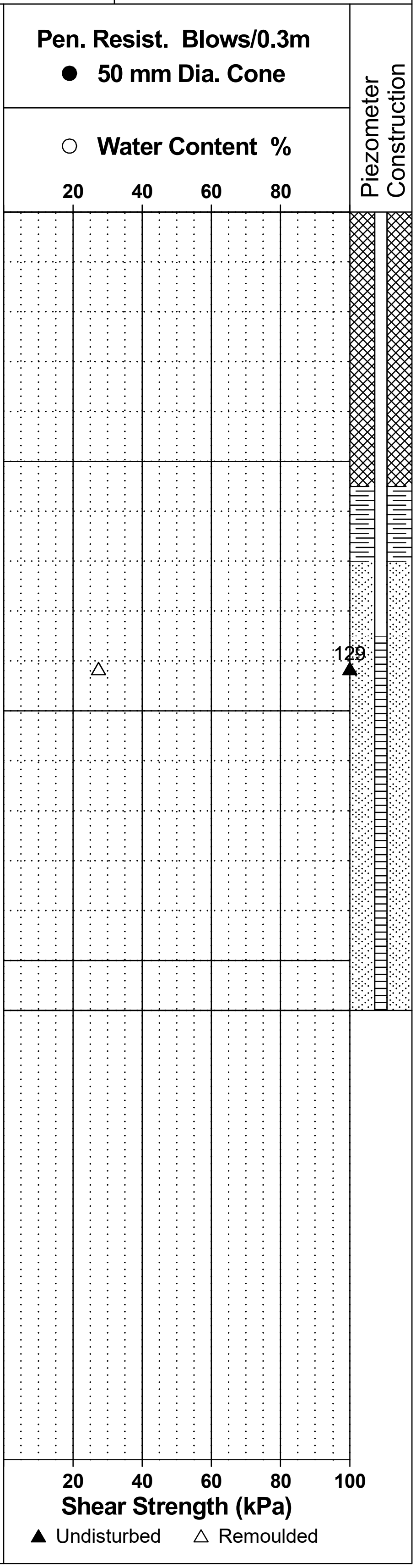
HOLE NO. BH21

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.33		AU	1		
Very stiff, brown **SILTY CLAY**		SS	2	79	5
- grey by 1.8m depth 2.29					
GLACIAL TILL: Compact to very dense, brown to grey sandy silt, some clay, gravel, cobbles and boulders		SS	3	71	13
3.20		SS	4	100	50+
End of Borehole Practical refusal to augering at 3.20m depth (BH Dry - Jan 29/19)					

DEPTH (m)	ELEV. (m)
0	103.58
1	102.58
2	101.58
3	100.58



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

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REMARKS

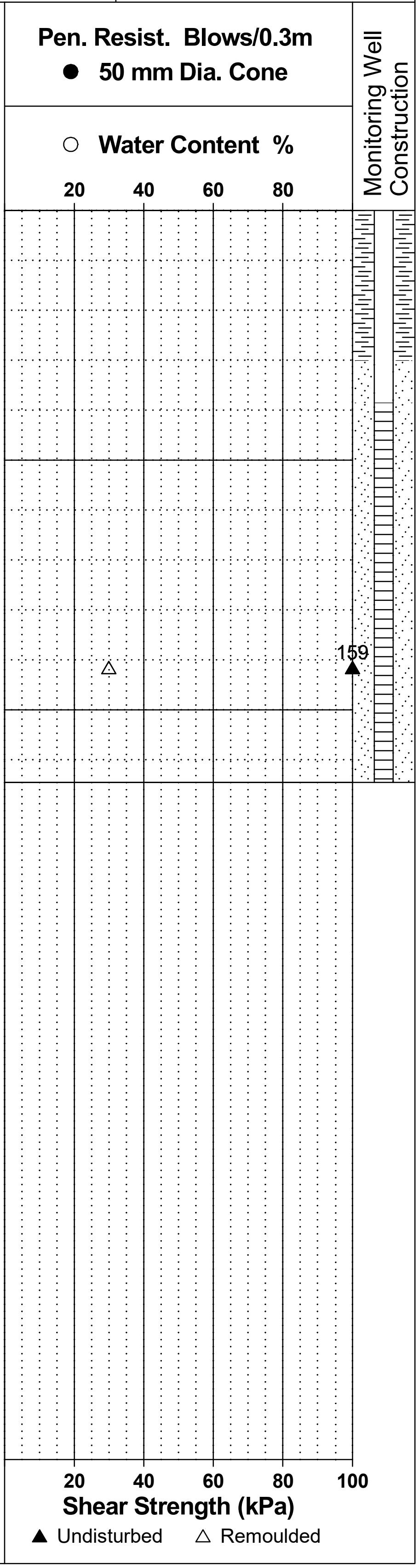
BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

HOLE NO. BH22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.65
TOPSOIL 0.25		AU	1				
Very stiff, brown **SILTY CLAY**		SS	2	71	5	1	102.65
- grey by 2.0m depth 2.29						2	101.65
End of Borehole Practical refusal to augering at 2.29m depth (BH Dry - Jan 29/19)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

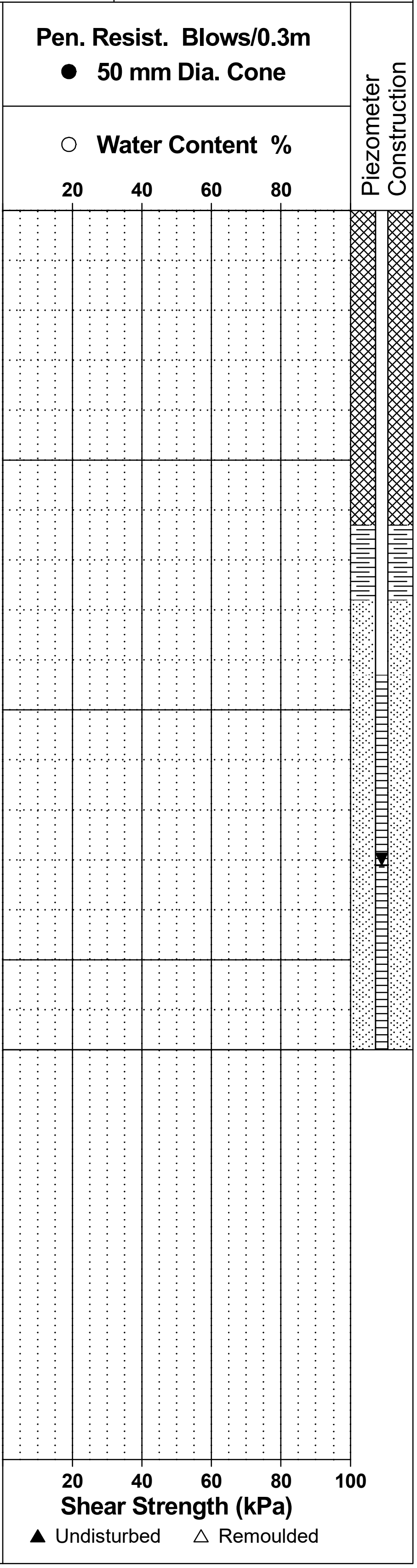
HOLE NO. BH23

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.87
TOPSOIL 0.30	AU	1				
Very stiff, brown **SILTY CLAY**, some sand	SS	2	0	6	1	102.87
1.52						
GLACIAL TILL: Dense to very dense, grey silty sand with clay, gravel, cobbles and boulders	SS	3	83	11	2	101.87
	SS	4	75	36	3	100.87
3.36	SS	5	31	50+		

End of Borehole

Practical refusal to augering at 3.36m depth

(GWL @ 2.62m depth - Jan 29/19)



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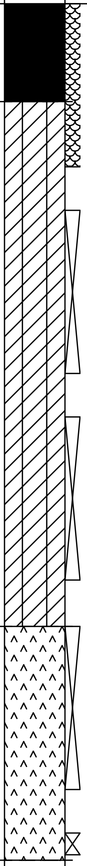
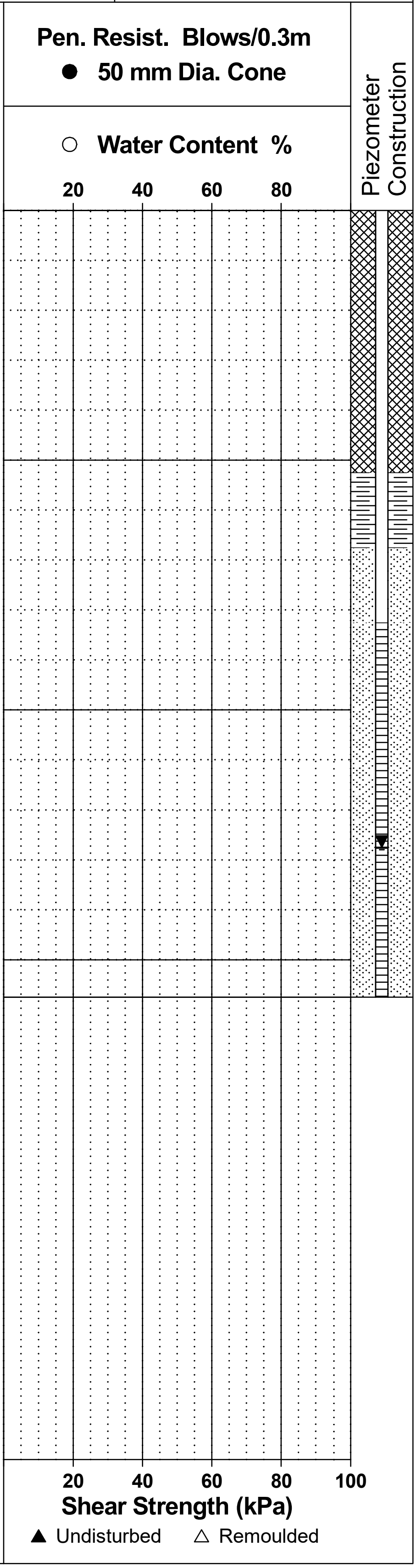
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.
FILE NO. **PG4772**
REMARKS
HOLE NO. **BH24**
BORINGS BY CME 55 Power Auger
DATE 2019 January 16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.04
TOPSOIL 0.36	AU	1				
Very stiff, brown to grey **CLAYEY SILT**	SS	2	67	10	1	103.04
	SS	3	79	29	2	102.04
2.29						
GLACIAL TILL: Compact to very dense, brown clayey silt, some sand, gravel, cobbles and boulders	SS	4	58	23		
3.15	SS	5	100	50+	3	101.04
End of Borehole Practical refusal to augering at 3.15m depth (GWL @ 2.55m depth - Jan 29/19)						





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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

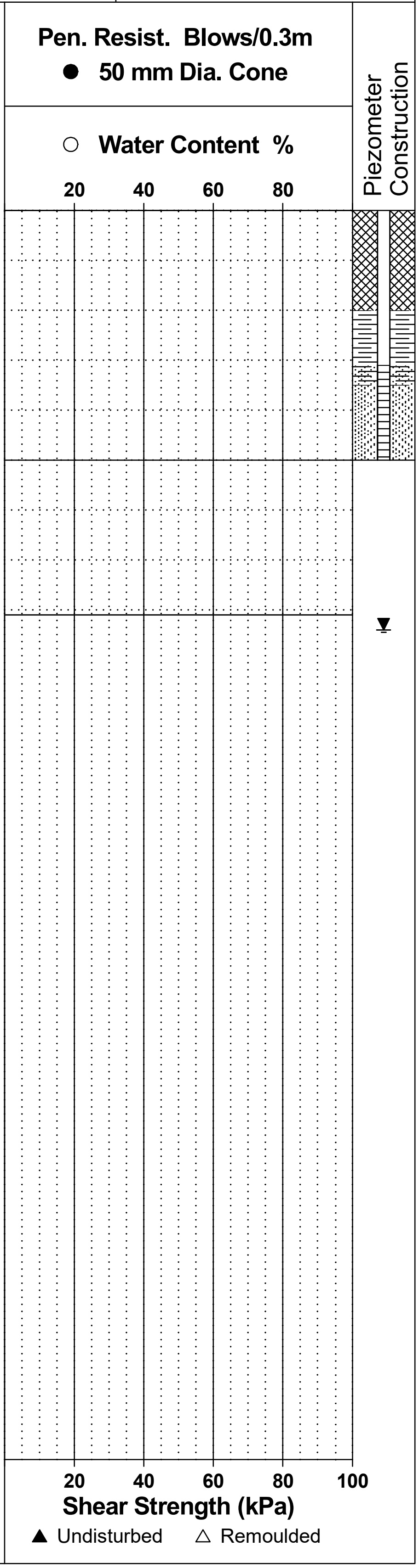
HOLE NO. BH25

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.07
TOPSOIL 0.38	AU	1				
Very stiff, brown **CLAYEY SILT** 1.52	SS	2	75	11	1	103.07
GLACIAL TILL: Very dense, grey clayey silt with sand, gravel, cobbles, boulders 1.62	SS	3	75	50+		

End of Borehole

Practical refusal to augering at 1.62m depth

(GWL @ 1.68m depth - Jan 29/19)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

FILE NO. PG4772

HOLE NO. BH26

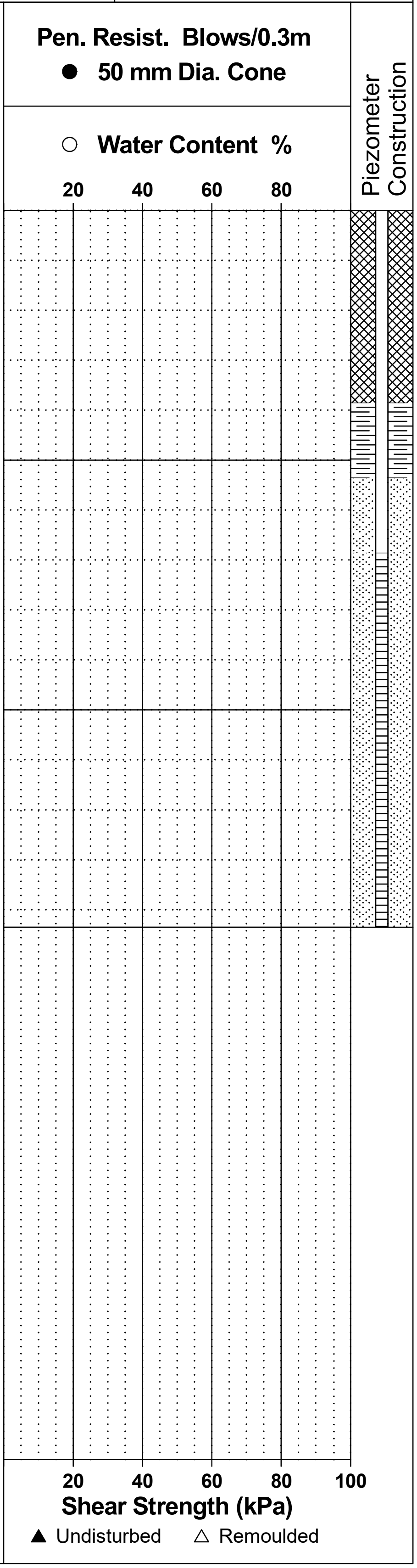
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.38		AU	1		
Very stiff, brown **CLAYEY SILT**		SS	2	75	9
1.83		SS	3	50	19
GLACIAL TILL: Compact to dense, grey silty clay with gravel, cobbles and boulders 2.87		SS	4	100	46

End of Borehole

Practical refusal to augering at 2.87m depth

(BH Dry - Jan 29/19)

DEPTH (m)	ELEV. (m)
0	104.30
1	103.30
2	102.30



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

FILE NO. PG4772

HOLE NO. BH27

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.97
TOPSOIL (to 0.33)		AU	1				
Very stiff, brown **CLAYEY SILT**		SS	2	71	8	1	102.97
- grey by 1.7m depth (to 1.93)		SS	3	88	50+		
End of Borehole Practical refusal to augering at 1.93m depth (BH Dry - Jan 29/19)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

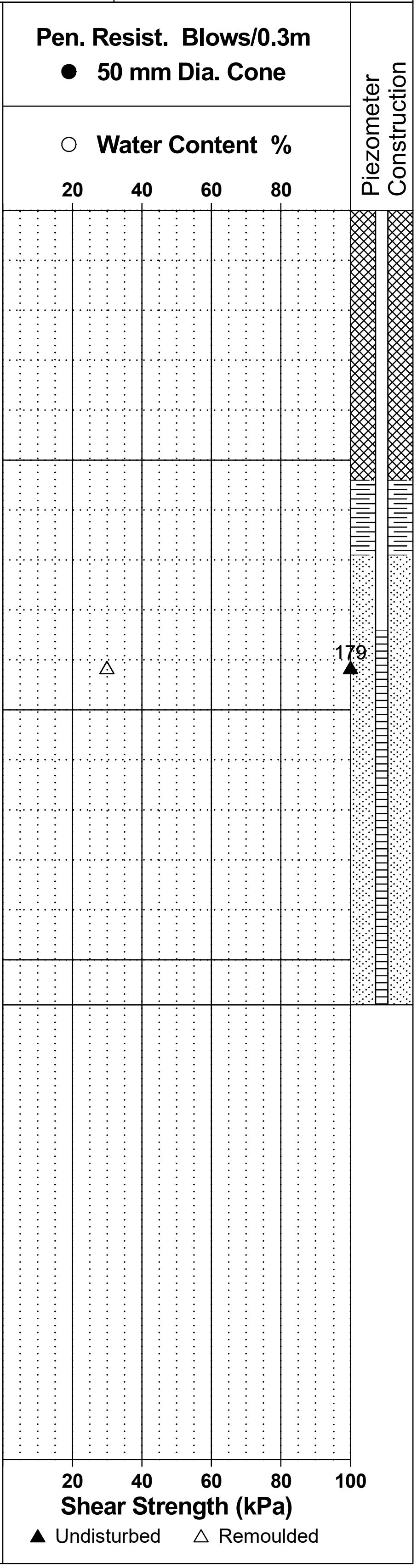
BORINGS BY CME 55 Power Auger

DATE 2019 January 17

FILE NO. PG4772

HOLE NO. BH28

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	103.78
TOPSOIL		AU	1				
0.36							
Very stiff, brown **SILTY CLAY**		SS	2	38	6	1	102.78
						2	101.78
2.29							
GLACIAL TILL: Loose to very dense, grey silty clay with sand, gravel, cobbles and boulders		SS	3	8	2		
		SS	4	0	50+	3	100.78
3.18							
End of Borehole Practical refusal to augering at 3.18m depth (BH Dry - Jan 29/19)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4772

REMARKS

HOLE NO. BH29

BORINGS BY CME 55 Power Auger

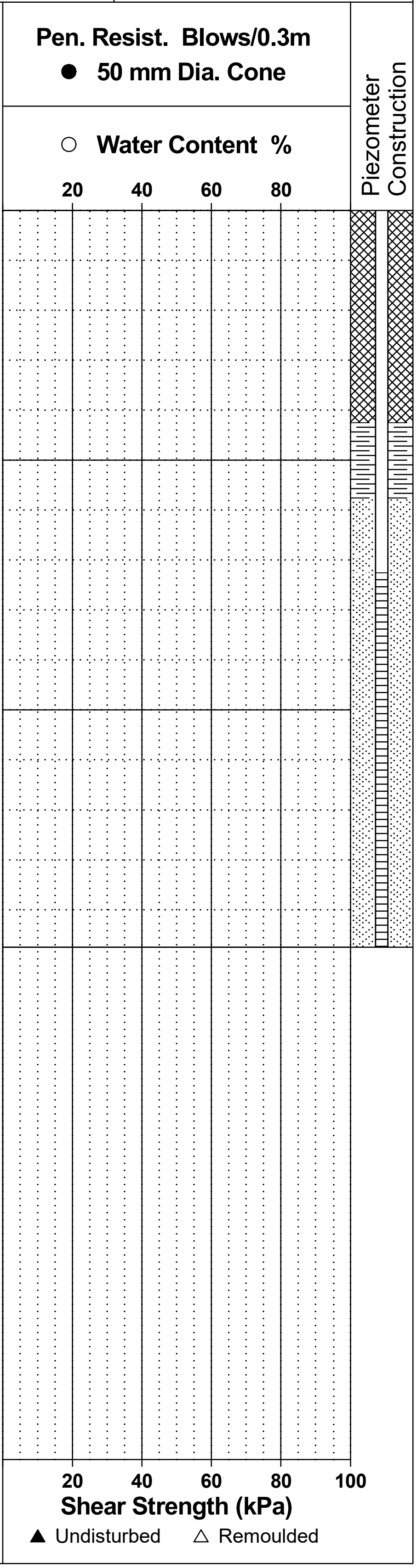
DATE 2019 January 17

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.71
TOPSOIL (to 0.38)	AU	1				
Very stiff, brown **SILTY CLAY** (0.38 to 2.29)	SS	2	50	7	1	102.71
	SS	3	71	4	2	101.71
GLACIAL TILL: Loose, grey silty clay with sand, gravel, cobbles and boulders (2.29 to 2.95)	SS	4	17	7		

End of Borehole

Practical refusal to augering at 2.95m depth

(BH Dry - Jan 29/19)



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)

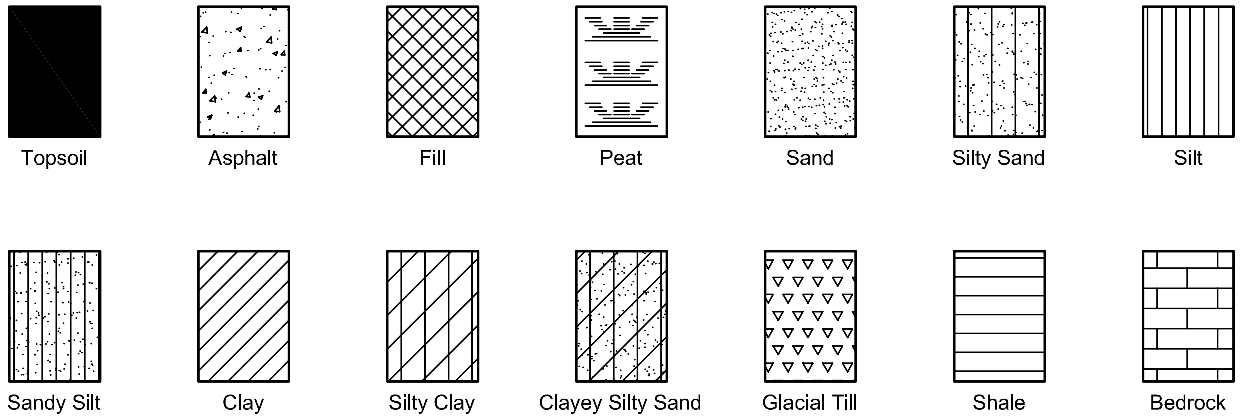
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

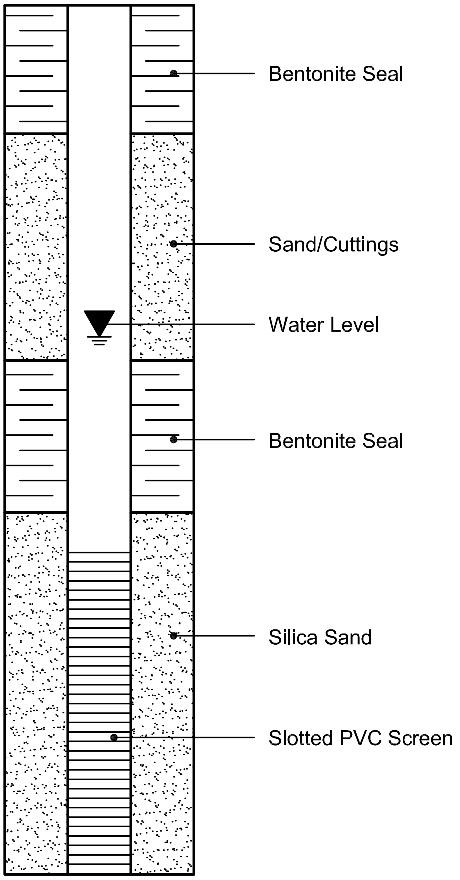
SYMBOLS AND TERMS (continued)

STRATA PLOT

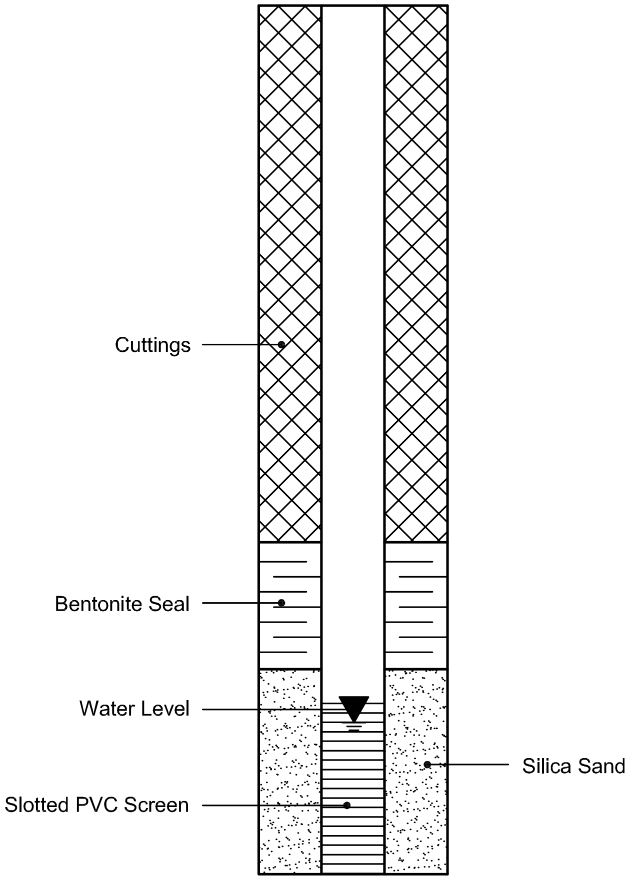


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 25648

Report Date: 22-Jan-2019

Order Date: 16-Jan-2019

Project Description: PG4772

	Client ID:	BH#16-19 SS#3	-	-	-
	Sample Date:	01/15/2019 09:00	-	-	-
	Sample ID:	1903309-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	85.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.80	-	-	-
Resistivity	0.10 Ohm.m	76.2	-	-	-
Anions					
Chloride	5 ug/g dry	6	-	-	-
Sulphate	5 ug/g dry	6	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 4 – SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG4772-1 - TEST HOLE LOCATION PLAN

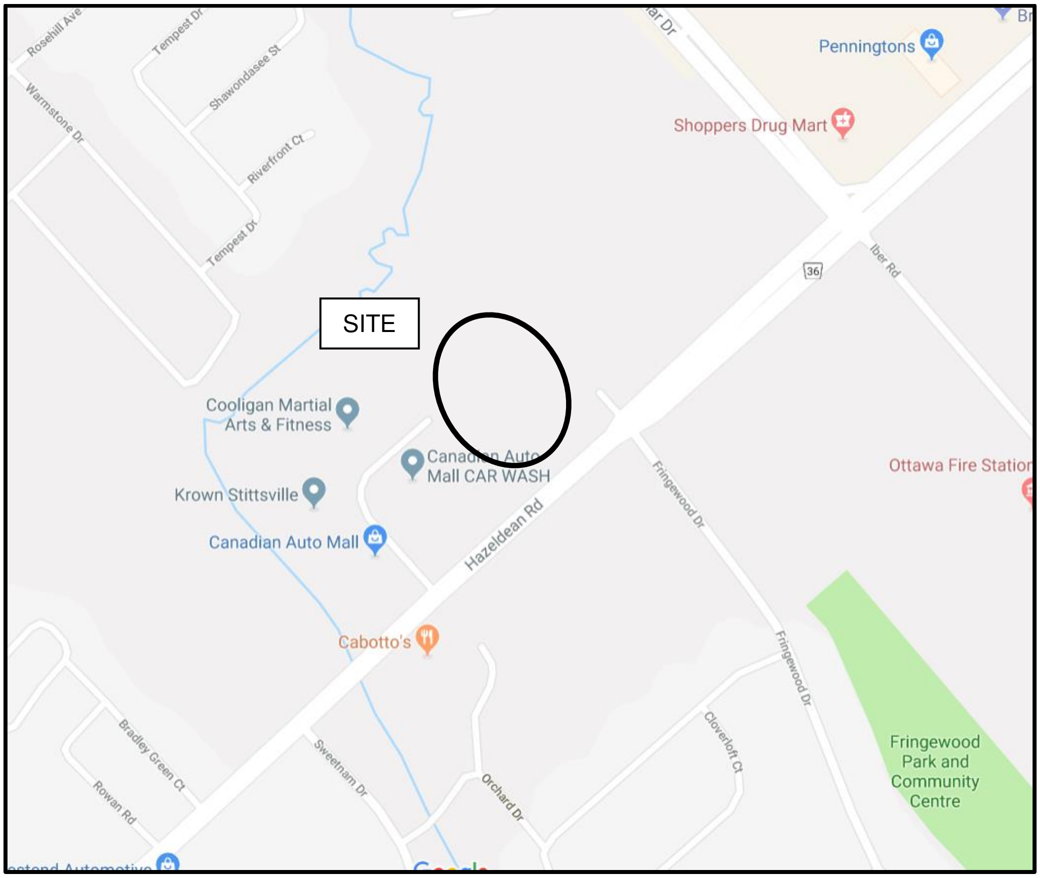


FIGURE 1

KEY PLAN

patersongroup

Figure 2A - Section A - Static Analysis

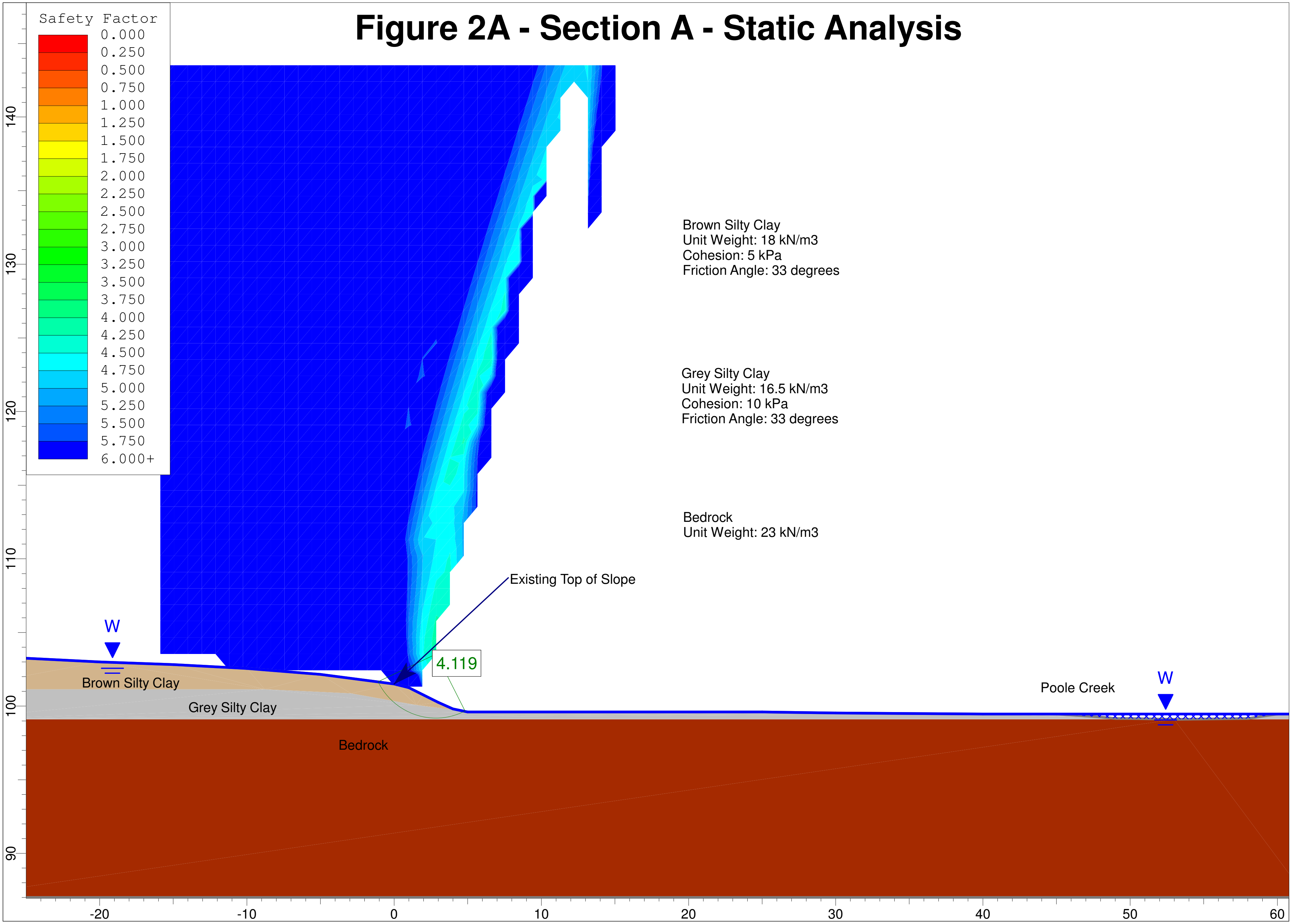


Figure 2B - Section A - Seismic Loading

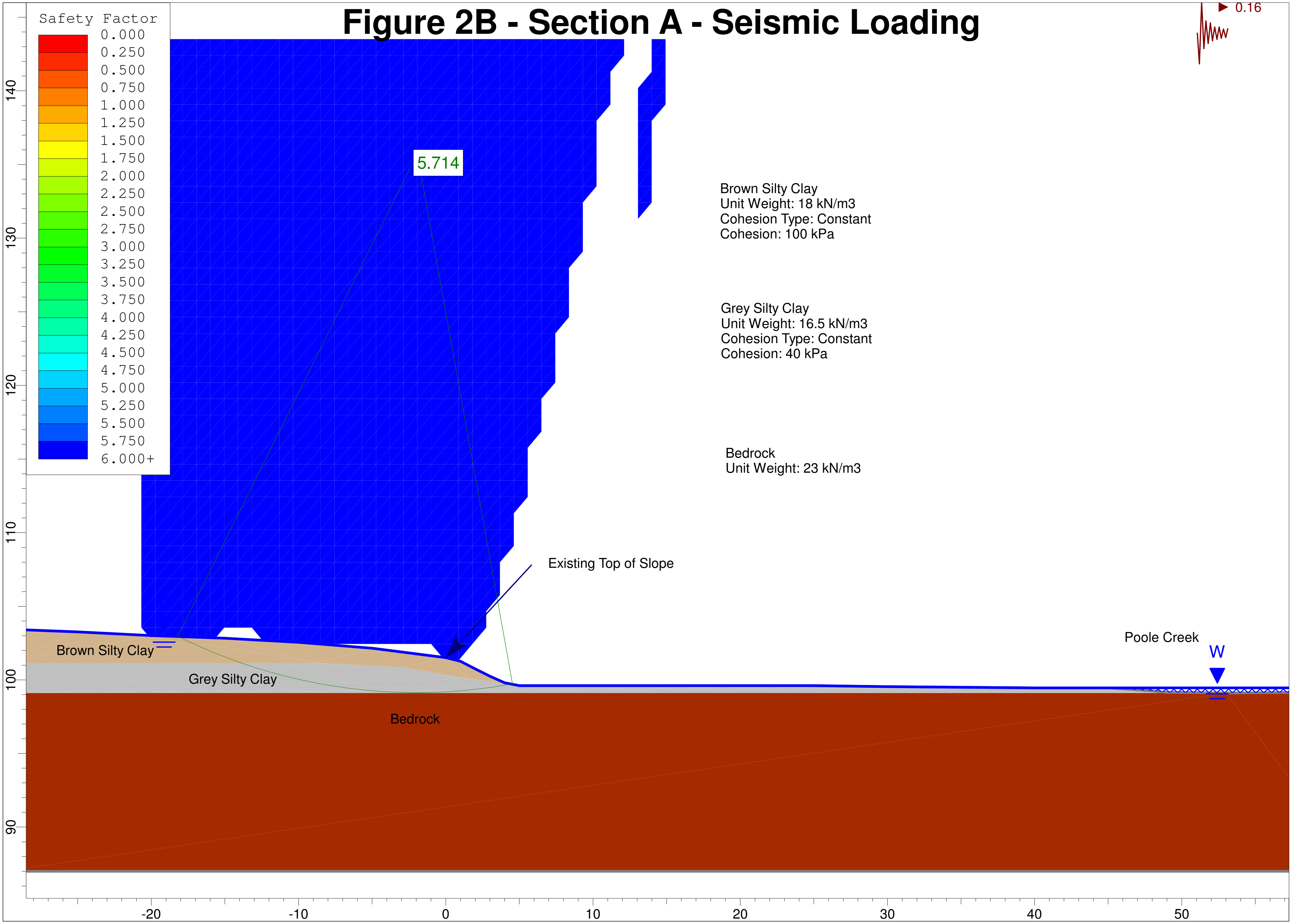
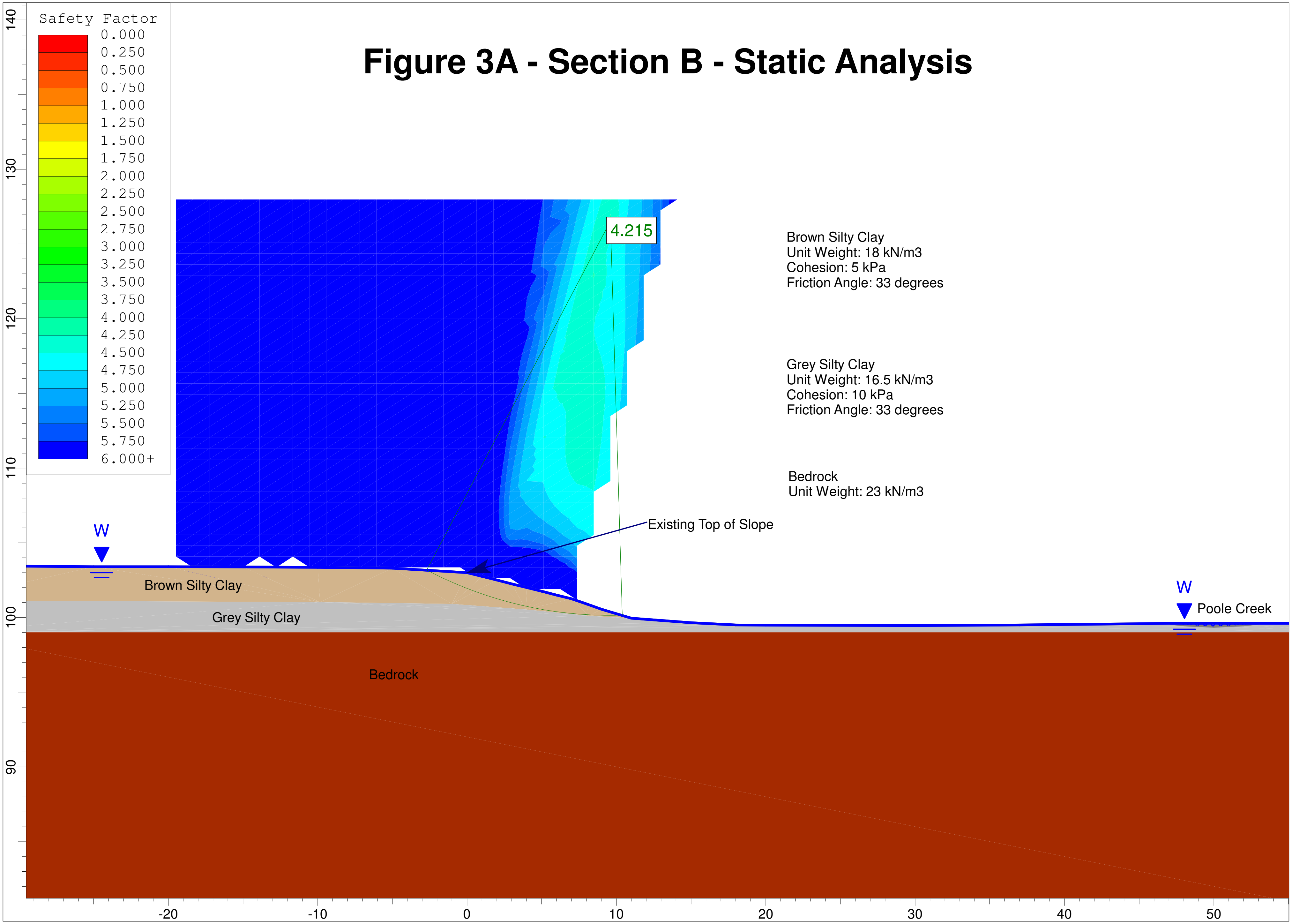


Figure 3A - Section B - Static Analysis



Brown Silty Clay
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees

Grey Silty Clay
Unit Weight: 16.5 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees

Bedrock
Unit Weight: 23 kN/m3

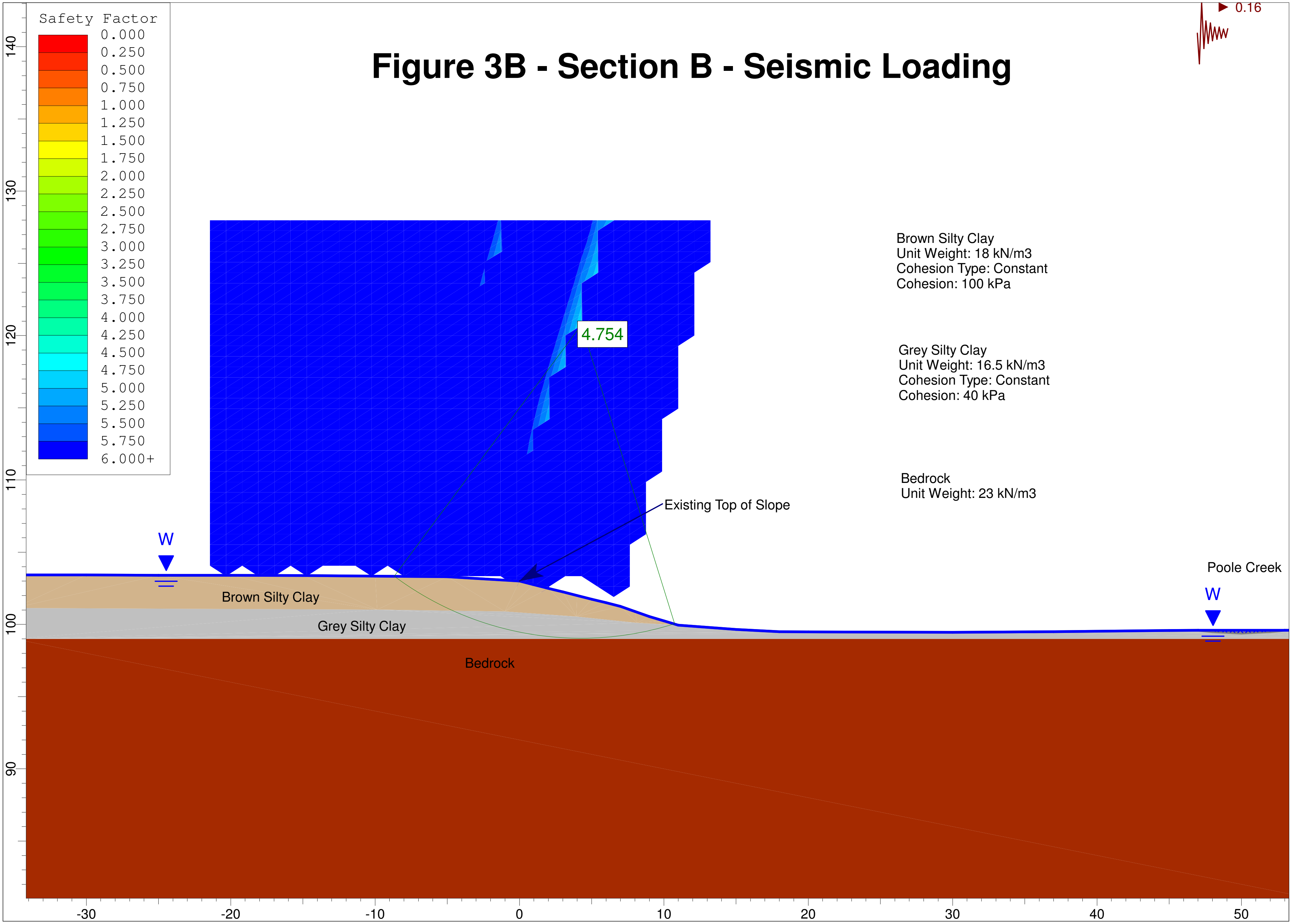
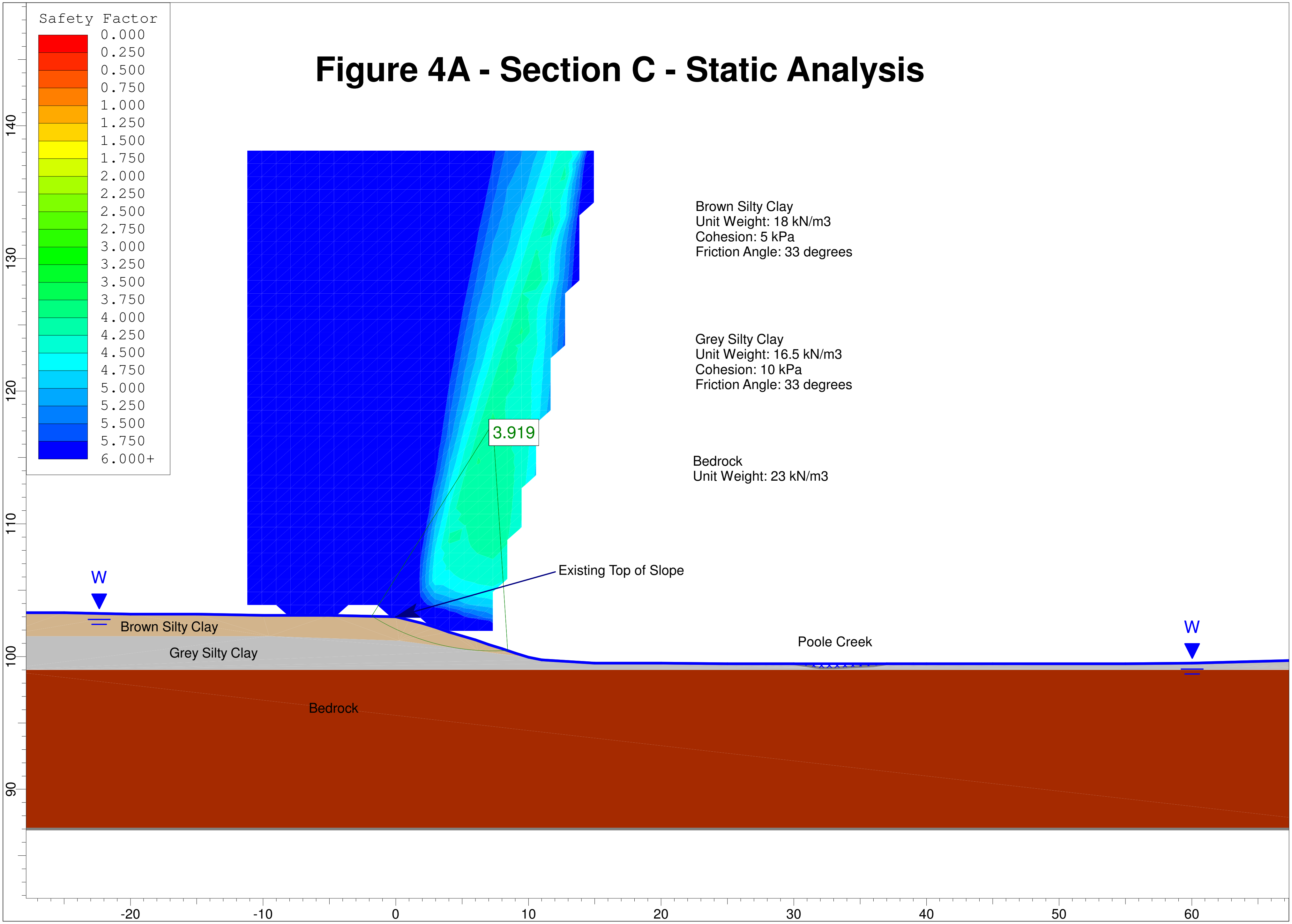


Figure 3B - Section B - Seismic Loading

Figure 4A - Section C - Static Analysis

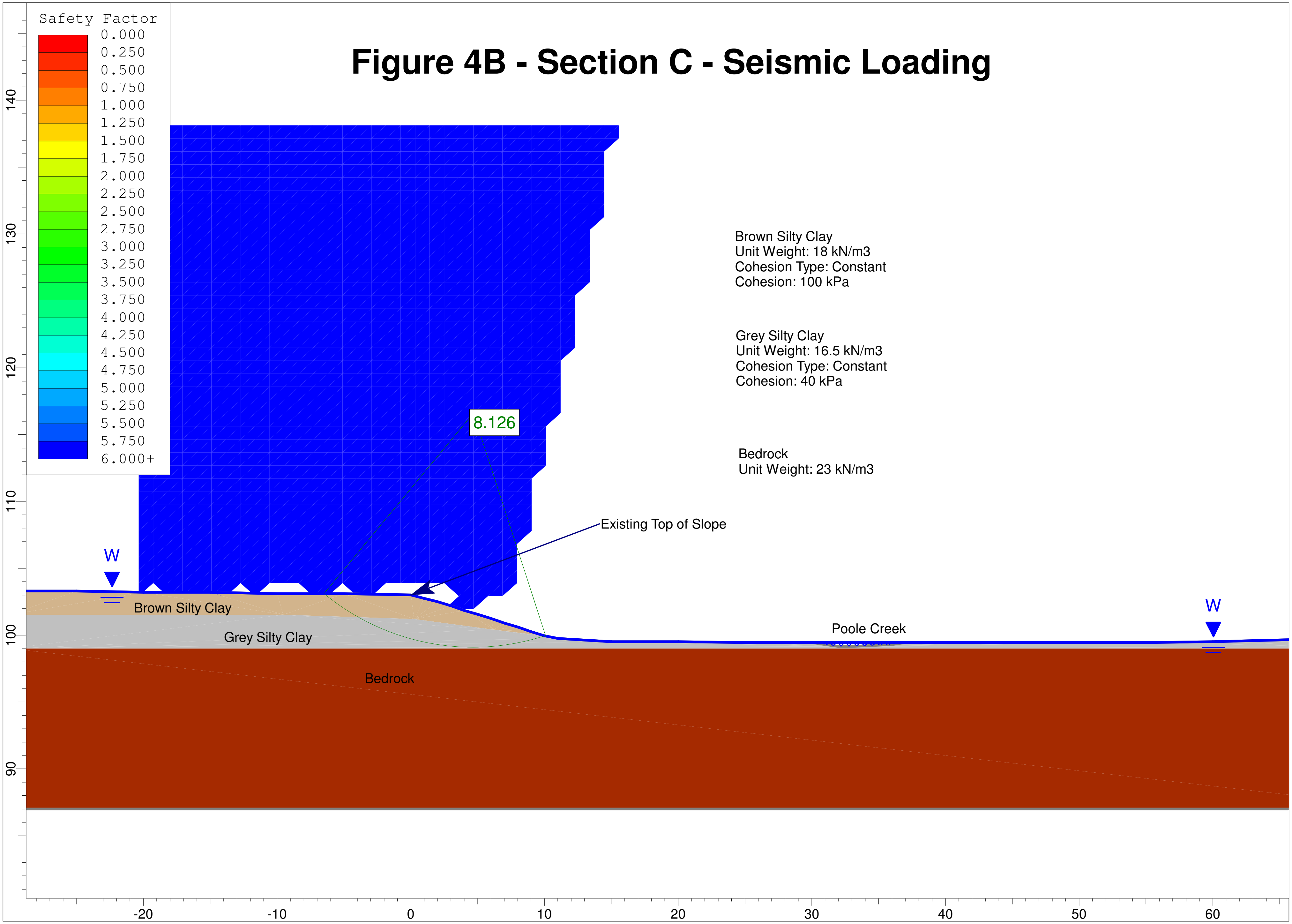


Brown Silty Clay
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees

Grey Silty Clay
Unit Weight: 16.5 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees

Bedrock
Unit Weight: 23 kN/m3

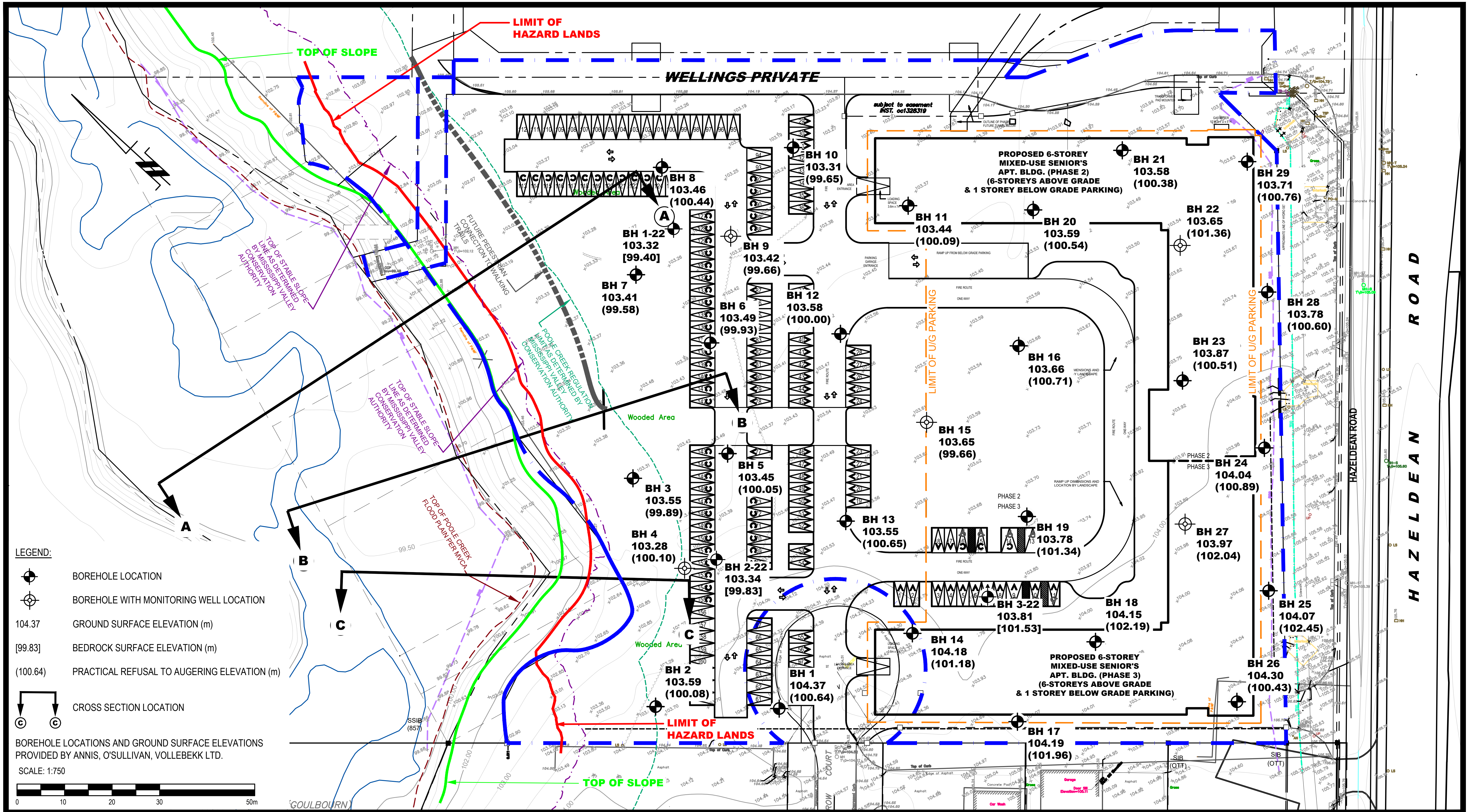
Figure 4B - Section C - Seismic Loading



Brown Silty Clay
Unit Weight: 18 kN/m3
Cohesion Type: Constant
Cohesion: 100 kPa

Grey Silty Clay
Unit Weight: 16.5 kN/m3
Cohesion Type: Constant
Cohesion: 40 kPa

Bedrock
Unit Weight: 23 kN/m3



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
104.37	GROUND SURFACE ELEVATION (m)
[99.83]	BEDROCK SURFACE ELEVATION (m)
(100.64)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
Ⓒ Ⓒ	CROSS SECTION LOCATION

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

SCALE: 1:750

PATERSON GROUP
SOLUTION ORIENTED ENGINEERING
9 AURIGA DRIVE
OTTAWA, ON
K2E 7S9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
4	UPDATED CONCEPTUAL PLAN ADDED TO DRAWING	22/08/2022	KP
3	2022 BOREHOLES ADDED TO PLAN	03/02/2022	OC
2	REVISED SITE PLAN	22/09/2020	JV
1	LIMIT OF HAZARD LANDS REVISED	28/02/2019	FA

NAUTICAL LANDS GROUP

GEOTECHNICAL INVESTIGATION

PROP. MIXED-USE DEVELOPMENT - WELLINGS OF STITTSVILLE PHASE 2 - 20 CEDAROW CT.

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 01/2019
Drawn by: MPG
Report No.: PG4772-1
Checked by: FA
Dwg. No.: PG4772-1
Approved by: DJG
Revision No.: 4

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