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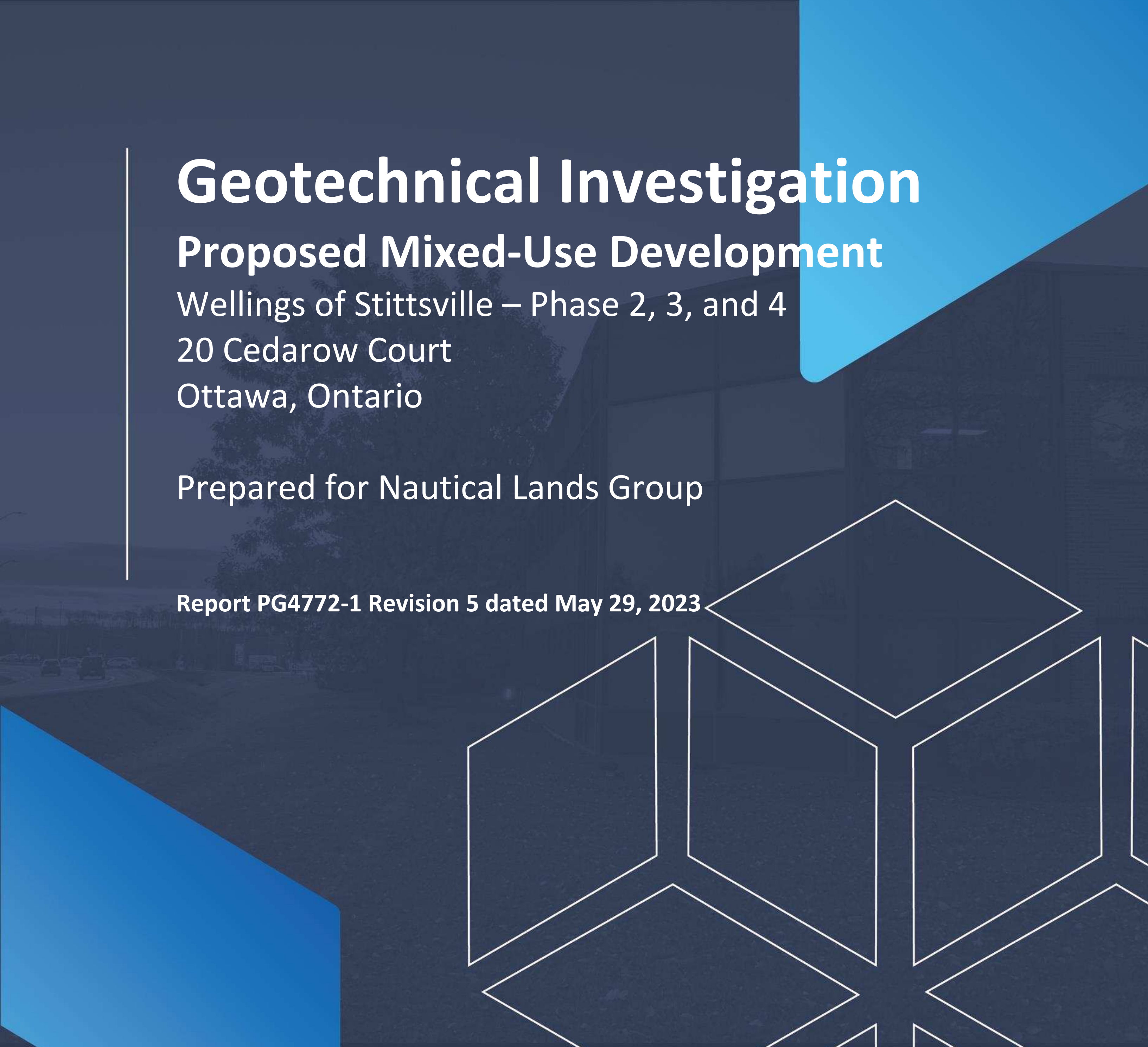

Geotechnical Investigation

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 5 dated May 29, 2023

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Appendices

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

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided on Figures 5 and 6, which are presented in Appendix 2 of this report.

Field Program

The seismic array testing location was placed as presented in Drawing PG4772-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

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

The shot locations were also completed in a forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 15, 1.5 and 1.0 m away from the first and last geophone, and at the center of the seismic array.

Data Processing and Interpretation

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

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

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

Site Class for Footings on Bedrock Surface

Based on our testing results, the bedrock shear wave velocity is **2,020 m/s**. Considering that the proposed development is founded on conventional footings placed on a bedrock surface, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below.

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

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{2{,}020\ m/s}\right)}$$

$$V_{s30=} 2{,}020\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30}, is **2,020 m/s** for conventional footings placed on bedrock surface. Therefore, a **Site Class A** is applicable for the design of the proposed mixed-use development founded on conventional footings placed on the bedrock surface, as per Table 4.1.8.4.A of the OBC 2012.

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

Site Class for Footings within 3 m of Bedrock Surface

Based on our testing results, the average overburden shear wave velocity is **135 m/s**, while the bedrock shear wave velocity is **2,020 m/s**.

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

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

$$V_{s30=} \frac{30\ m}{\left(\frac{3\ m}{135\ m/s} + \frac{27\ m}{2{,}020\ m/s}\right)}$$

$$V_{s30=} 843\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **843 m/s** for conventional spread footings founded within 3 m of the bedrock surface. Therefore, a **Site Class B** is applicable for design of the proposed mixed-use development where this applies, as per Table 4.1.8.4.A of the OBC 2012.

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

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 with 13 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·ac·γ·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 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 detention system

In summary the chambers are suitable to function as a stormwater detention system for the proposed development. It is recommended that the proposed system be founded with a minimum of 1 m of separation between the base of the system and the static, long-term groundwater table be maintained in order to comply with the LID guidelines. The infiltration rates are as assumed to be 10-30 mm per hour, any LID’s being considered should incorporate within the above-mentioned range of infiltration.

Due to the recommended separation between the groundwater table and the base of the system, uplift forces will be negligible.

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 storm water detention system is sufficient, therefore, the slope will not be negatively impacted by the proposed storm water system.

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.

LICENSED PROFESSIONAL ENGINEER
May 29, 2023
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Fernanda Carozzi, PhD. Geoph.

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

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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.

REMARKS

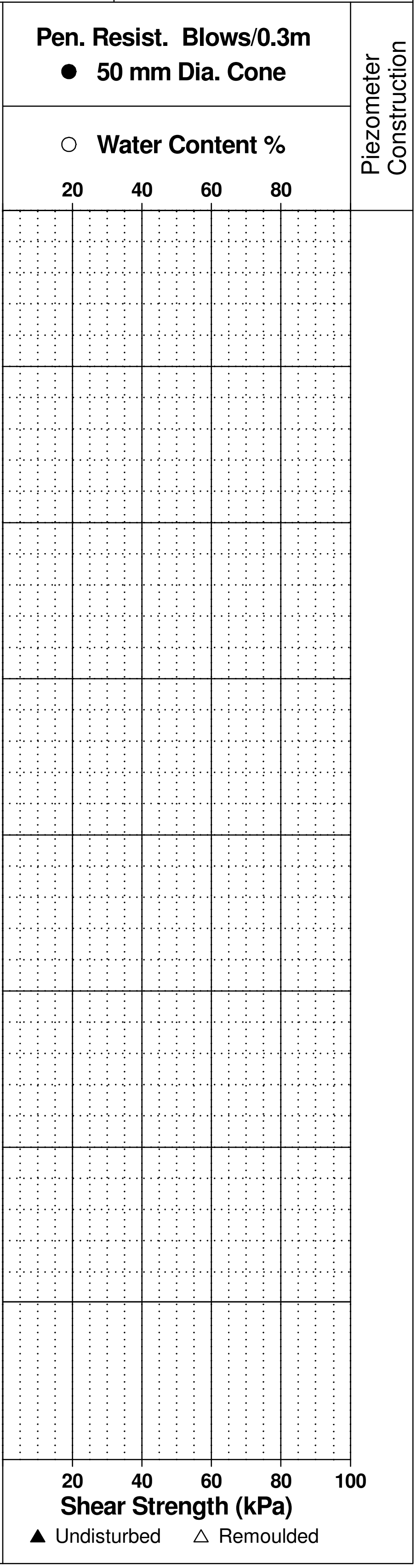
BORINGS BY CME 55 Power Auger

DATE February 2, 2022

FILE NO. PG4772

HOLE NO. BH1-22

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
3.73						3	100.32
						4	99.32
BEDROCK		RC	1	100	100	5	98.32
		RC	2	100	100	6	97.32
6.99 End of Borehole							



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

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REMARKS

BORINGS BY CME 55 Power Auger

DATE February 2, 2022

FILE NO. PG4772

HOLE NO. BH2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.34
OVERBURDEN						1	102.34
						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							

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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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 February 2, 2022

FILE NO. PG4772

HOLE NO. BH3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
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									

20 40 60 80

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

SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

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FILE NO. **PG4772**

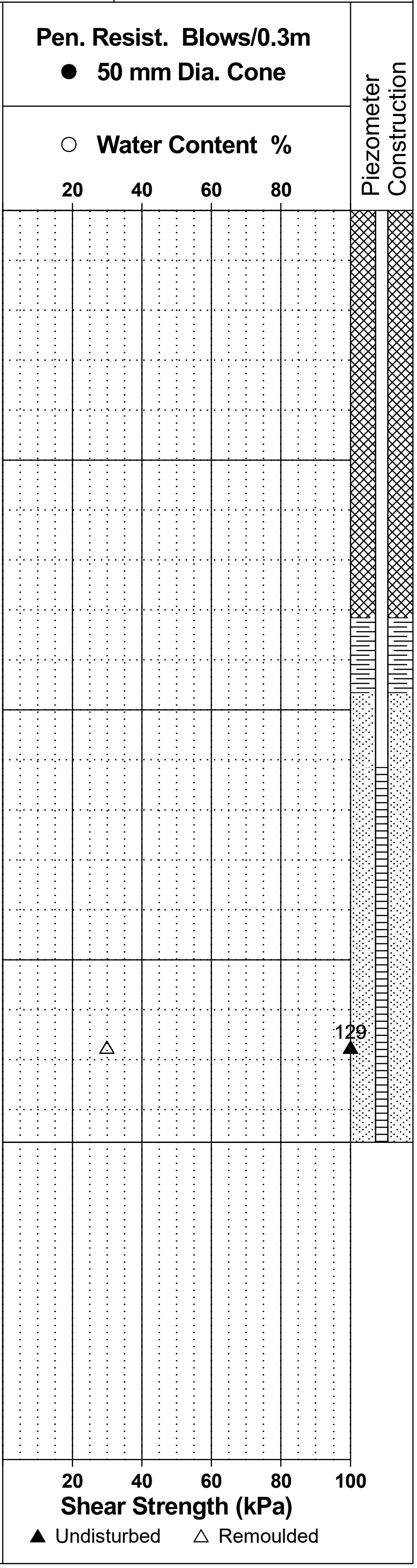
REMARKS

HOLE NO. **BH 1**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
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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Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

FILE NO. PG4772

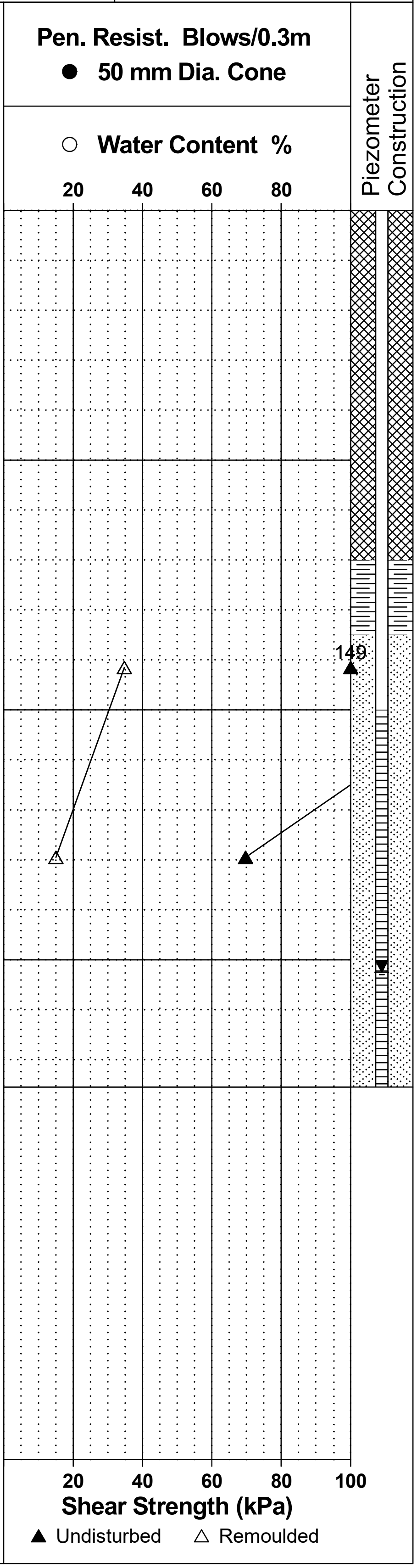
HOLE NO. BH 2

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							
Very stiff to stiff, brown **SILTY CLAY**		SS	2	33	4	1	102.59
						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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Ottawa, Ontario

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REMARKS

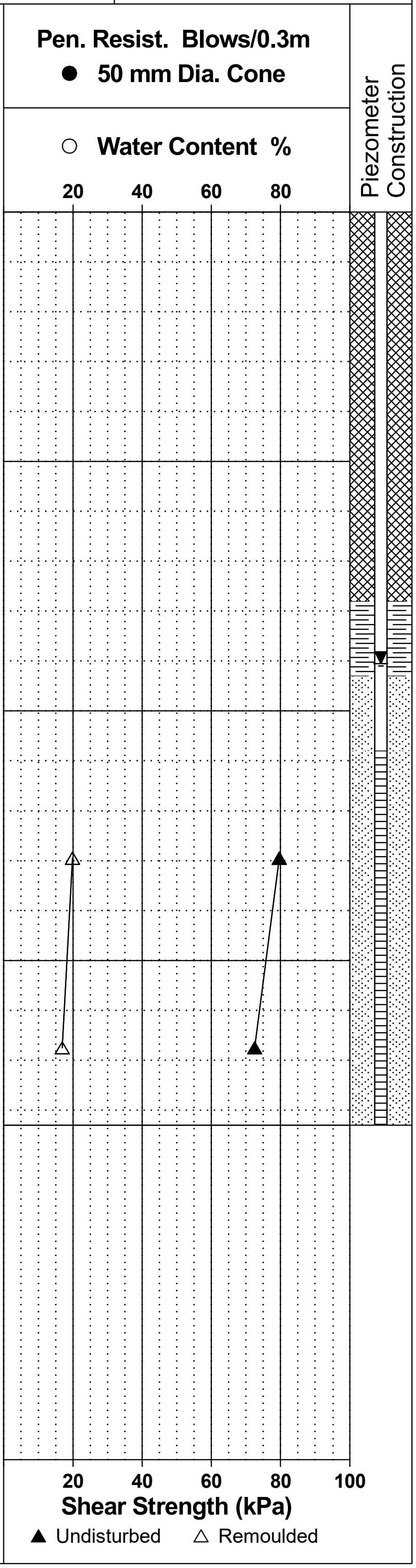
BORINGS BY CME 55 Power Auger

DATE 2019 January 14

FILE NO. PG4772

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.55
TOPSOIL		AU	1				
0.33							
Very stiff to stiff, brown **SILTY CLAY**		SS	2	21	7	1	102.55
		SS	3	62	7	2	101.55
- grey by 2.3m depth						3	100.55
3.66							
End of Borehole Practical refusal to augering at 3.66m depth (GWL @ 1.81m depth - Jan 29/19)							



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

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 14

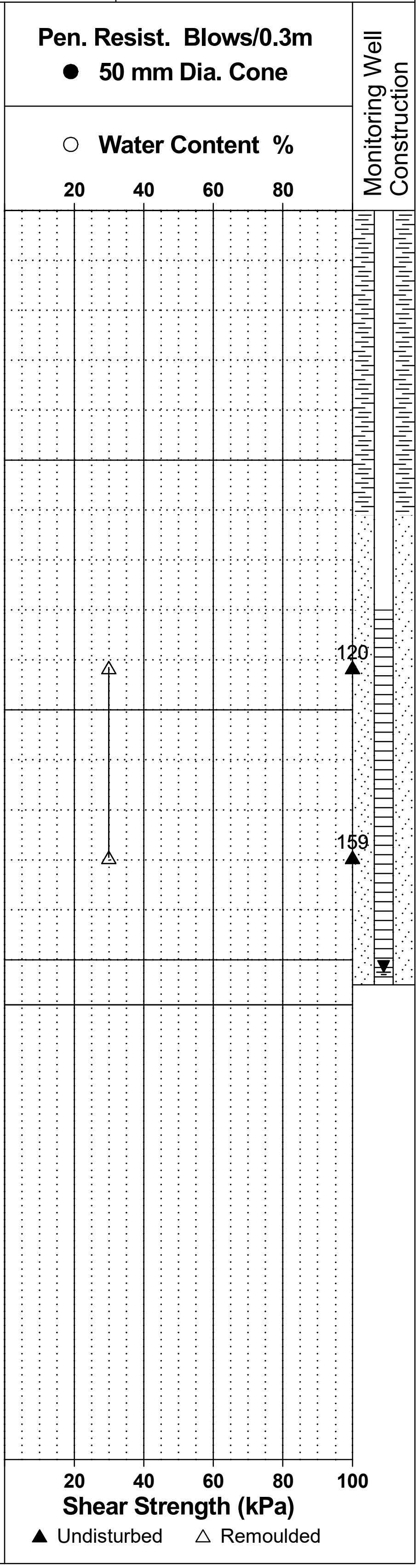
FILE NO. PG4772

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL						0	103.28
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

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

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

FILE NO. **PG4772**

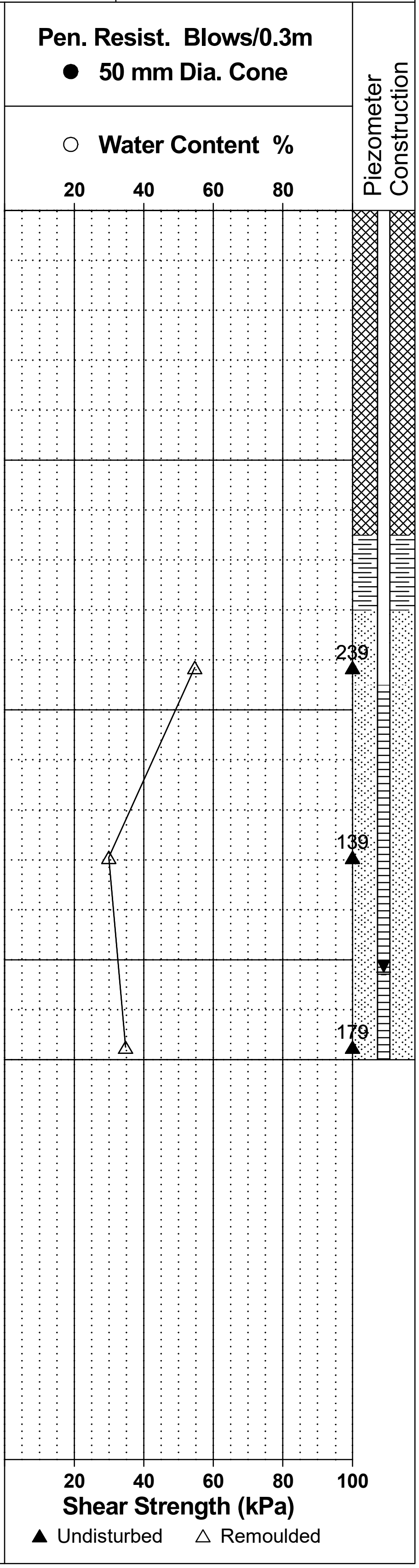
REMARKS

HOLE NO. **BH 5**

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	103.45
TOPSOIL 0.36		AU	1				
Hard to very stiff, brown **SILTY CLAY**		SS	2	38	6	1	102.45
- grey by 2.1m depth						2	101.45
3.40						3	100.45
End of Borehole Practical refusal to augering at 3.40m depth (GWL @ 3.05m depth - Jan 29/19)							



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SOIL PROFILE AND TEST DATA

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

FILE NO. **PG4772**

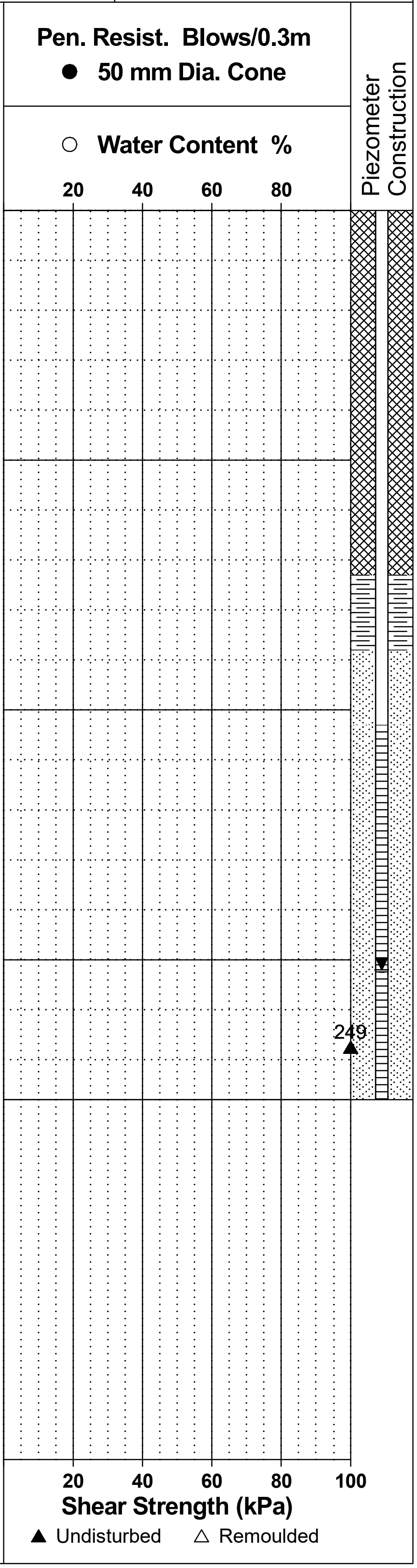
REMARKS

HOLE NO. **BH 6**

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	103.49
TOPSOIL		AU	1				
0.38							
Very stiff, brown **SILTY CLAY**		SS	2	58	8	1	102.49
		SS	3	71	9		
- grey by 2.0m depth						2	101.49
		SS	4	100	5		
						3	100.49
3.56							
End of Borehole Practical refusal to augering at 3.56m depth (GWL @ 3.04m 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 14

FILE NO. PG4772

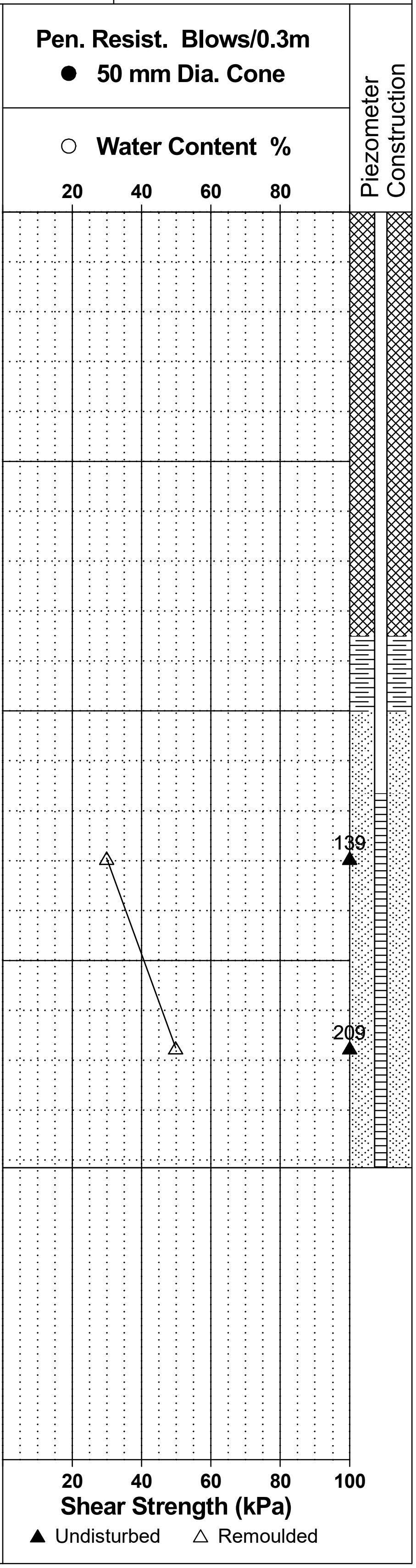
HOLE NO. BH 7

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.41
TOPSOIL 0.43	AU	1				
Very stiff to hard, brown **SILTY CLAY**	SS	2	58	7	1	102.41
- grey by 1.8m depth	SS	3	92	6	2	101.41
3.83					3	100.41

End of Borehole

Practical refusal to augering at 3.83m 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 14

FILE NO. PG4772

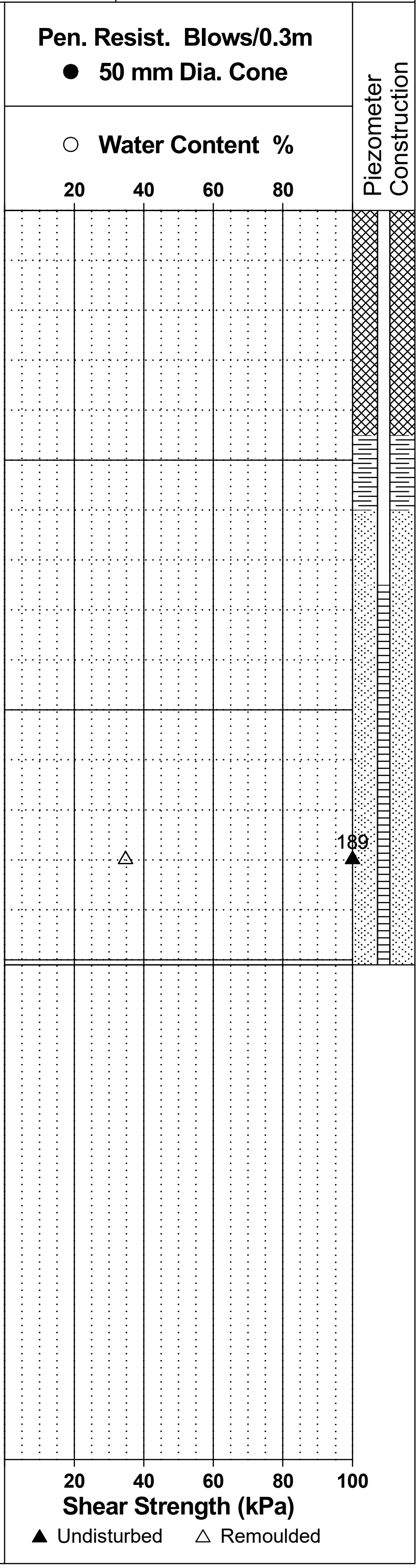
HOLE NO. BH 8

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

FILE NO. **PG4772**

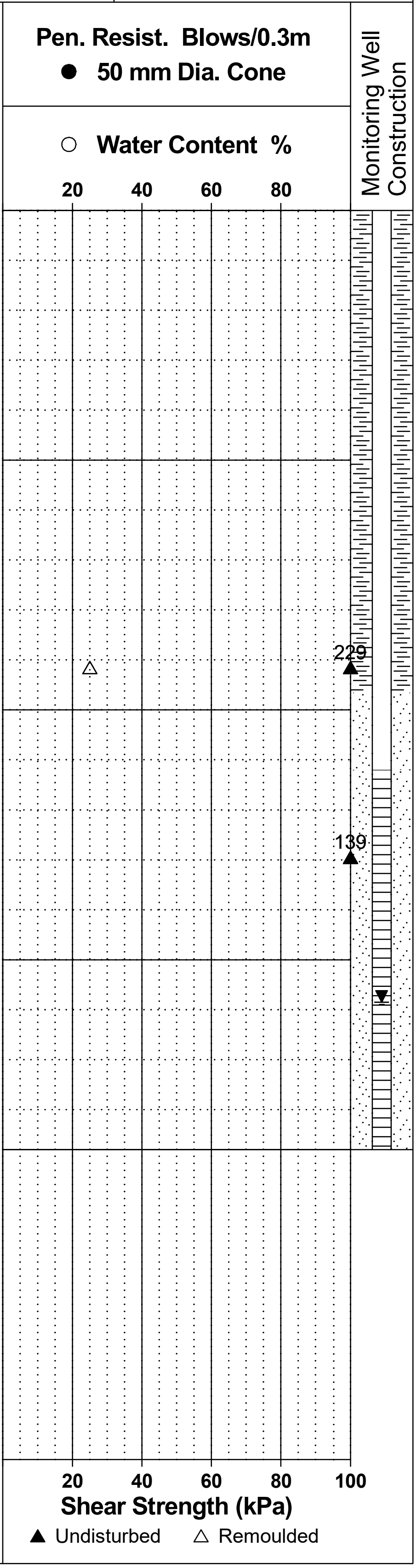
REMARKS

HOLE NO. **BH 9**

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	103.42
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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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

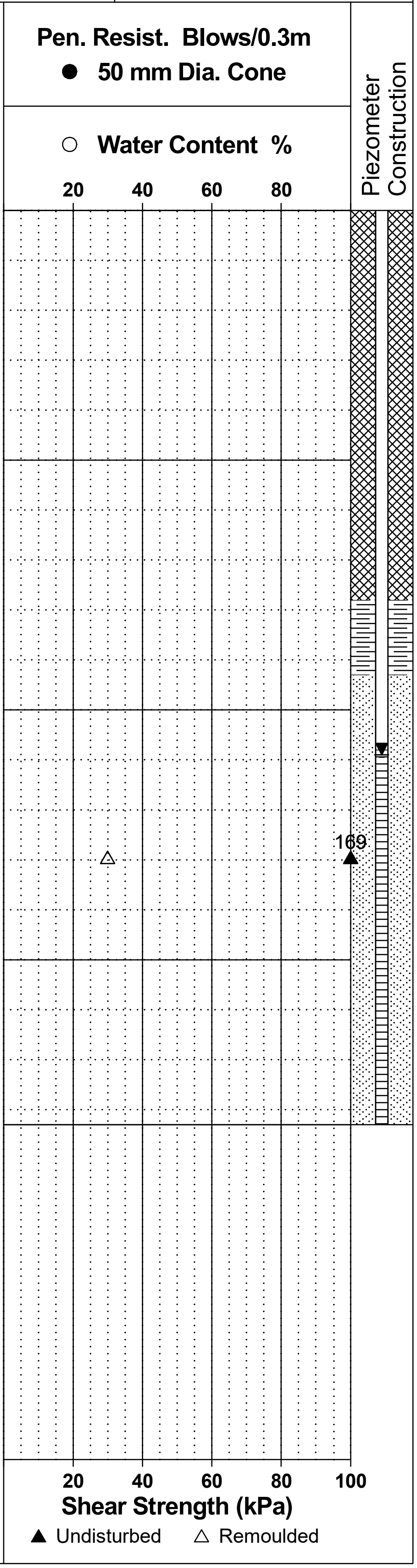
DATE 2019 January 15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
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	2	101.31
- grey by 2.1m depth							
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

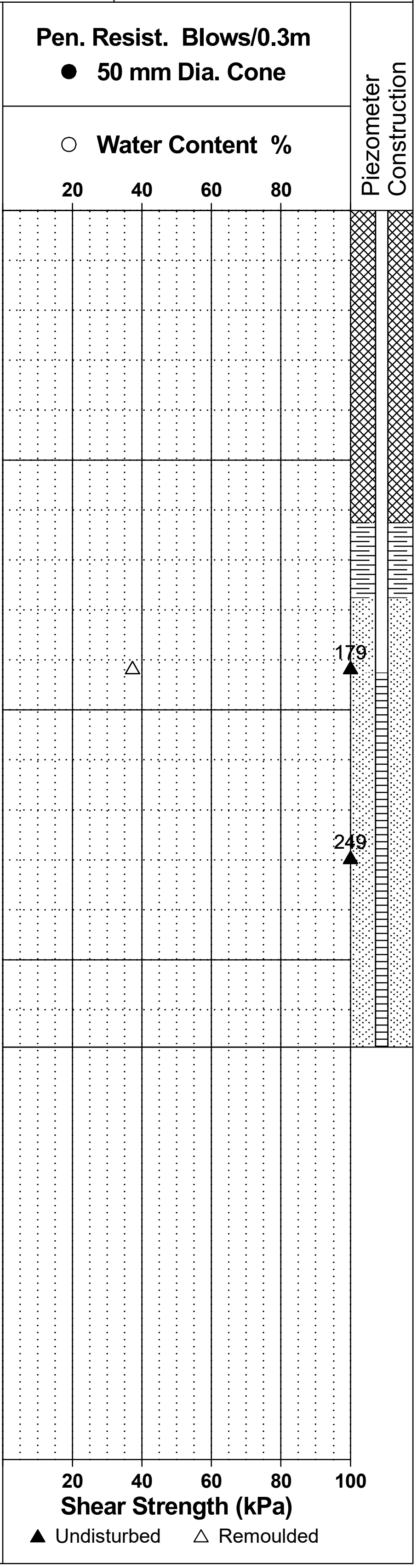
HOLE NO. BH11

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.44
TOPSOIL — 0.38	AU	1				
Very stiff, brown **SILTY CLAY**	SS	2	71	4	1	102.44
					2	101.44
— 3.05					3	100.44
GLACIAL TILL: Very dense brown to grey sandy silt, trace clay and gravel, occasional cobbles and boulders — 3.35	SS	3	100	50+		

End of Borehole

Practical refusal to augering at 3.35m 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

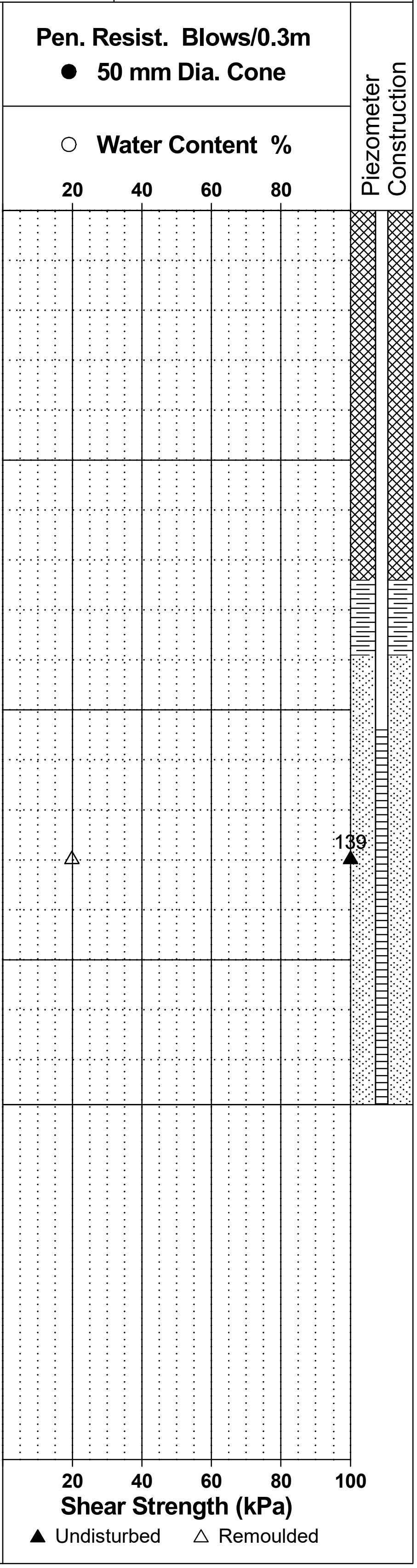
HOLE NO. BH12

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE 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**

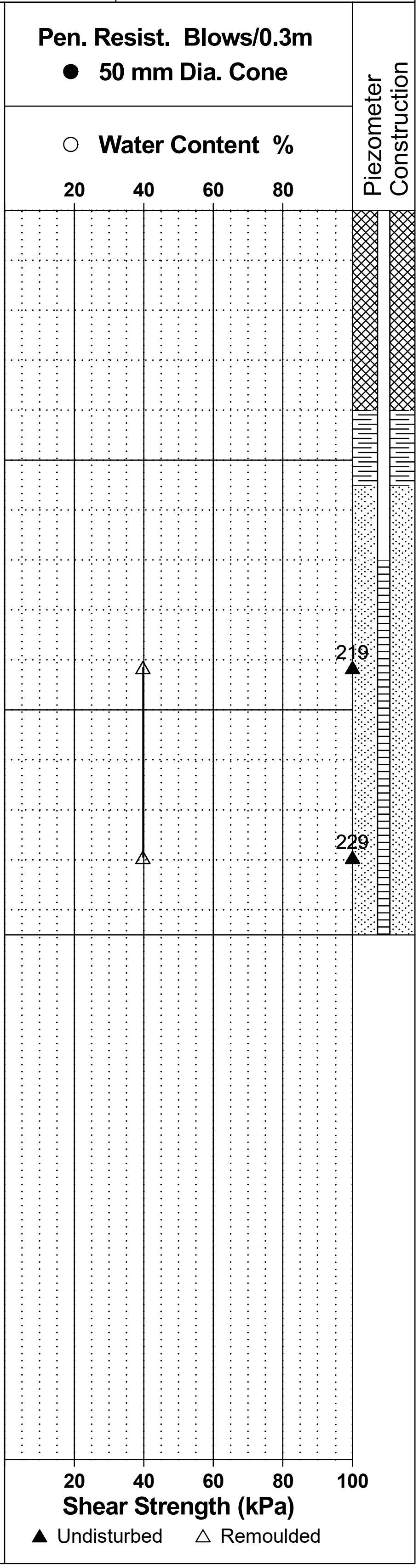
REMARKS

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.
Ottawa, Ontario

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FILE NO. **PG4772**

REMARKS

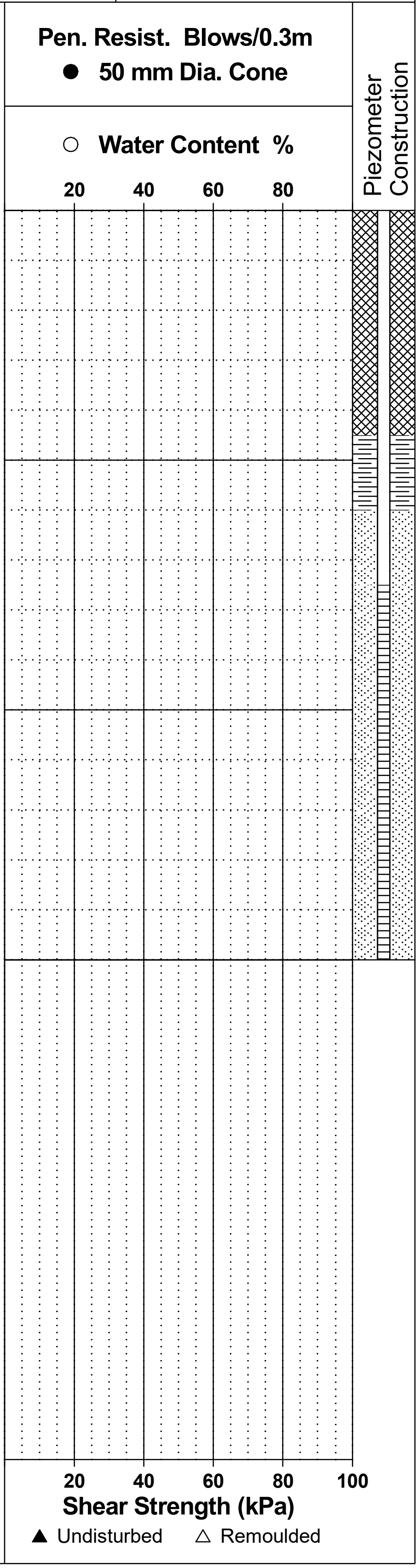
HOLE NO. **BH14**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

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

DEPTH (m)	ELEV. (m)
0	104.18
1	103.18
2	102.18
3	101.18



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

FILE NO. PG4772

HOLE NO. BH15

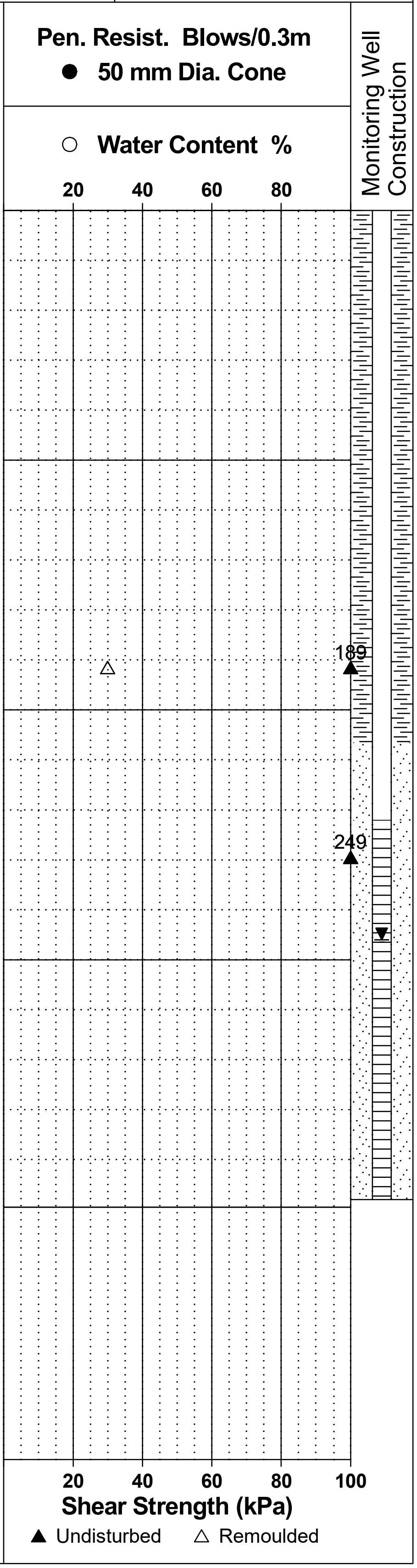
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.36		AU	1		
Very stiff, brown **SILTY CLAY** 2.29		SS	2	71	6
Hard, brown **CLAYEY SILT** 3.05					
GLACIAL TILL: Compact to very dense, grey clayey silt, some sand, trace gravel, occasional cobbles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

End of Borehole

Practical refusal to augering at 3.99m depth

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

DEPTH (m)	ELEV. (m)
0	103.65
1	102.65
2	101.65
3	100.65



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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 15

FILE NO. PG4772

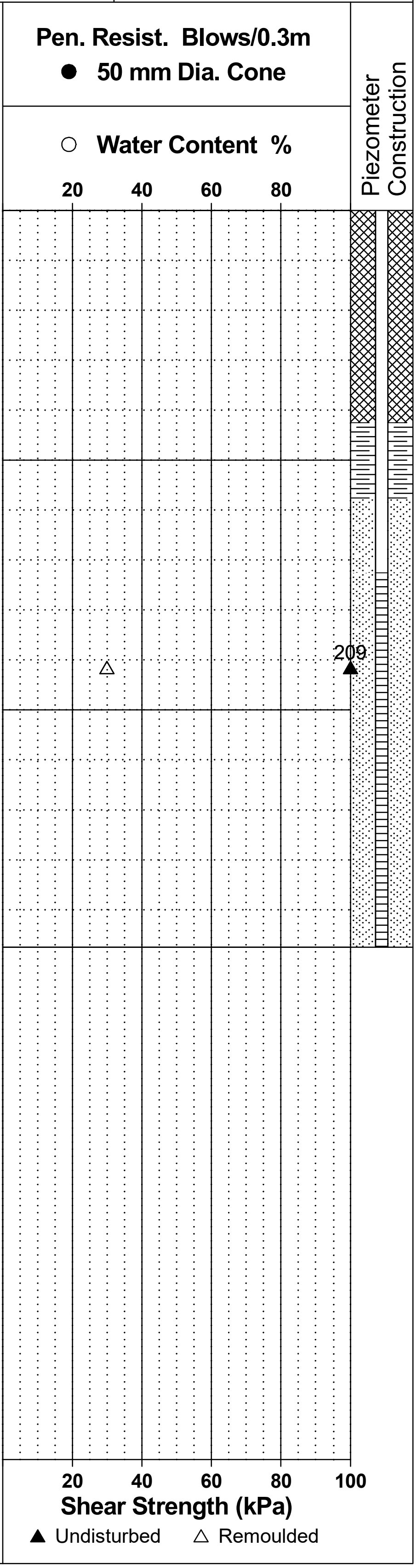
HOLE NO. BH16

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

End of Borehole

Practical refusal to augering at 2.95m depth

(BH Dry - Jan 29/19)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 20 Cedarow Ct.
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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

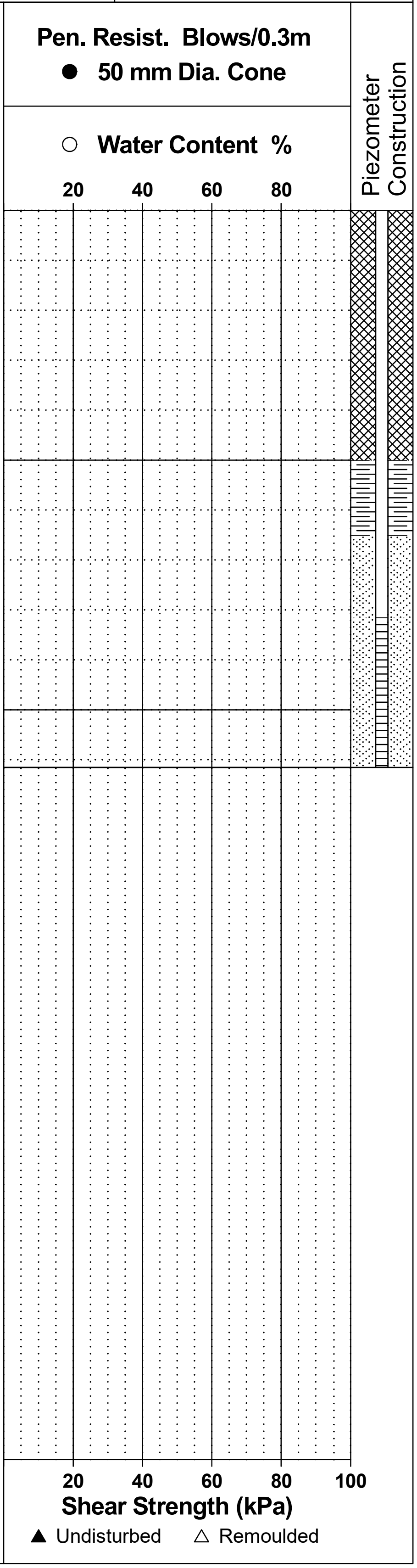
HOLE NO. BH17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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)



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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. **BH18**

BORINGS BY CME 55 Power Auger

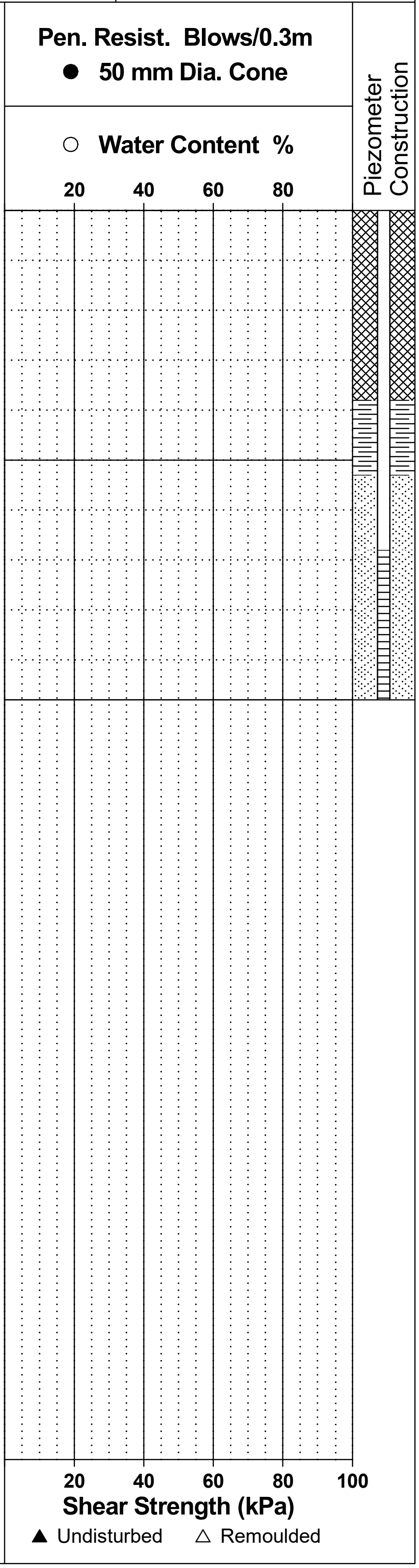
DATE 2019 January 16

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

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

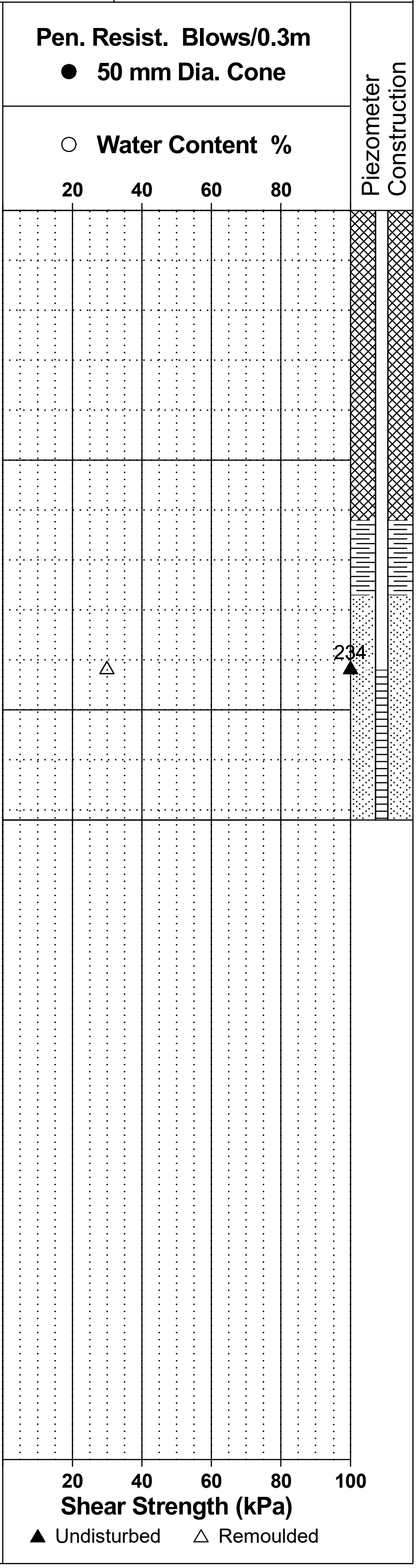
HOLE NO. BH19

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 (to 0.36)		AU	1				
Hard, brown to grey **SILTY CLAY**		SS	2	88	3	1	102.78
						2	101.78
(to 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

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FILE NO. PG4772

REMARKS

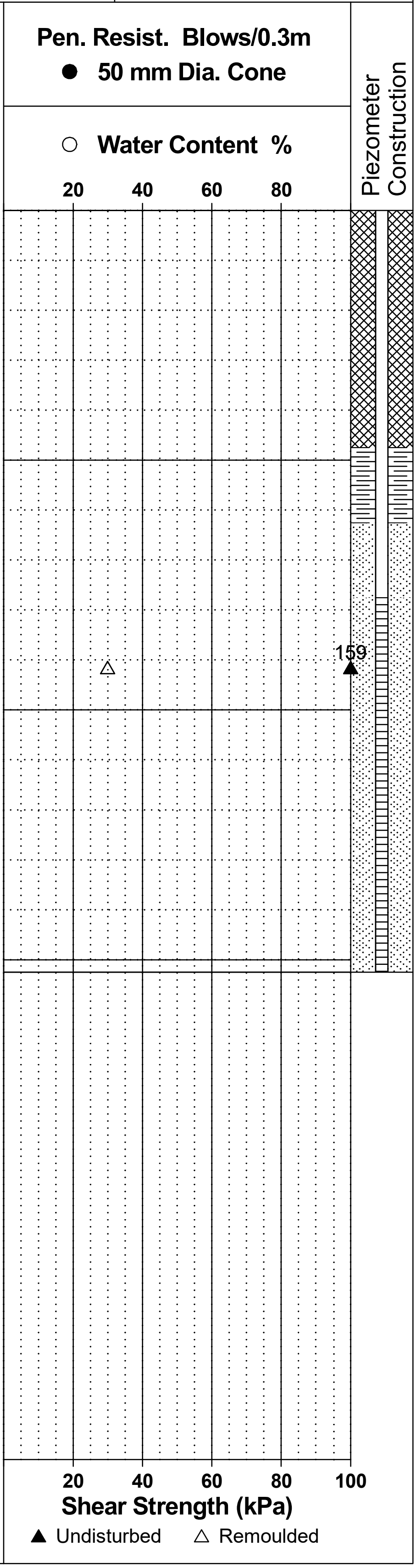
HOLE NO. BH20

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.33	AU	1		
Very stiff, brown **SILTY CLAY**	SS	2	83	4
- grey by 1.8m depth 2.30				
Loose, grey **CLAYEY SILT,** trace sand and gravel 3.05	SS	3	83	9
End of Borehole Practical refusal to augering at 3.05m depth (BH Dry - Jan 29/19)				

DEPTH (m)	ELEV. (m)
0	103.59
1	102.59
2	101.59
3	100.59



SOIL PROFILE AND TEST DATA

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

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FILE NO. **PG4772**

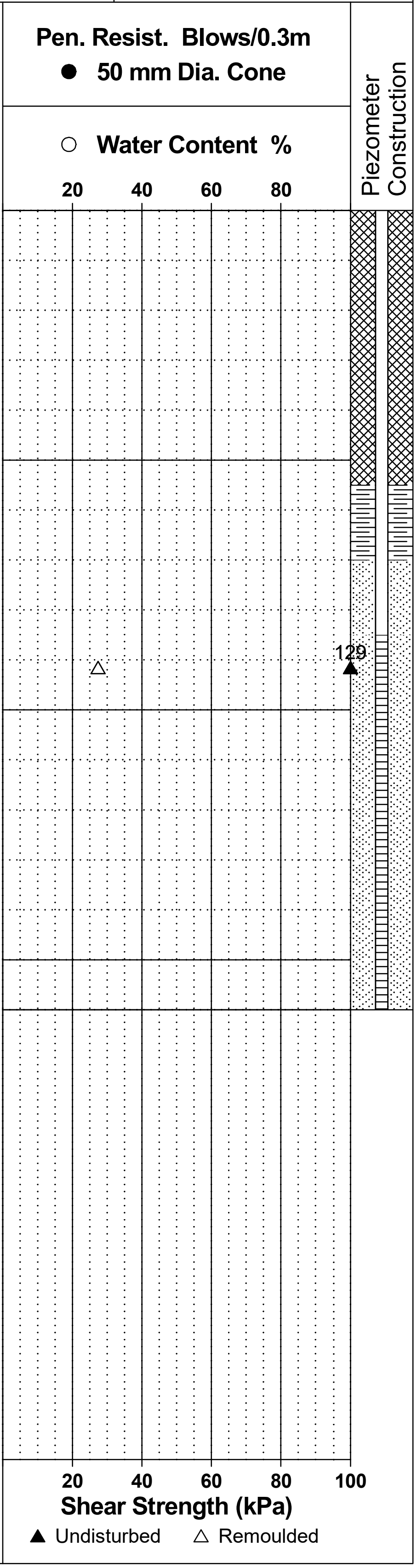
REMARKS

HOLE NO. **BH21**

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	103.58
TOPSOIL 0.33	AU	1				
Very stiff, brown **SILTY CLAY**	SS	2	79	5	1	102.58
- grey by 1.8m depth					2	101.58
2.29						
GLACIAL TILL: Compact to very dense, brown to grey sandy silt, some clay, gravel, cobbles and boulders	SS	3	71	13	3	100.58
3.20	SS	4	100	50+		
End of Borehole Practical refusal to augering at 3.20m 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

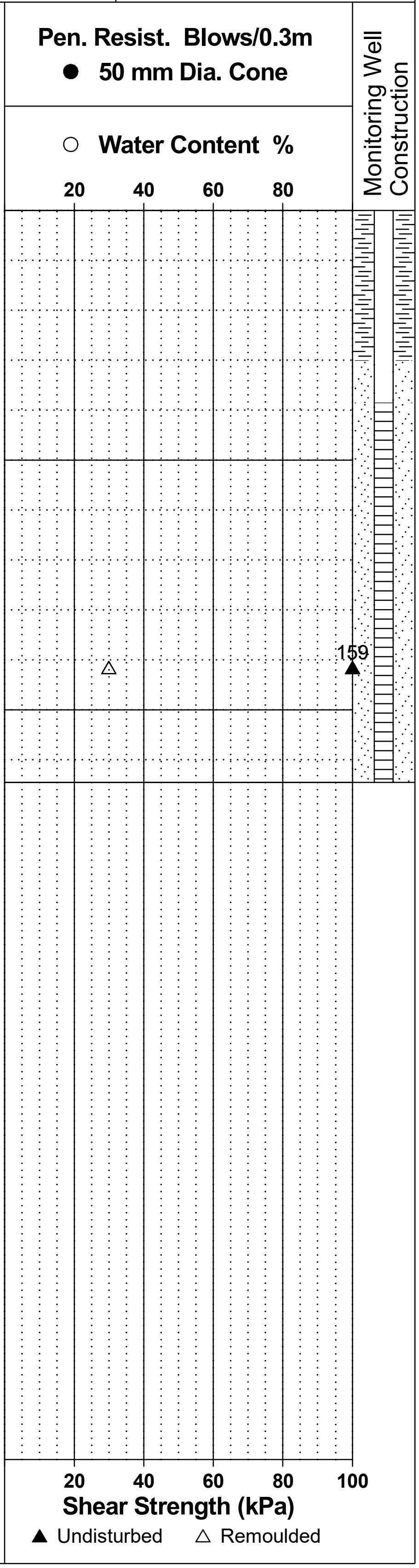
BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

HOLE NO. 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)							



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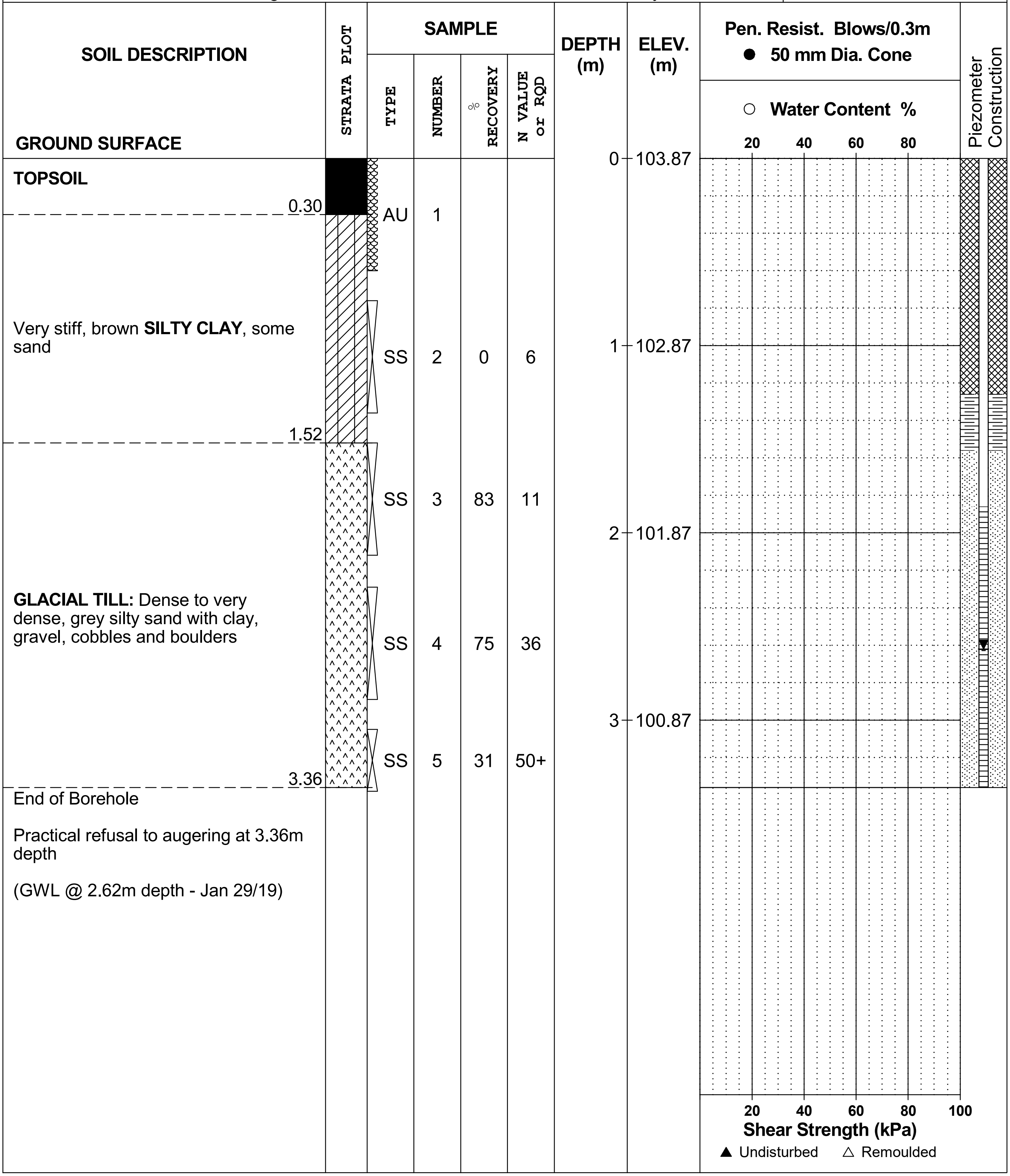
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH23**

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	103.87
TOPSOIL ... 0.30	AU	1				
Very stiff, brown **SILTY CLAY**, some sand ... 1.52	SS	2	0	6	1	102.87
GLACIAL TILL: Dense to very dense, grey silty sand with clay, gravel, cobbles and boulders ... 3.36	SS	3	83	11	2	101.87
	SS	4	75	36		
	SS	5	31	50+	3	100.87

End of Borehole

Practical refusal to augering at 3.36m depth

(GWL @ 2.62m 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

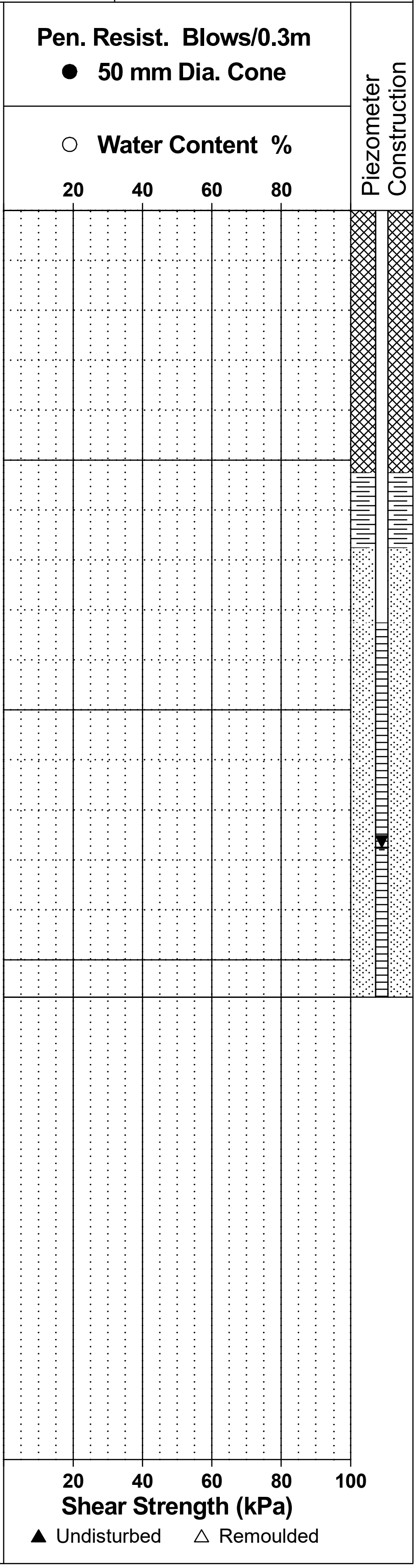
BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. PG4772

HOLE NO. BH24

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.04
TOPSOIL	AU	1				
0.36						
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	101.04
3.15	SS	5	100	50+		
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

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

REMARKS

HOLE NO. **BH25**

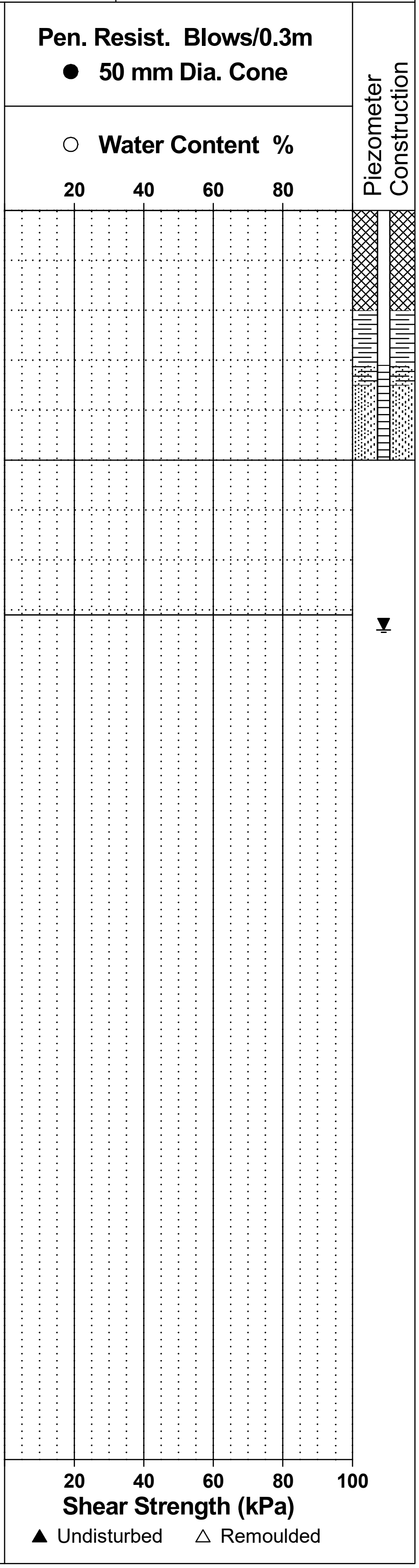
BORINGS BY CME 55 Power Auger **DATE** 2019 January 16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.07
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)



SOIL PROFILE AND TEST DATA

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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

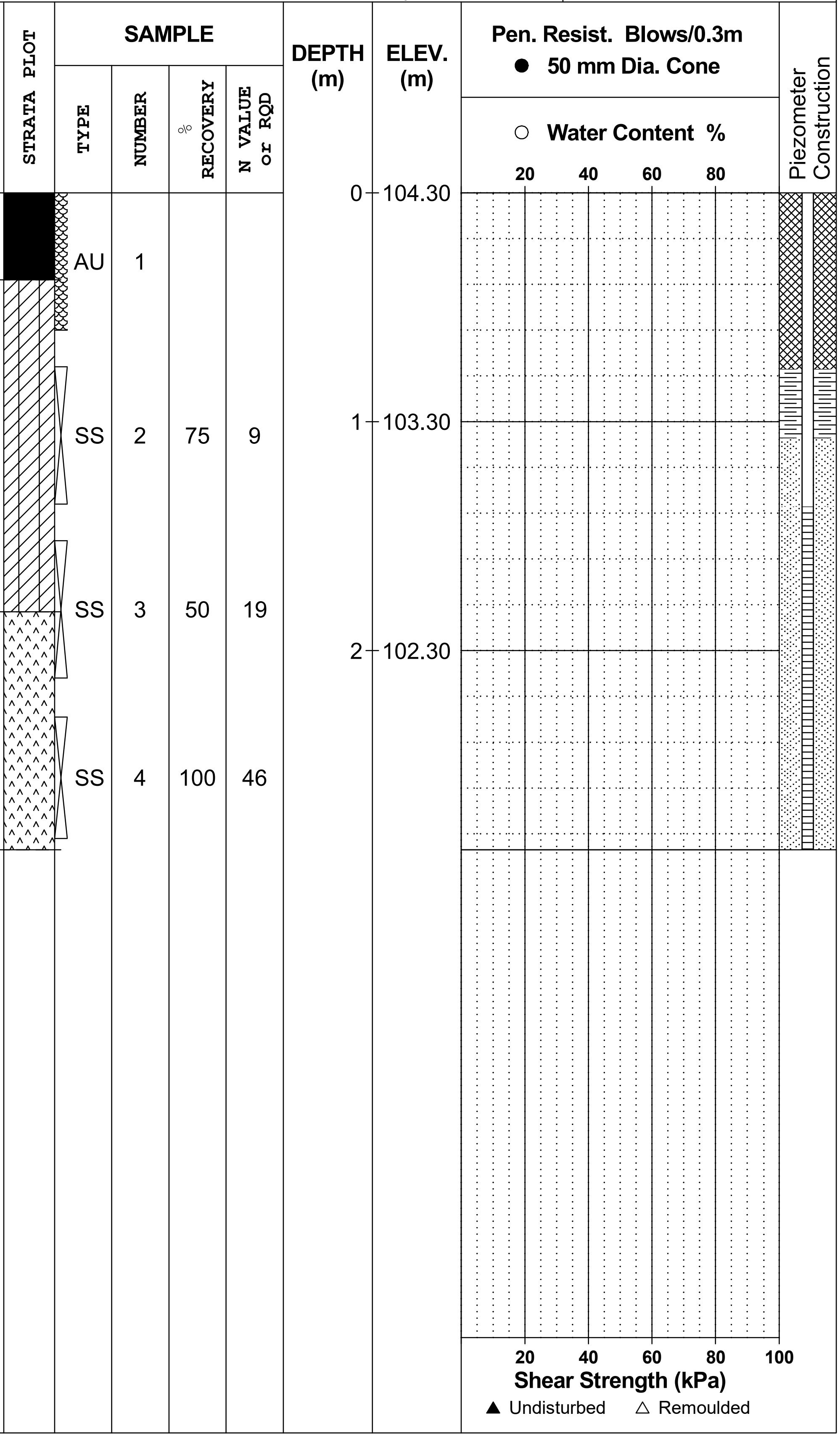
HOLE NO. BH26

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.30
TOPSOIL (to 0.38)	AU	1				
Very stiff, brown **CLAYEY SILT** (0.38 to 1.83)	SS	2	75	9	1	103.30
	SS	3	50	19		
GLACIAL TILL: Compact to dense, grey silty clay with gravel, cobbles and boulders (1.83 to 2.87)					2	102.30
	SS	4	100	46		
End of Borehole (2.87)						

End of Borehole

Practical refusal to augering at 2.87m depth

(BH Dry - Jan 29/19)



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.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

FILE NO. **PG4772**

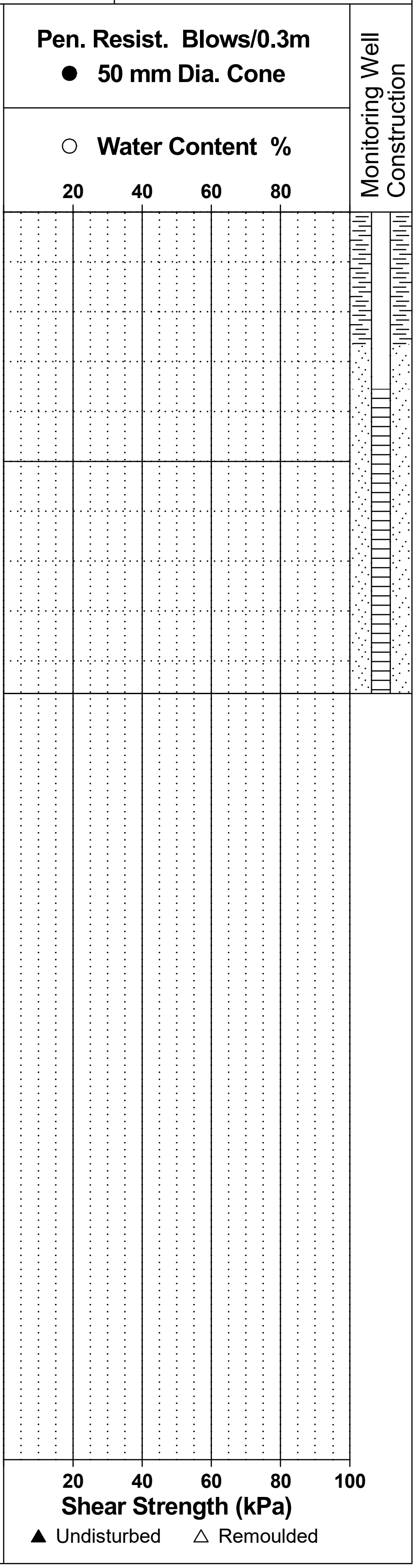
HOLE NO. **BH27**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE 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)



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.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

FILE NO. PG4772

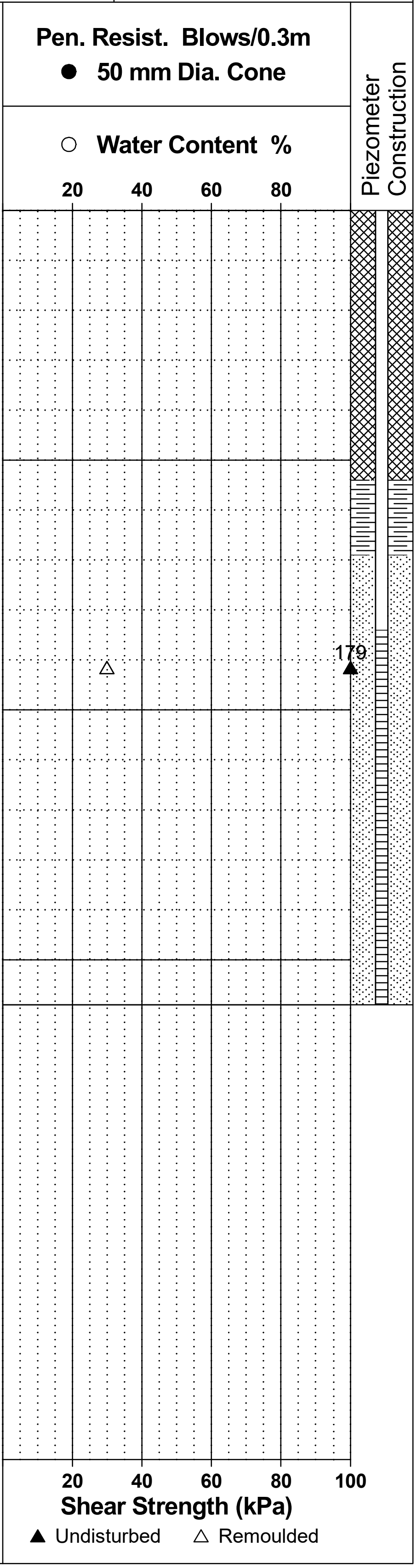
HOLE NO. BH28

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	103.78
TOPSOIL (to 0.36)	AU	1				
Very stiff, brown **SILTY CLAY** (0.36 to 2.29)	SS	2	38	6	1	102.78
					2	101.78
GLACIAL TILL: Loose to very dense, grey silty clay with sand, gravel, cobbles and boulders (2.29 to 3.18)	SS	3	8	2		
	SS	4	0	50+	3	100.78

End of Borehole

Practical refusal to augering at 3.18m depth

(BH Dry - Jan 29/19)



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.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

FILE NO. PG4772

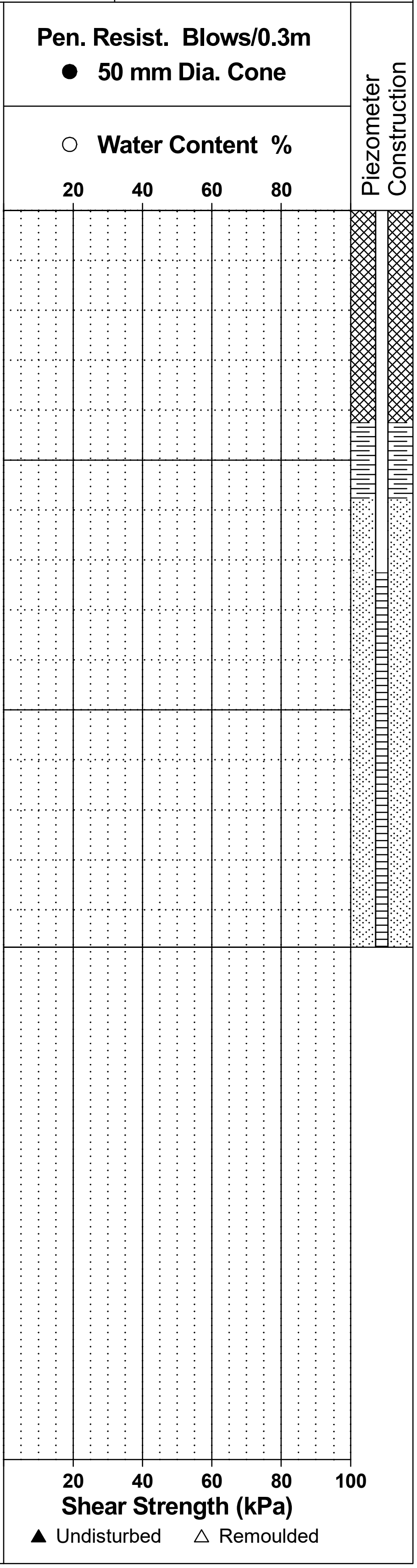
HOLE NO. BH29

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

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)

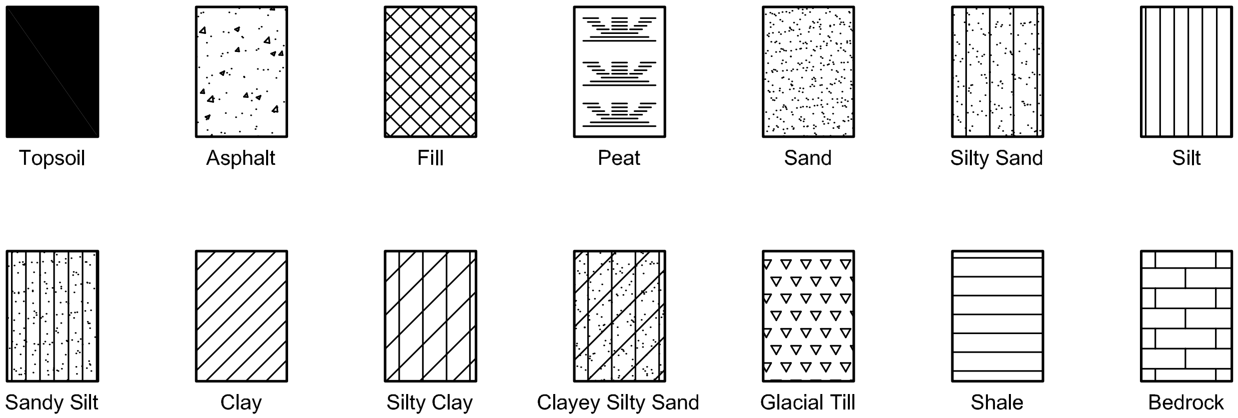
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

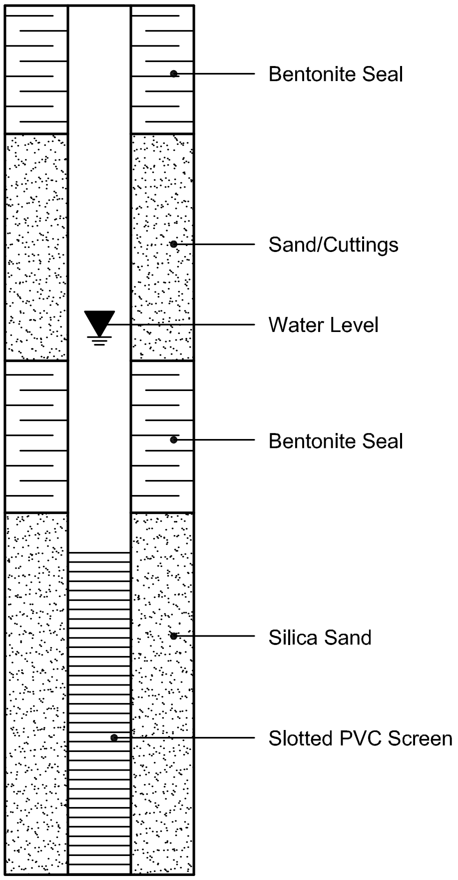
SYMBOLS AND TERMS (continued)

STRATA PLOT

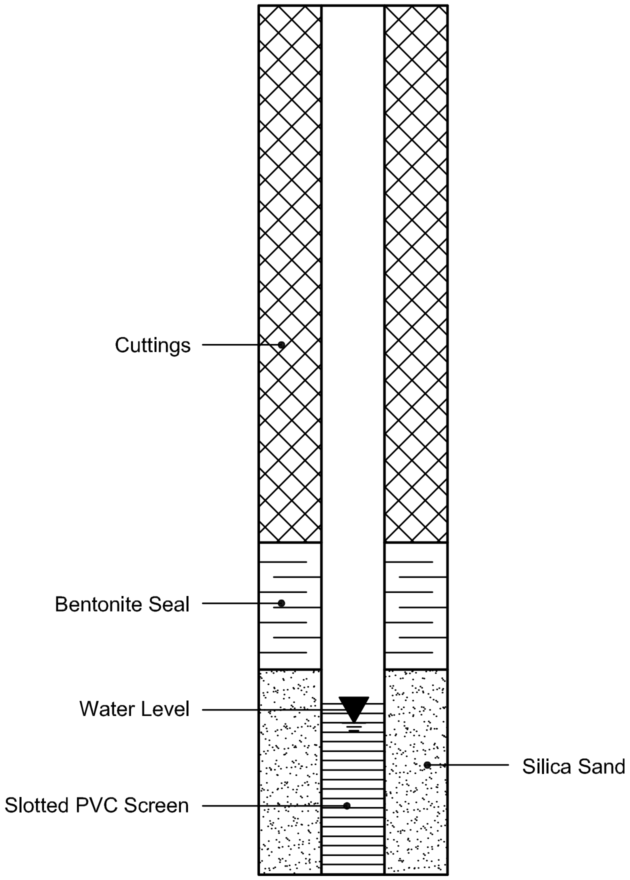


MONITORING WELL AND PIEZOMETER CONSTRUCTION

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

FIGURES 5 & 6 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4772-1 - TEST HOLE LOCATION PLAN

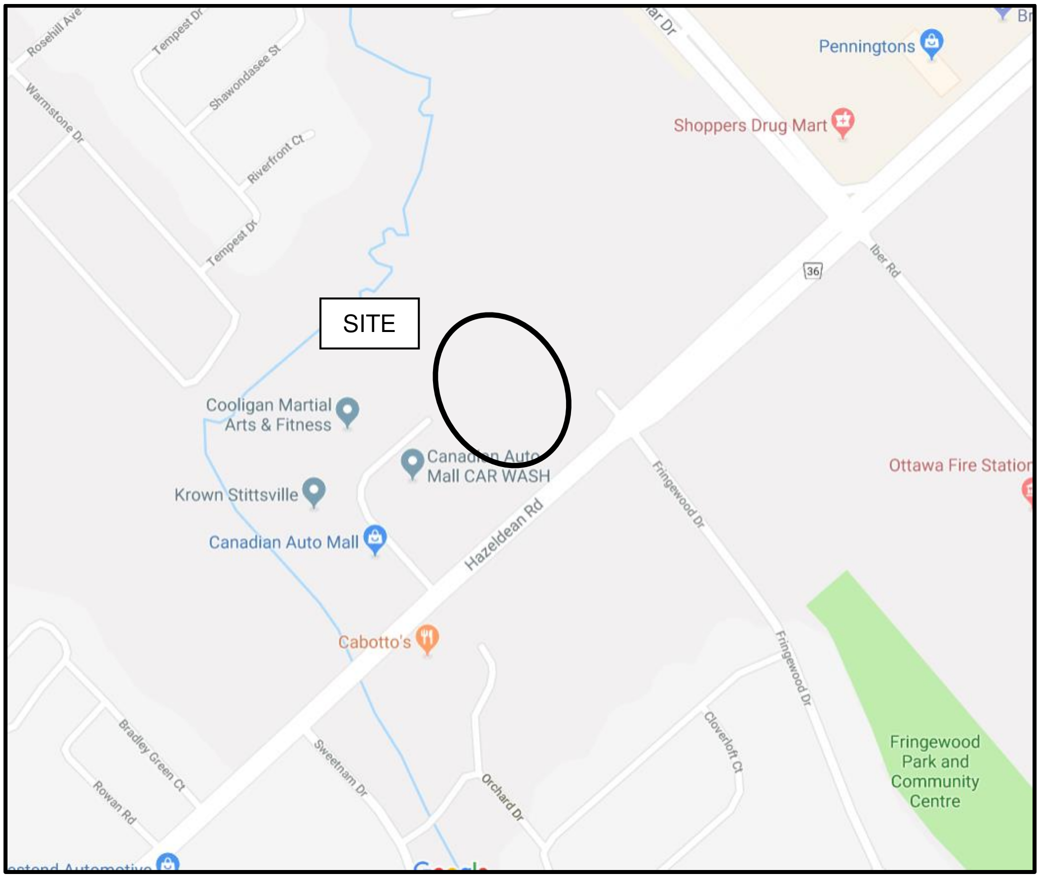


FIGURE 1

KEY PLAN

Figure 2A - Section A - Static Analysis

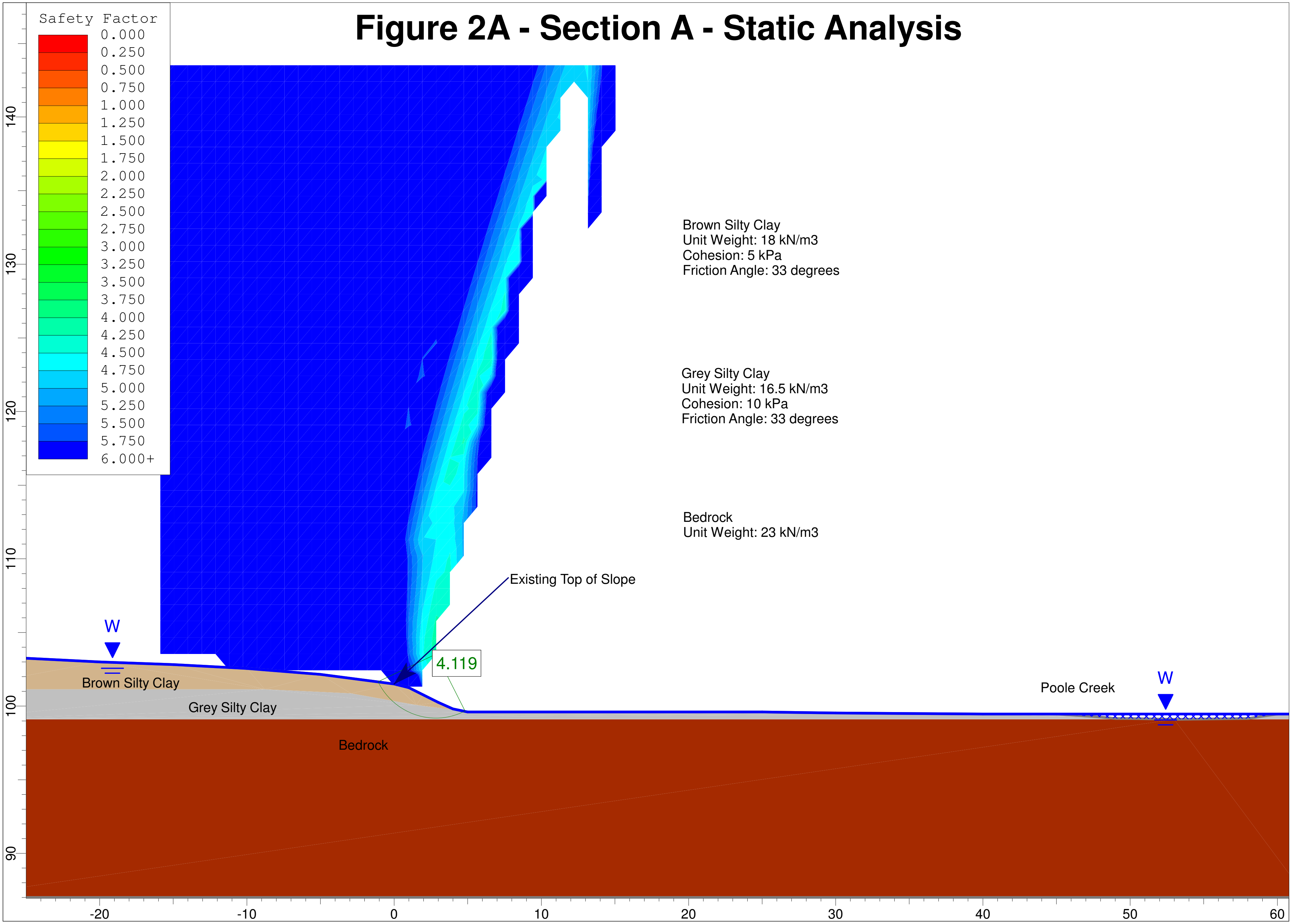


Figure 2B - Section A - Seismic Loading

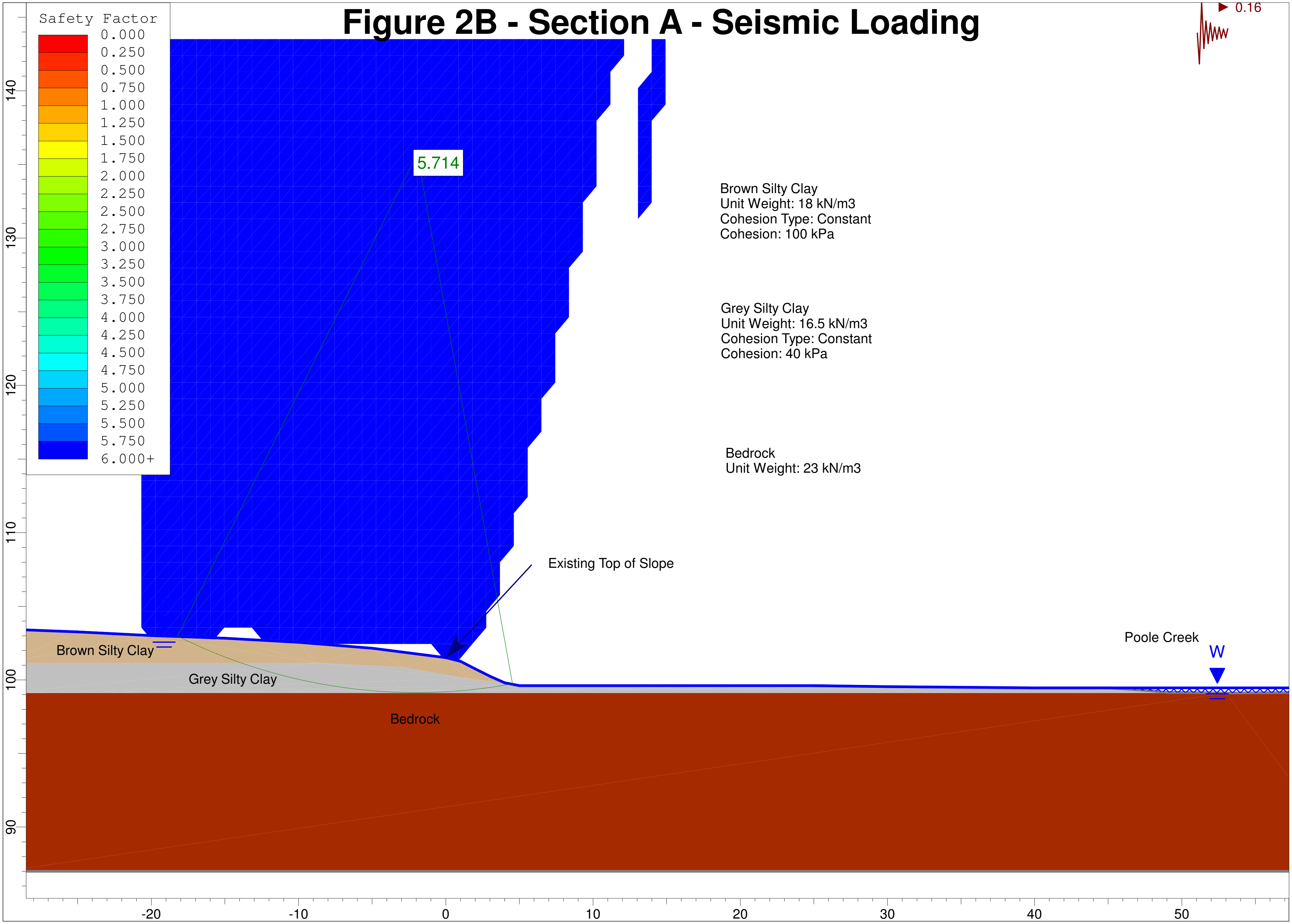
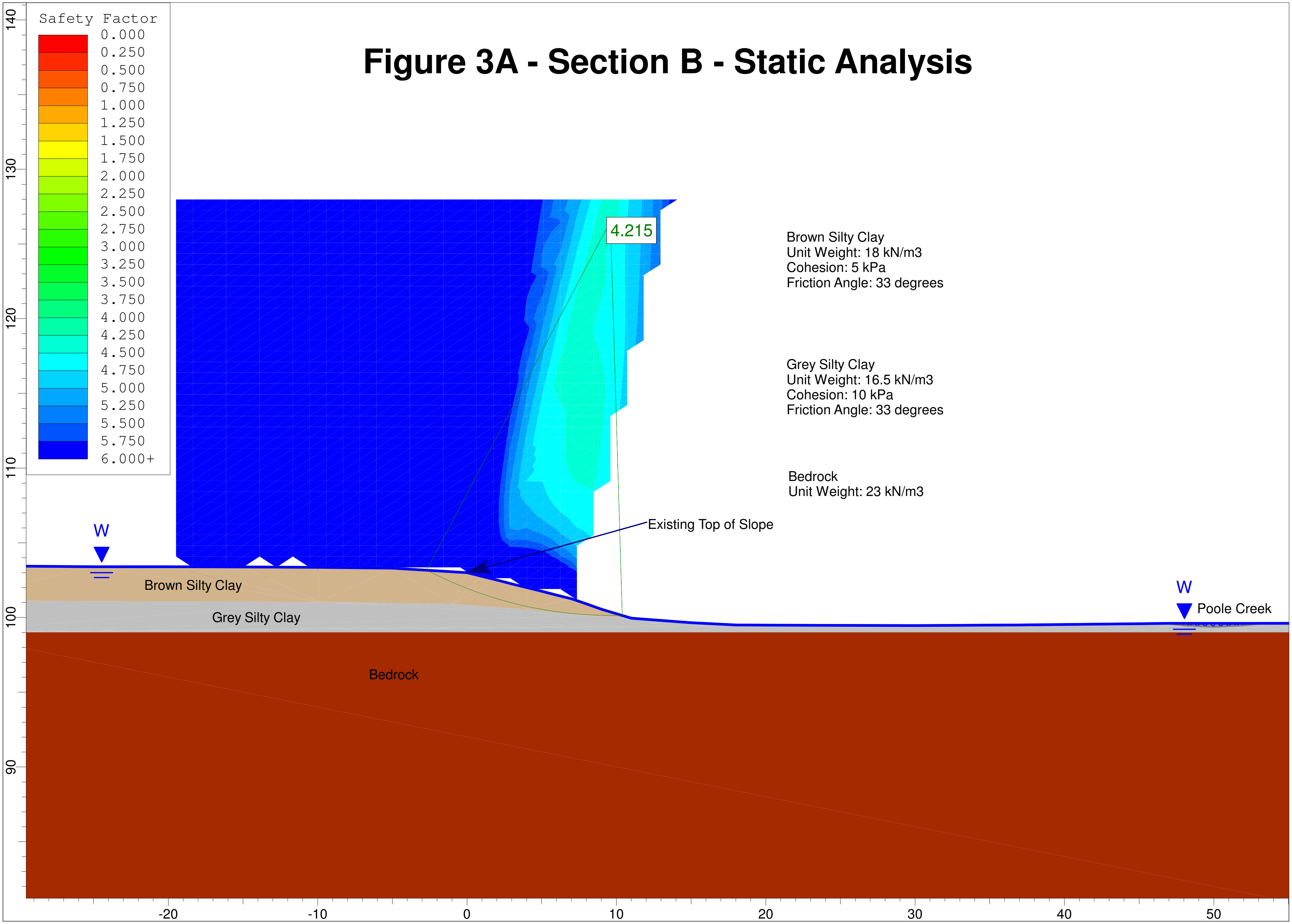


Figure 3A - Section B - Static Analysis

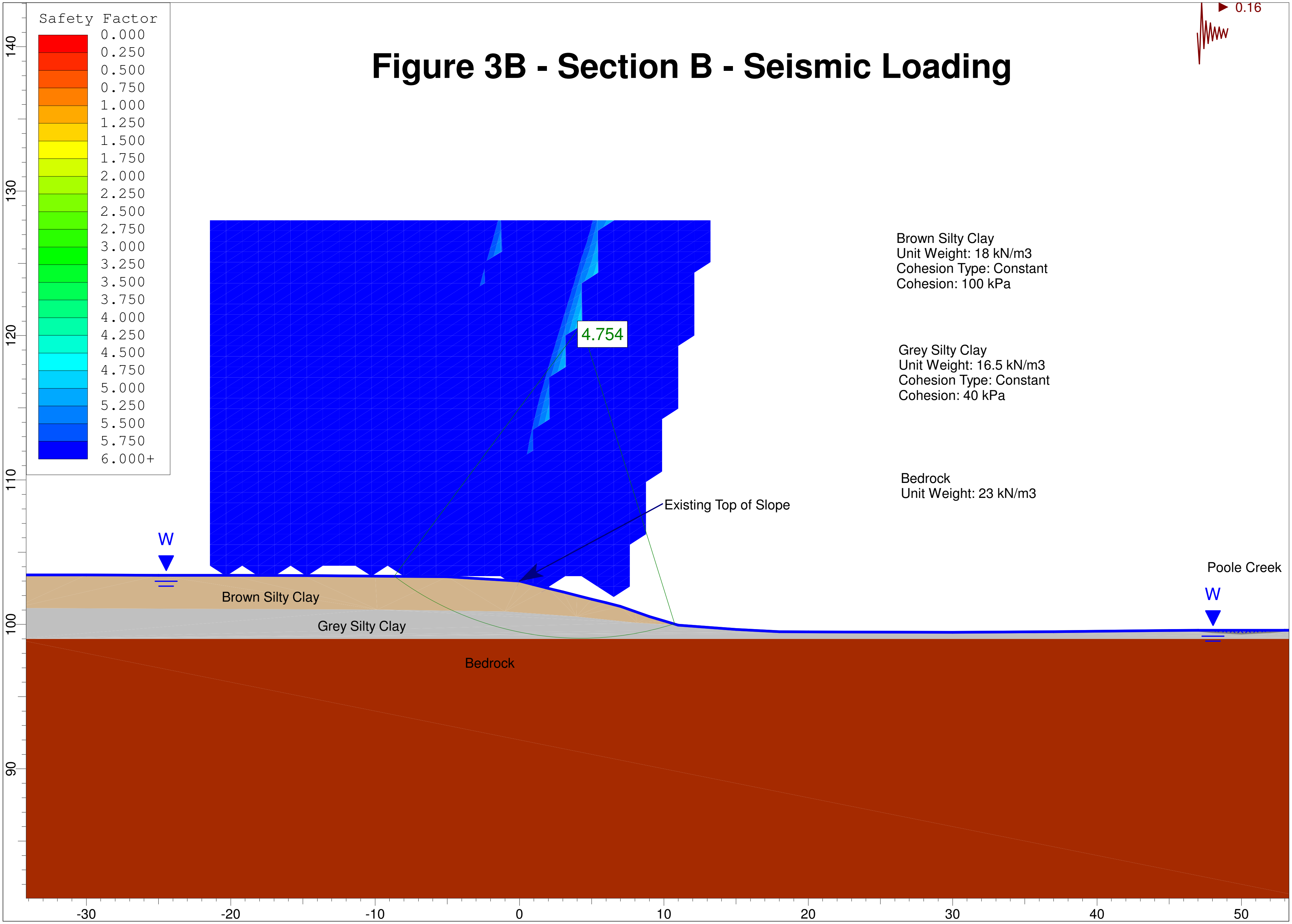


Brown Silty Clay
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees

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



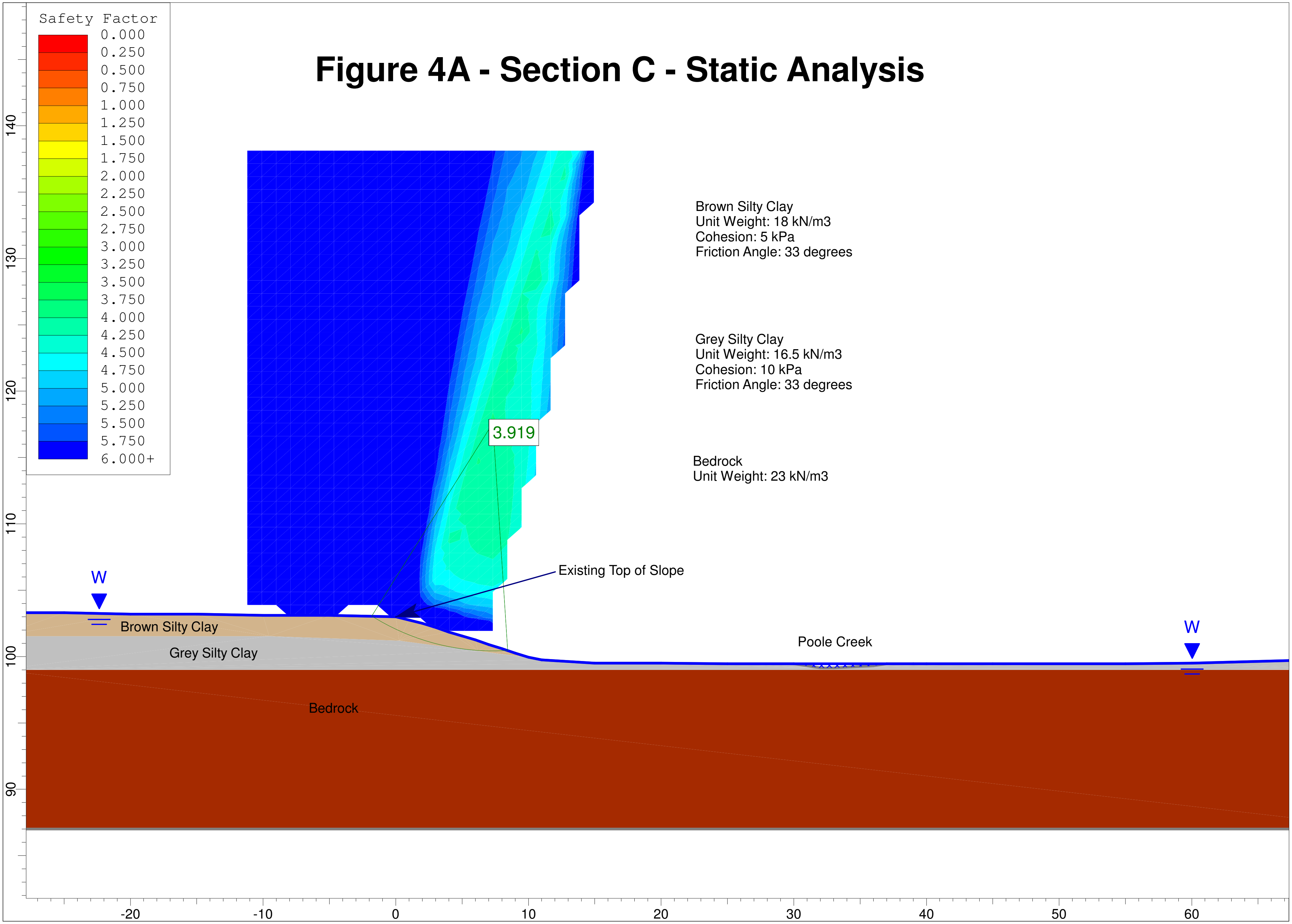


Figure 4A - Section C - Static Analysis

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

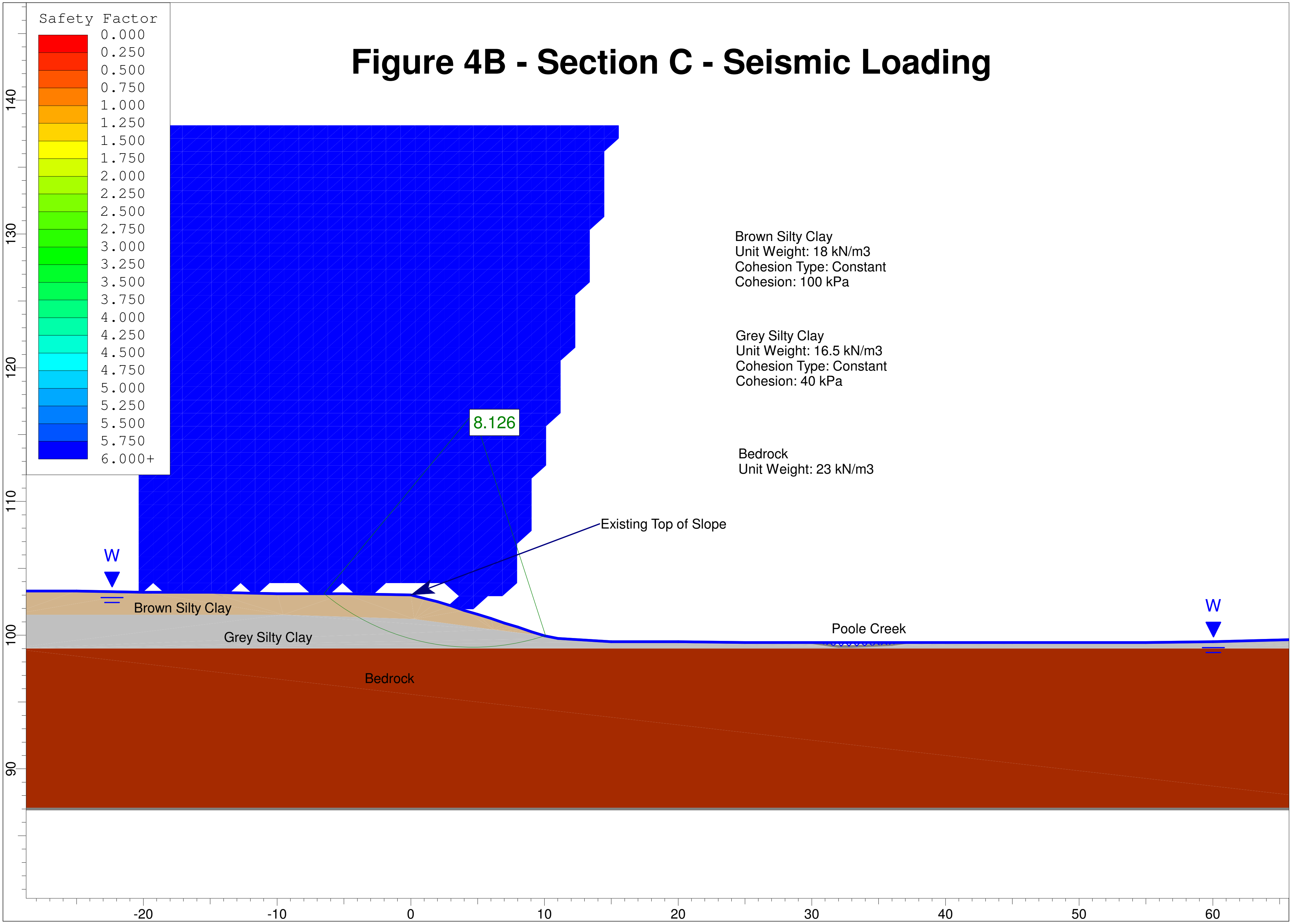


Figure 4B - Section C - Seismic Loading

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

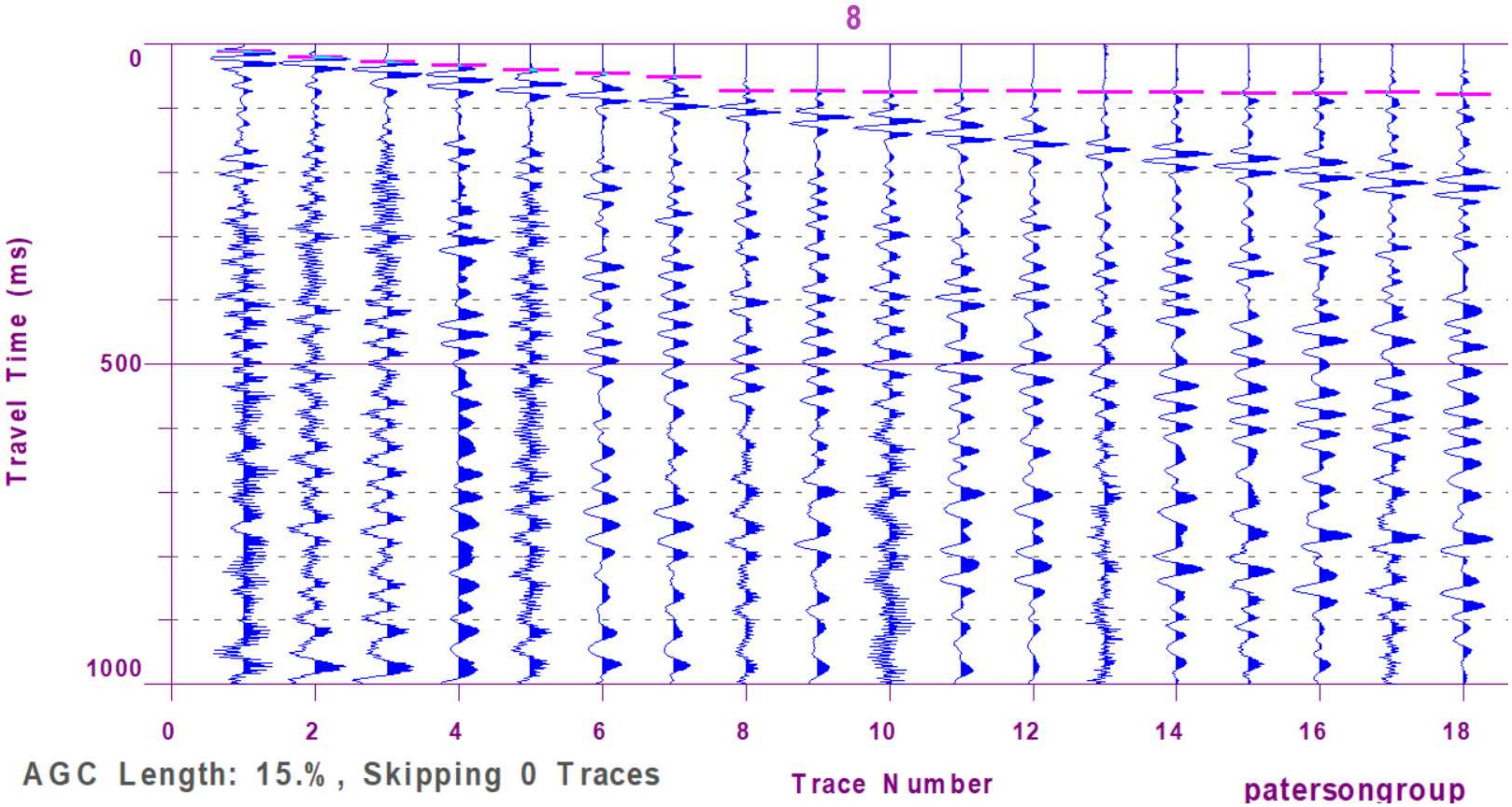


Figure 5 – Shear Wave Velocity Profile at Shot Location -1 m

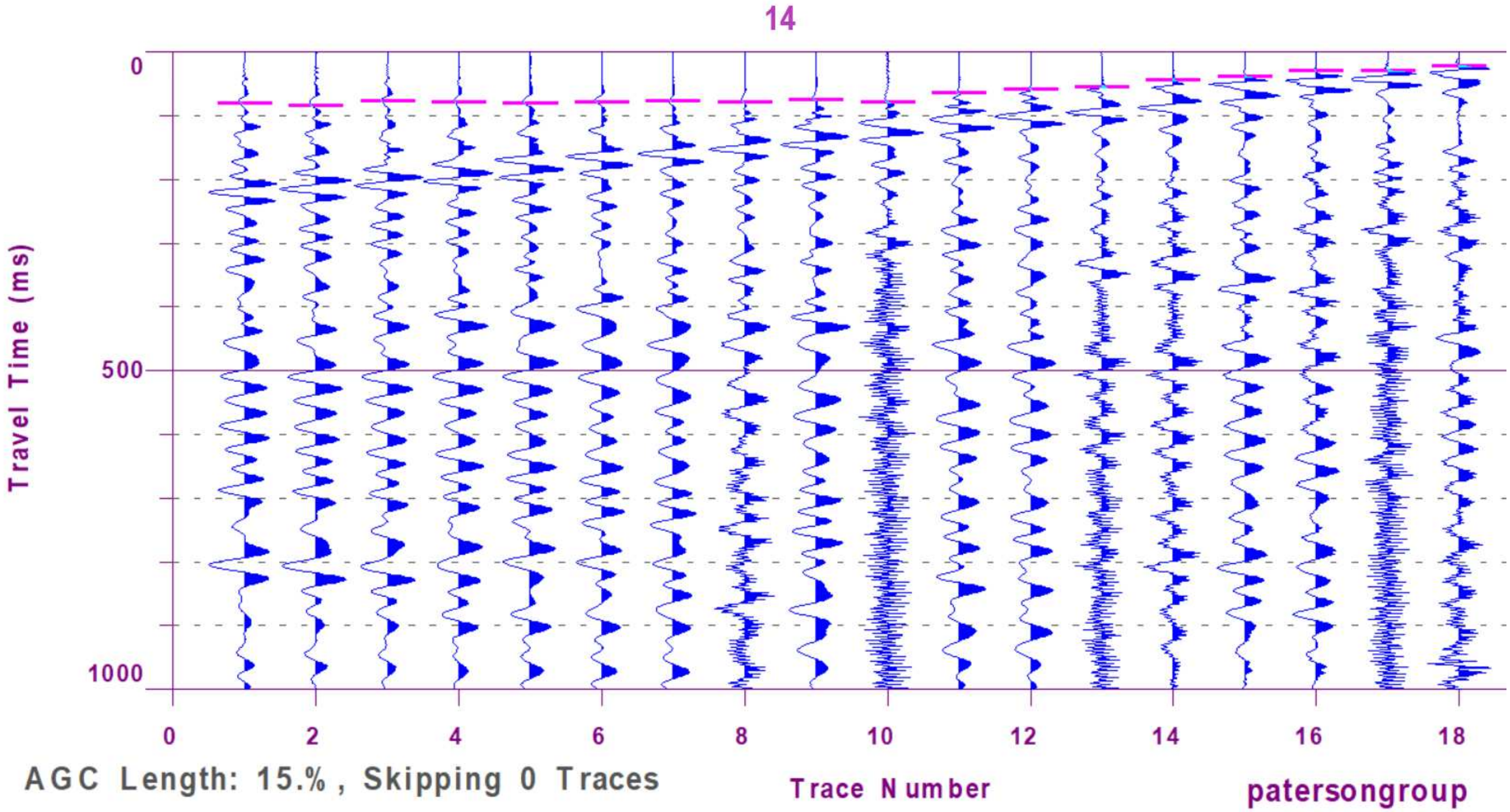
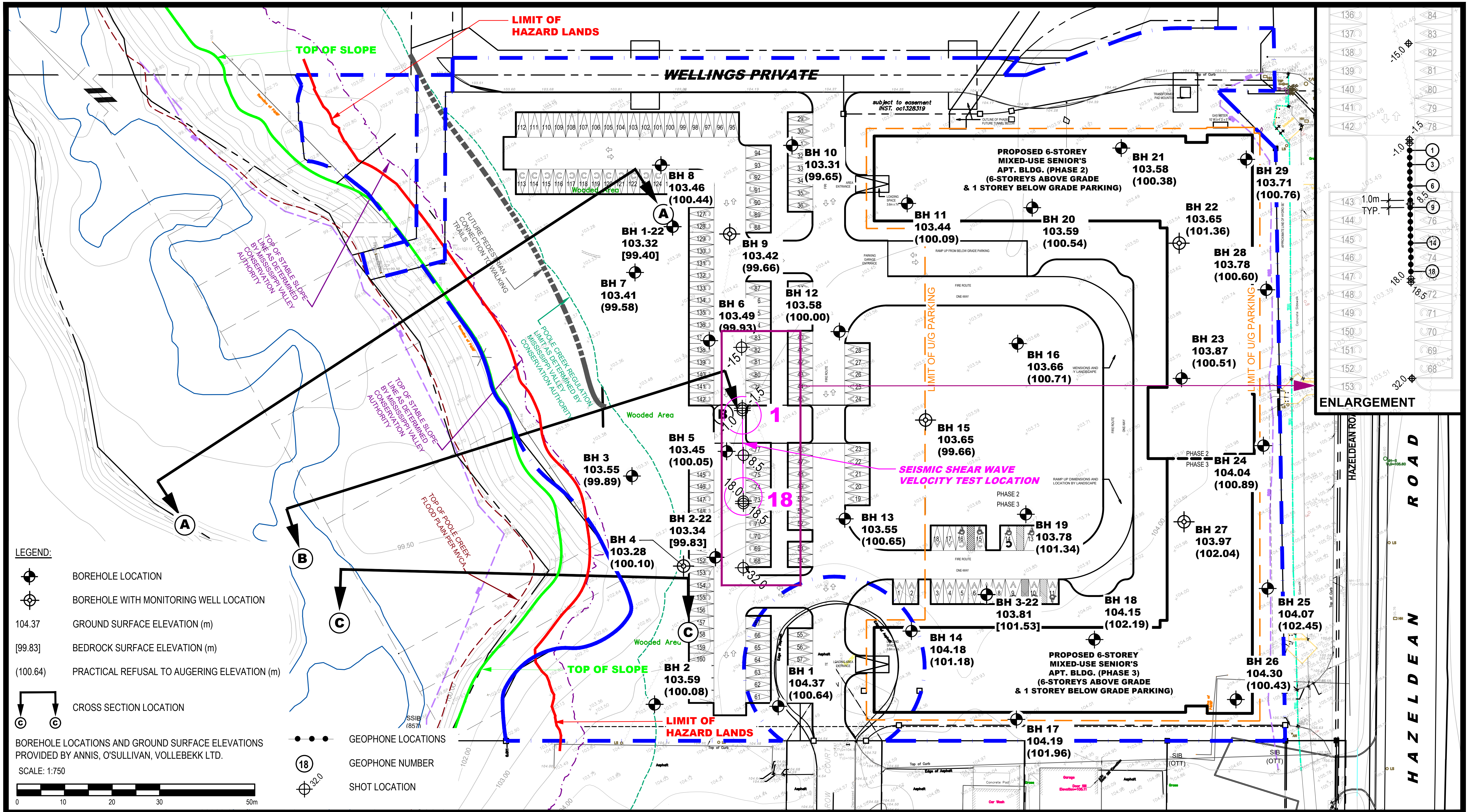


Figure 6 – Shear Wave Velocity Profile at Shot Location 1.5 m



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

0 10 20 30 50m

● ● ● GEOPHONE LOCATIONS

⑱ GEOPHONE NUMBER

⊕ 32.0 SHOT LOCATION

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
5	SEISMIC SHEAR WAVE VELOCITY ADDED TO PLAN	07/03/2023	FC
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.: 5

p:\autocad drawings\geotechnical\pg47xx\pg4772\pg4772-1 thlp (rev.05).dwg

APPENDIX 3

RELEVANT MEMORANDUMS

patersongroup
consulting engineers

memorandum

re: **Geotechnical Response to City of Ottawa Comments**
Proposed Mixed-Use Development - Wellings of Stittsville Phase 2
20 Cedarow Court - Ottawa

to: Nautical Lands Group - **Ms. Angela Mariani** - angela@nlgc.com

date: September 29, 2020

file: PG4772-MEMO.01

As requested, Paterson Group (Paterson) prepared the current memorandum to provide geotechnical responses to the City of Ottawa engineering comments for the proposed mixed-use development to be located at the aforementioned site. This memorandum should be read in conjunction with Paterson Report PG4772-1 Revision 1 dated September 29, 2020.

Geotechnical Comments

Item B19

Comment: *Paterson Group shall submit a memo to the City of Ottawa signing off on the Grading Plan prepared by Stantec to verify that the grading is acceptable from a geotechnical perspective and the proposal is in conformance with the permissible grade raise restrictions, recommendations and statements of the latest Geotechnical Investigation.*

Response: Please refer to Paterson's grading plan review memo Report PG4772-MEMO.02 dated September 29, 2020, for grading plan review and approval.

Item B20:

Comment: *A deep excavation and dewatering operations have the potential to cause damages to the neighbouring adjacent buildings/ City infrastructure. Document that construction activities (excavation, dewatering, vibrations associated with construction, etc.) will not have an impact on any adjacent buildings and infrastructure.*

Response: Based on our observations, the groundwater level is anticipated at a 2.5 to 3 m depth and within the glacial till layer and bedrock. Therefore, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings and services. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded within native glacial till and/or directly over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures or city infrastructures surrounding the proposed multi-storey development.

Item B21:

Comment: *Investigate the effect of short-term and long-term lowering of the groundwater level and the impact on the adjacent lands and existing neighbouring structures. The impact of groundwater lowering on adjacent properties needs to be discussed and investigated to ensure there will be no short term and long-term damages associated with lowering the groundwater in this area.*

Response: Please refer to our response to Item B20, no impacts are expected from short and long term groundwater lowering.

Item B22:

Comment: *Due to the consideration of blasting as part of the excavation processes, a pre-blast survey and report are required and will be part of the conditions of SPA. Monitoring of all sewers and watermains, will be required coupled with pre and post CCTV sewer surveys. Document the monitoring requirements in the report.*

Response: A preconstruction survey program will be completed by Paterson prior to the commencement of construction of the proposed development.

Item B23

Comment: *The site plan identifies locations where 6-storeys are proposed. For Section 2.0 – Proposed Development, revise and ensure the geotechnical discussion and recommendations are valid based on the proposed development.*

Response: Please refer to Section 2 in the revised geotechnical report PG4772-1 Revision 1 dated September 29, 2020. It is important to note that the additional storey does not impact our original recommendations for the proposed development.

Item B24:

Comment: *The report should be speaking in more detail to impacts on neighbouring properties with respect to foundation shoring, groundwater lowering, etc, now that more detailed plans for the proposed underground parking area are available.*

Response: Please refer to Subsection 6.5 in our revised geotechnical Report PG4772-1 Revision 1 dated September 29, 2020. It should be noted that any temporary shoring system is expected to be located outside the lateral support zone of neighboring structures. Therefore, no negative impacts are expected as a result of the shoring system installation.

Item B25:

Comment: *The report identifies a permissible grade raise restriction of 2m for areas where settlement sensitive structures are founded over the silty clay deposit. These areas need to be defined for coordination with the site grading plan.*

Response: The permissible grade raise restriction should be applied for the entire site. Once the site grading plan is available, Paterson will review the grading plans from a geotechnical perspective and will provide a sign-off, if the grading is in compliance with the permissible grade raise restrictions.

Item B26:

Comment: *Please provide discussion on the measured groundwater level readings in relation to the proposed underside of footing for the proposed buildings. If the foundation is in the proximity of the groundwater table elevation, consideration should be made for a watertight foundation.*

Response: The longterm groundwater table was found t be located at an elevation between 100.5 and 101.5 m throughout the site. Therefore, a watertight foundation is not required. The composite drainage system along with the proposed perimeter and underfloor drainage system will discharged any surface/rain water infiltrating along the foundation walls. No groundwater lowering is expected as a result of this system, as discussed in the previous responses.

Item B27:

Comment: *More detailed recommendations with respect to slope stability shall be made now that more detailed plans are available.*

Response: A review of the slope stability for the proposed grade raise and servicing was completed in the revised geotechnical report under subsection 6.8. The proposed development is situated in an area beyond the limit of hazard lands and therefore, no issues regarding slope instability is expected.

Iteam B28:

Comment: *Update the drawings within the Appendix to reflect the latest site plan.*

Response: The test hole location plan base plan has been updated with the latest site plan in the revised geotechnical report.

Item B29:

Comment: *The Geotechnical Investigation shall discuss the suitability of stormwater storage chambers being implemented as well as geotechnical requirements as they relate to the proposed chambers (i.e. frost protection, separation from bedrock/the high ground water table, slope stability, etc.).*

Response: Please refer to SubSection 6.10 of the revised geotechnical report. The proposed tanks are suitable for the proposed development.

We trust that the current submission meets your immediate requirements.

Paterson Group Inc.

Joey R. Villeneuve, M.A.Sc., P.Eng., ing.

Faisal I. Abou-Seido, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Sept 29, 2020
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

patersongroup **memorandum**
consulting engineers

re:

Grading Plan Review

Proposed Mixed-Use Development - Wellings of Stittsville Phase 2
20 Cedarow Court - Ottawa

to: Nautical Lands Group - **Ms. Angela Mariani** - angela@nlgc.com
date: September 29, 2020
file: PG4772-MEMO.02

Further to your request and authorization, Paterson Group (Paterson) prepared the current memorandum to provide a review of the grading plan for Phase 2 of the mixed-use development to be constructed at the aforementioned site. This memorandum should be read in conjunction with Paterson Report PG4772-1 dated March 7, 2019.

Grading Plan Review

Paterson reviewed the following grading plan prepared by Stantec regarding the aforementioned development:

❑ Grading Plan - Wellings of Stittsville Phase 2 - Project No. 160401511, Drawing No. GP-1, Sheet 4 of 7, Revision 1 dated May 22, 2020.

Based on our review of the above noted grading plan, the proposed grade raises within Phase 2 of the aforementioned development are acceptable from a geotechnical perspective. No exceedances of the grade raise restriction were noted, therefore lightweight fill will not be required for the proposed buildings within Phase 2 of the proposed development.

We trust that this information satisfies your immediate requirements.

Best Regards,

Paterson Group Inc.

Sept. 29, 2020
F. I. ABOU-SEIDO
100156744

Joey R. Villeneuve, M.A.Sc, P.Eng., ing.

Faisal I. Abou-Seido, P.Eng.

patersongroup
consulting engineers

memorandum

re: **Grading Plan Review**
Proposed Mixed-Use Development – Wellings of Stittsville Phase 2
20 Cedarow Court - Ottawa

to: Nautical Lands Group – **Mr. Mark Williams** – mwilliams@nauticallandsgroup.com

cc: Stantec – **Mr. Mike Sharp** – Mike.Sharp@stantec.com

date: August 12, 2021

file: PG4772-MEMO.03

Following your request and authorization, Paterson Group (Paterson) prepared the current memorandum to complete a grading plan review from a geotechnical perspective for Phase 2 of the mixed-use development to be constructed at the aforementioned site. The following memorandum should be read in conjunction with Paterson Group Report PG4772-1 Revision 1, dated September 29, 2020.

Grading Plan Review

Paterson reviewed the following grading plan prepared by Stantec regarding the aforementioned development:

☐ Grading Plan - Wellings of Stittsville Phase 2 - Project No. 160401511 - Drawing No. GP-1 - Sheet No. 4 of 7 - Revision 2 - dated August 3, 2021.

Based on our review of the above noted grading plan, the proposed grades within Phase 2 of the aforementioned development are within the permissible grade raise restriction of 2 m provided throughout the subject site in the aforementioned geotechnical investigation report. Therefore, the proposed grading is considered acceptable from a geotechnical perspective. No exceedances of the grade raise restriction were noted, therefore lightweight fill or other considerations to accommodate the proposed grades are not required at this time.

We trust that this information satisfies your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Aug. 12, 2021
F. I. ABOU-SEIDO
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patersongroup
consulting engineers

memorandum

re: **Geotechnical Response to MVCA Comment**
Proposed Mixed-Use Development - Wellings of Stittsville Phase 2
20 Cedarow Court - Ottawa
to: Nautical Lands Group - **Mr. Mark Williams** - mwilliams@nauticallandsgroup.com
date: February 24, 2022
file: PG4772-MEMO.04

As requested, Paterson Group (Paterson) prepared the current memorandum to provide a geotechnical response to the Mississppi Valley Conservation Authority (MVCA) engineering comment for the proposed mixed-use development to be located at the aforementioned site. This memorandum should be read in conjunction with Paterson Report PG4772-1 Revision 1 dated September 29, 2020.

Geotechnical Comment

Comment 2

Comment: *In addition, with the proposed infiltration trench in close proximity to the top of the slope, MVCA would like to see additional comments from the Geotechnical Engineer regarding the proposed location of the infiltration trench and any potential impacts on slope stability. Will an infiltration trench in proximity of the top of slope cause excessive saturation of the slope and lead to instability?*

Response: It is understood based on discussion with Stantec that the site requires the installation of an infiltration trench near the top of an existing slope. The proposed trench will be approximately 40 m long, 5.5 m wide, and 1.4 m deep. Further, Paterson has reviewed the following drawing prepared by Stantec:

❑ Site Servicing Plan - Wellings of Stittsville Phase 2 - 20 Cedarow Court - Project No. 160401511 - Drawing No. SSP-1 Sheet 3 of 7 - Revision 3 dated September 9, 2021.

Based on our review of the above-mentioned drawing, the proposed trench location does not fall into the established Limit of Hazard Lands as indicated in the geotechnical report. Therefore, the location of the trench does not present any issues from a geotechnical perspective. It is understood that the trench will be in-filled with clear stone, it is recommended that a non-woven geotextile be installed against the base and sidewalls of the trench prior to the placement of clear stone. Doing so will limit the potential for erosion of the side walls of the trench from the running water.

It should be noted that water is not expected to infiltrate the silty clay soils at the trench due to the cohesive nature of the soil. Surface water which comes into contact with the silty clay at the trench will sheet drain at the surface. It should be further noted that the slope stability analysis completed for the subject slope assumed full saturation of the soils behind the top of slope and within the slope face. Therefore, in the unlikelihood of the saturation of the soils behind the top of slope, no negative impacts on the slope stability is expected from a geotechnical perspective.

Based on our review, there are no slope stability issues anticipated from a geotechnical perspective with the excavation of the infiltration trench at the proposed location.

We trust that the current submission meets your immediate requirements.

Paterson Group Inc.

Patey

Nicole R.L. Patey, B.Eng.

LICENSED PROFESSIONAL ENGINEER
Feb. 24, 2022
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

PATERSON GROUP

memorandum

re: **Foundation Drainage System Design Recommendations**
Proposed Mixed-Use Development - Wellings of Stittsville - Phase 2 & 3
20 Cedarow Court - Ottawa
to: Nautical Lands Group - **Ms. Angela Mariani** – angel@nlgc.com
to: Chmiel Architects - **Ms. Elzbieta Chylinska** - ElaC@chmielarchitects.com
cc: Laurin General Contractors - **Mr. Matthew Smith** - matthew.smith@laurin.ca
date: April 18, 2023
file: PG4772-MEMO.05 Revision 3

Further to your request and authorization, Paterson Group (Paterson) prepared the current memorandum to provide geotechnical recommendations regarding the foundation drainage system design for the proposed structure. This memorandum should be read in conjunction with Paterson Group report PG4772-1 Revision 3 dated August 31, 2022.

As part of the review, Paterson has reviewed the following architectural drawings prepared by Chmiel Architects:

☐ Section Details – Project No. 19-1764 – Wellling of Stittsville Phase 2 – Drawing No. A-323, Revision 4, dated February, 2023 – Prepared by Chmiel Architects

☐ Section Details – Project No. 19-1764 – Wellling of Stittsville Phase 2 – Drawing No. A-500, Revision 1, dated February, 2023 – Prepared by Chmiel Architects

1.0 Foundation Drainage

Based on our review of the latest design details, it is understood that the proposed building will accommodate 1 underground parking level and will be extended to a sound bedrock surface. It is anticipated that intermittent sections of the north, east and south foundation walls will be placed in close proximity to the site boundaries. It is expected that insufficient room will be available for exterior backfill along these sections of the walls and, therefore, sections of the foundation walls will be poured directly against a foundation drainage system placed against the shoring face in a blind-side fashion. Refer to Figure 1 - Foundation Drainage System - Blind-Side Pour, for specific details of the foundation drainage recommendations, attached to the current memorandum. The majority of the foundation walls are anticipated to be poured using conventional double-sided formwork and that the foundation drainage system will be placed directly against the foundation wall one the formwork has been removed. Refer to Figure 2 - Foundation Drainage System - Double-Sided Pour.

Foundation Drainage System

A foundation drainage system is recommended for the underground parking level to prevent water from seeping through the concrete foundation walls. No adverse effects are expected for surrounding structures with respect to the groundwater lowering that would cause short-term and long-term damage to the adjacent structures surrounding the proposed building.

Furthermore, to manage and control groundwater infiltration to the building’s storm sump pump(s) over the long term, the following foundation drainage system is recommended to be installed on the building’s foundation walls using the following methodology:

- ☐ It is recommended that a composite foundation drainage membrane, such as 6000 series membrane by DeltaDrain, G100N by MiraDrain or equivalent approved other, be placed on the exterior perimeter foundation wall surfaces. The composite foundation drainage board should extend from finished grade to the underside of footing level with the geotextile layer facing the prepped substrate surface. It is highly recommended that the drainage board be installed horizontally, in a shingle-fashion, with a minimum overlapping of 150 mm between the sheets to minimize seams throughout the system.

- ☐ Blind-Sided Pour: Where the foundation walls are constructed using a blind-sided pour and the drainage board is fastened directly to the shoring system/bedrock, the horizontal overlaps should be completed such that the top of the lower sheet of drainage board is underlying the bottom of the upper sheet. Refer to Figure 1 - Foundation Drainage System - Blind-Side Pour.

- ☐ Double-Sided Pour: Where the foundation walls are constructed using a conventional double-sided pour and the drainage board is fastened directly to the foundation walls, the vertical overlaps should be completed such the top of the lower sheet of drainage board overlies the bottom of the upper sheet. Refer to Figure 2 - Foundation Drainage System - Double-Sided Pour.

- ☐ It is recommended that 150 mm diameter sleeves placed at 12 m centres be cast in the foundation wall at the footing interface to allow water to flow to an underfloor drainage system. The underfloor drainage system should direct water to the storm sump pit(s) within the lower basement area.

PVC Drainage Sleeves

As previously noted, the drainage sleeves installed as part of the foundation drainage system should be at least 150 mm diameter and be spaced 12 m center to center along the exterior perimeter foundation walls.

The installer should ensure that a “X” shaped incision is opened or partially cut out from the composite foundation drainage layer to allow the passage of water into the perimeter sleeves. The upper 1/3 of the sleeve should be cut at a 45-degree angle on the side to be inserted into the drainage board. This connection should be verified by Paterson personnel at the time of construction.

Prior to casting foundation walls surrounding the sleeves, it is recommended that an adhesive such as 3M tape or an approved equivalent be utilized to seal the 150 mm drainage sleeve to act as a barrier in preventing concrete from entering the drainage sleeve during the placement of concrete. Once the temporary form work has been removed, the adhesive tape can be cut away to permit drainage from the foundation drainage system. It should be noted that the BlueSkin is not recommended for use in freezing conditions, therefore, if winter construction is proposed, 3M Tape is highly recommended.

Exterior Perimeter and Underfloor Drainage System

The exterior perimeter and underfloor drainage system will be required to control water infiltration below the underground parking level slab and to redirect groundwater from the buildings foundation drainage system to the buildings sump pit(s). The exterior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated PVC pipe surrounded by a minimum of 150 mm of 19 mm clear crushed stone. It is recommended that the exterior perimeter drainage system be mechanically connected through the 150 mm drainage sleeves to the underfloor drainage system, which in turn is connected to the buildings storm sump pit(s). The mechanical connection locations to the underfloor drainage system is presented on Figure 6 – Perimeter and Underfloor Drainage plan attached to the end of this report.

Where construction of the foundation wall transitions from a conventional double sided pour to a blind sided pour, and where the exterior foundation wall abuts the property line, it is recommended that the exterior perimeter drainage pipe with the transition to an interior drainage pipe.

It should be noted that where the foundation walls are constructed using a blind-sided pour and the drainage board is fastened directly to the shoring system, the perimeter drainage pipe of the underfloor drainage system should be installed at the interior side of the foundation wall at the junction point between the foundation wall and the footing. In addition, where the foundation walls are constructed using a conventional double-sided pour and the drainage board is fastened directly to the foundation walls, the perimeter drainage pipe of the underfloor drainage system should be installed exterior side of the foundation wall on top of the footing. Refer to Figure 6 for the recommended location of the exterior perimeter drainage pipe with the transition to an interior drainage pipe. Reference should also be made to Figure – 3 – Foundation Drainage System Blind Side to Double Side transition.

Transition from Blind Side to Double Sided Formwork

Based on our review, the construction of the exterior perimeter foundation walls above the temporary shoring system is anticipated to transition from a blind-sided pour into a conventional double sided pour at the south side of the excavation. The integrity of the drainage system should be maintained across this transition zone as per the following methodology:

❒ It is critical that the composite foundation drainage board is extended in a suitable manner through this transition zone to maintain the long-term performance of the foundation drainage system. It is recommended that the composite foundation drainage board be extended a minimum of 0.6 m beyond the limits of the temporary shoring system where a transition from blind side to double sided formwork is anticipated. As a result, the contractor should be prepared to provide supplemental temporary formwork to ensure this transition is completed in a suitable manner.

❒ The purpose of additional custom formwork is to provide a suitable surface on which to maintain the vertical continuity and application of the foundation drainage system across the transition zone from blind-side to double-sided pours. Based on our experience, the additional formwork will be required in areas where the bedrock/overburden interface has an unsuitable quantity of voids, jagged surfaces and/or fractures from blasting and bedrock removal procedures. This custom formwork typically consists of suitably prepared plywood, rigid insulation, or other concrete formwork materials as procured by the formwork contractor which is cut and sized to match the contours of the bedrock/overburden interface. This additional effort and material by the formwork contractor will mitigate the risks associated with over-pouring the foundation beyond the overburden/bedrock interface and with pouring concrete onto the system bridging gaps and voids. Lastly, carrying out this measure will ensure that the foundation drainage system is installed in a relatively flat and vertical fashion.

❒ It is **NOT** recommended to fold the composite foundation drainage board to accommodate temporary formwork and the placement of concrete at the transition zone. It is therefore recommended that the contractor modifies temporary forms to match the contour of the surface which the formwork will be placed upon (i.e., temporary shoring system) and secure the drainage board against the inside of the temporary forms.

❒ Since the bulk of the custom formwork will most likely remain in place, the foundation drainage board must extend a minimum of 1 m above the horizontal and vertical extent of the custom formwork and onto the inside of the prefabricated formwork. Once the foundation wall has been poured and the modified temporary formwork has been removed, the exposed portion of the composite foundation drainage board can be overlapped in a shingle fashion in accordance with the manufacturer’s specifications and in general conformance with our geotechnical recommendations.

The above-noted recommendations are further illustrated in Figure 3 – Foundation Drainage System Transition from Blind Side to Double Sided Formwork at the South Excavation Face, attached to the current memorandum.

Further, all the above-noted recommendations should be verified and approved at the time of construction/installation/placement by Paterson personnel.

2.0 Elevator Pit Waterproofing System

To Accommodate the proposed elevator shafts, additional bedrock removal below the building’s perimeter strip footings will be required. it is expected that the elevator shaft may extend below the general bottom of excavation.

In addition, due to the elevations proposed for the bottom of the elevator shafts, water may seep through into the exterior sides of the elevator shafts. As a result, the following elevator shaft waterproofing system is recommended:

- Once the concrete slab and elevator pit sidewalls are poured in place, it is recommended that a waterproofing membrane, such as an elastomeric membrane (Elastochem), Colphene Torch’n Stick or approved other be applied to the exterior of the elevator pit sidewalls down to the bottom of the slab and horizontally over the elevator slab in accordance with the manufacturer’s specifications.

- If the bedrock encountered below the elevator shaft is observed to have excessive fractures and fissures, a waterproofing membrane will be required below the elevator slab. This should be confirmed by Paterson at the time of construction.

- A Xypex additive can be used in the concrete mix of the elevator shaft walls and slan to provide a secondary waterproofing measure. This is optional and **cannot** be used as a standalone waterproofing for the elevator pits.

- As a secondary defense, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed along the interface between the concrete slab and elevator pit sidewalls.

- A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. Should over excavation of the bedrock face occur during construction of the elevator pit, the area between the elevator pit and bedrock excavation face should be in-filled with lean concrete, OPSS Granular B Type II or OPSS Granular A crushed stone.

- A drainage pipe should extend from the nearest wall to the elevator sump pit to ensure that any water infiltration into the interior walls of the elevator shaft. The pipe should have a positive drainage towards the elevator sump pit. Alternatively, A PVC pipe can be installed vertically into the elevator slab and through the slab into the sump pit.

- Reference should be made to Figure 4 - Waterproofing System for Elevator, for specific details of the elevator waterproofing attached to the current memorandum.

It should be noted that the current detail of the elevator pit waterproofing system proposed by Chmiel Architects on above noted architectural drawings has been reviewed by Paterson and it is considered acceptable from a geotechnical perspective.

3.0 Podium Deck Tie-In for Waterproofing System

It is expected that a waterproofing system will be provided for the podium deck surface. It is recommended that the podium deck waterproofing system consist of a layer of hot rubber membrane applied to the concrete surface. The concrete should be cleaned of any dust, dirt, or debris prior to the application of the hot rubber. The hot rubber should be overlain by a 50 mm thick layer of HI-60 rigid insulation, or equivalent, and further overlain by a foundation drainage board (6000 series by DeltaDrain, G100N MiraDrain, or approved equivalent) installed with the geotextile side facing up.

It should be noted that HI-60 rigid insulation will be used in heavy load traffic areas, parking areas , access lane, and landscaped areas where the backfill is compacted. However, it is acceptable that HI-40 rigid insulation will be installed in areas where the top backfill is not lightly compacted.

The hot rubber should be applied to the geotextile side of the drainage board to cover the cold joint a minimum of 150 mm. A termination bar should be installed as per manufacture’s specifications. The podium deck drainage board can then be overlapped to cover the cold joint a minimum of 150 mm. Reference should be made to Figure 5 - Podium Waterproofing Tie In and Backfill Detail for Hardscape Areas.

Where podium deck finished areas are to be hardscaped, backfilling can be completed as per the above-mentioned figure. Where parking areas, access lanes and other paved areas are proposed to be overlying the podium the pavement structure provided in Table 1 below should be used.

Table 1 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - Superpave 12.5 Asphaltic Concrete
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - To consist of concrete podium deck.	

It should be noted that the current detail of the podium deck tie-in for the waterproofing system proposed by Chmiel Architects on above noted architectural drawings has been reviewed by Paterson and it is considered acceptable from a geotechnical perspective.

4.0 Proposed Tunnel Tie-In for Waterproofing System

It is understood that a precast concrete tunnel is proposed to connect the underground parking levels at Phase 1 and Phase 2 of the development. It is recommended that the tunnel's upper deck be waterproofed in a similar fashion as the podium deck with a layer of hot rubber membrane applied to the concrete surface followed by rigid insulation and drainage boards. In addition, the drainage/waterproofing membranes should be wrapped around the side wills in a similar fashion as the podium deck. The side walls should be covered with composite drainage boards installed as per the above sections for double-sided pours.

An exterior perimeter drainage system will be required to control water infiltration the exterior sides of the tunnel and to redirect groundwater from the tunnels drainage system to the existing Phase 1 and Phase 2/3 perimeter drainage systems. The exterior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated PVC pipe surrounded by a minimum of 150 mm of 19 mm clear crushed stone. It is recommended that the exterior perimeter drainage system be gravity connected to the existing perimeter drainage system at Phase 1 and Phase 2/3 of the development, which in turn is connected to the buildings storm sump pit(s).

Where finished areas above the tunnel are to be hardscaped and/or paved, backfilling can be completed as per the recommendations in Section 3.0 above.

Based on our review, It should be noted that the current detail of the tunnel tie-in for the waterproofing system proposed by Chmiel Architects on above noted architectural drawings is considered acceptable from a geotechnical perspective.

5.0 Additional Considerations and Field Inspections

Should a layer of rigid insulation be specified by others to be placed on the exterior portion of the buildings foundation wall, it is recommended to install this layer over the foundation drainage layer, which would be further underlain by the concrete foundation wall.

To ensure an adequate incorporation of these recommendations in the buildings design, it is recommended that Paterson review up-to-date architectural, structural, and mechanical drawings that may incorporate these recommendations. Paterson should also review all specification sheets of equivalent materials to be used other than what's recommended herein.

It is also recommended that Paterson personnel carry out routine inspections of the implementation of the following items at the time of construction:

☐ Bedrock removal, if required, and building footprint excavation in consideration of the future application of the foundation drainage system

☐ Installation of the foundation drainage system, including the composite foundation drainage board and drainage sleeves;

☐ Installation of the perimeter and underfloor drainage system.

☐ Waterproofing of all elevator shafts.

☐ Application of the foundation drainage system over the transition from blind-sided to double-sided pours.

☐ Application of the podium deck waterproofing and tie-in.

☐ Installation of foundation wall insulation, if required by the project's architect.

We trust that the current submission meets your requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
April 18, 2023
F. I. ABOU-SEIDO
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PROVINCE OF ONTARIO

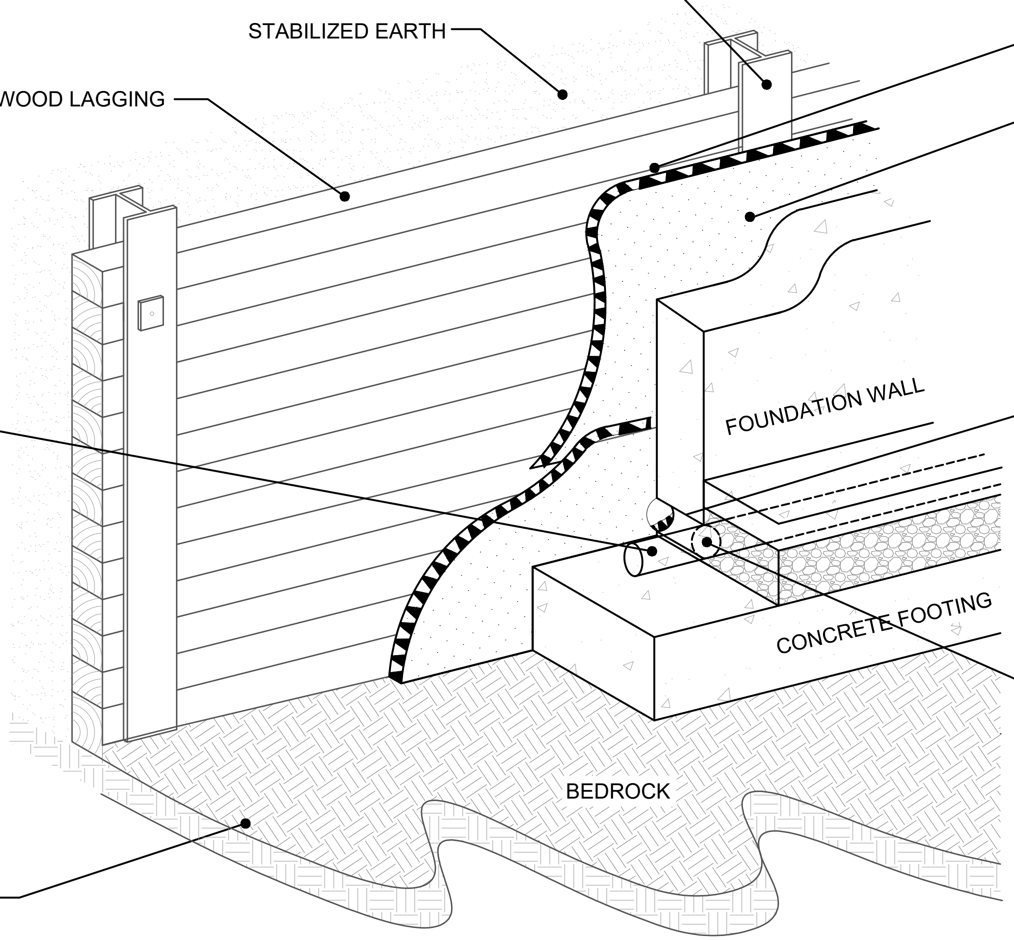
Nicole, R.L. Patey, B.Eng

Faisal I. Abou-Seido, P.Eng.

Attachments:

☐ Figure 1 - Foundation Drainage System - Blind-Side Pour
☐ Figure 2 - Foundation Drainage System - Double-Sided Pour
☐ Figure 3 - Foundation Drainage System Transition from Blind Side to Double Sided
☐ Figure 4 - Waterproofing System for Elevator
☐ Figure 5 - Podium Waterproofing Tie-in and Backfill Detail for Hardscape Areas
☐ Figure 6 - Perimeter and Underfloor Drainage Plan

IT IS RECOMMENDED THAT THE DRAINAGE BOARD BE INSTALLED ON THE INSIDE OF THE TEMPORARY FRAMEWORK IN THE DOUBLE SIDED CONCRETE WALL POUR (WHERE APPLICABLE) TO ENSURE THE DRAINAGE BOARD MAINTAINS A FLAT AND VERTICAL INSTALL.



GEOTEXTILE LINED SIDE OF THE DRAINAGE BOARD FACING THE VERTICAL BEDROCK SURFACE

COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH 3M TAPE OR APPROVED EQUIVALENT.

150mm DIAMETER INTERIOR PERIMETER DRAINAGE PIPE WITH POSITIVE OUTLET TO SUMP PIT. THE DRAINAGE PIPE IS RECOMMENDED TO BE CORRUGATED AND PERFORATED AND PLACED ON THE EXTERIOR SIDE OF THE FOUNDATION WALLS. WRAPPED WITH FILTER CLOTH AND SURROUNDED ON ALL SIDES BY 300 mm OF CLEAR CRUSHED STONE

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE.

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER EXTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEOTECHNICAL CONSULTANT

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE SYSTEM.

PATERSON GROUP

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TEL: (613) 226-7381

NAUTICAL LANDS GROUP c/o WELLINGS OF STITTSVILLE INC.

PROPOSED MIXED-USE DEVELOPMENT

20 CEDAROW COURT

OTTAWA, ONTARIO

Title:

FOUNDATION DRAINAGE SYSTEM - BLIND SIDE POUR

Scale: N.T.S.

Date: 02/2023

Drawn by: YA

Report No.: PG4772-MEMO.05

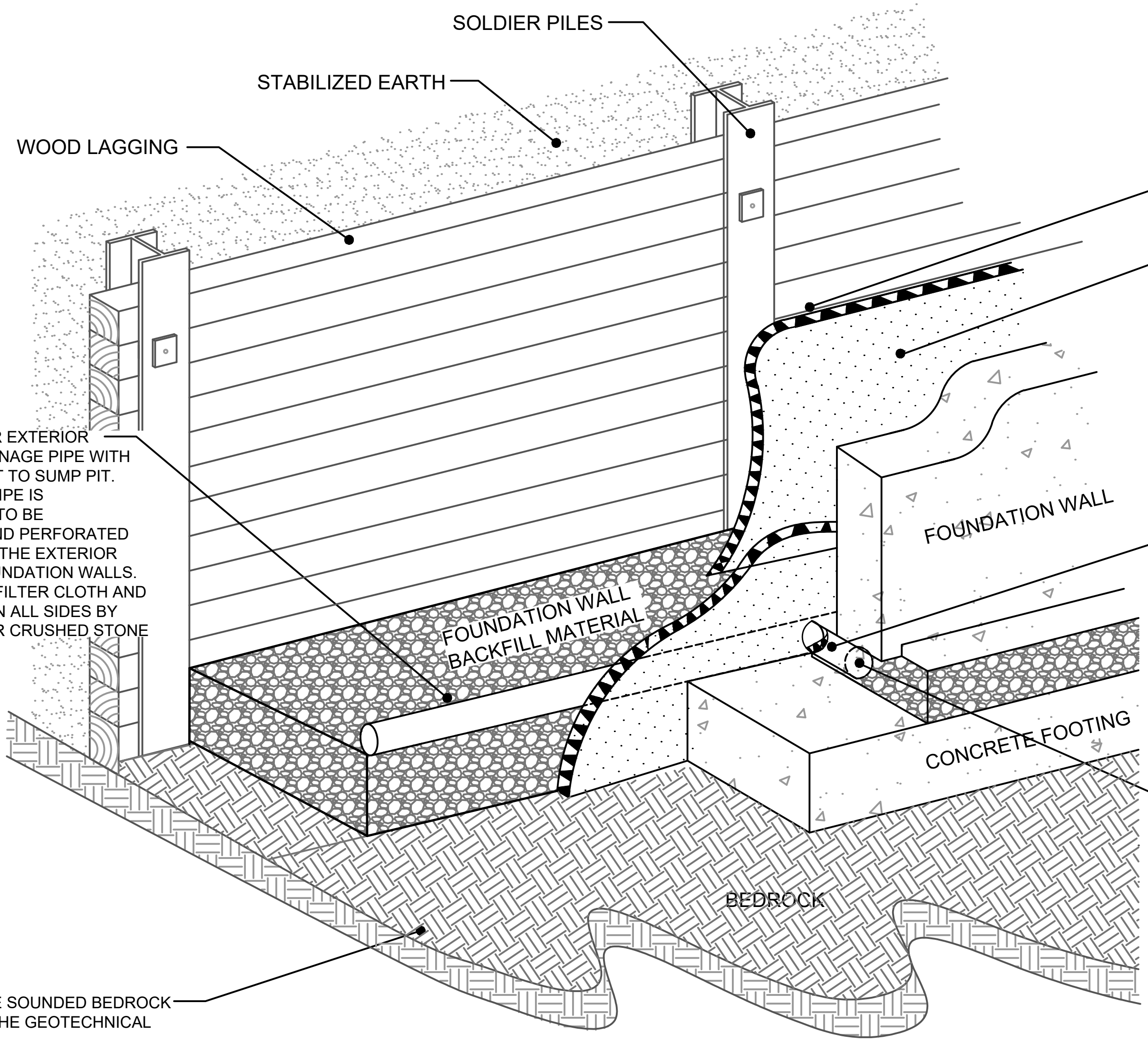
Checked by: NP

Drawing No.: FIGURE 1

Approved by: FA

Revision No.: 2

p:\autocad drawings\geotechnical\pg47xx\pg4772\pg4772 figure 1- foundation drainage system.dwg



IT IS RECOMMENDED THAT THE DRAINAGE BOARD BE INSTALLED ON THE INSIDE OF THE TEMPORARY FRAMEWORK IN THE DOUBLE SIDED CONCRETE WALL POUR (WHERE APPLICABLE) TO ENSURE THE DRAINAGE BOARD MAINTAINS A FLAT AND VERTICAL INSTALL.

GEOTEXTILE LINED SIDE OF THE DRAINAGE BOARD FACING THE VERTICAL BEDROCK SURFACE

COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH 3M TAPE OR APPROVED EQUIVALENT.

150mm DIAMETER EXTERIOR PERIMETER DRAINAGE PIPE WITH POSITIVE OUTLET TO SUMP PIT. THE DRAINAGE PIPE IS RECOMMENDED TO BE CORRUGATED AND PERFORATED AND PLACED ON THE EXTERIOR SIDE OF THE FOUNDATION WALLS. WRAPPED WITH FILTER CLOTH AND SURROUNDED ON ALL SIDES BY 300 mm OF CLEAR CRUSHED STONE

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE.

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER EXTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEOTECHNICAL CONSULTANT

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE SYSTEM.

PATERSON GROUP

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NAUTICAL LANDS GROUP c/o WELLINGS OF STITTSVILLE INC.

PROPOSED MIXED-USE DEVELOPMENT

20 CEDAROW COURT

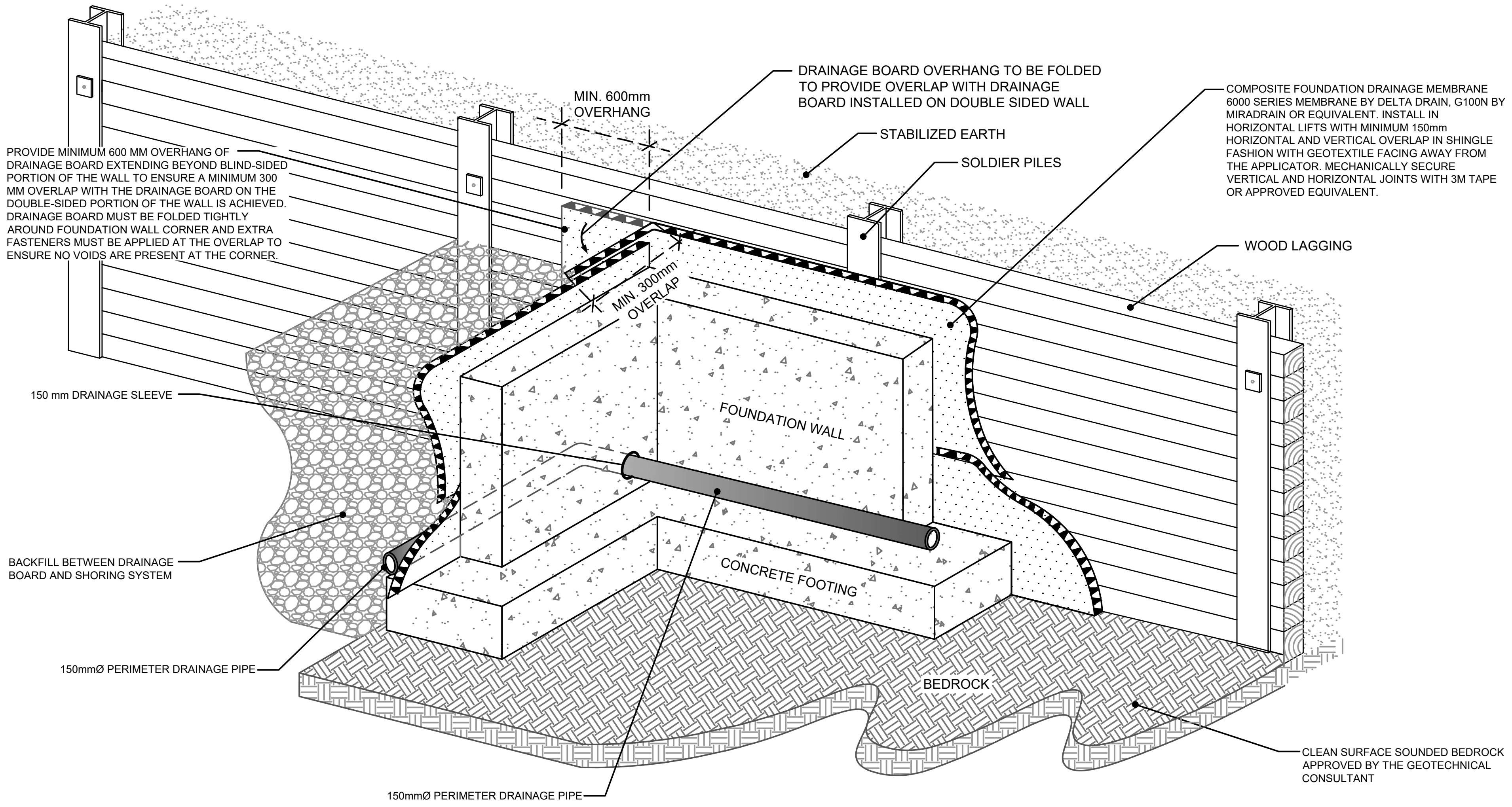
OTTAWA, ONTARIO

Title:

FOUNDATION DRAINAGE SYSTEM - DOUBLE SIDE POUR

Scale: N.T.S.	Date: 02/2023
Drawn by: YA	Report No.: PG4772-MEMO.05
Checked by: NP	Drawing No.: FIGURE 2
Approved by: FA	Revision No.: 2

p:\autocad drawings\geotechnical\pg47xx\pg4772\pg4772 figure 1- foundation drainage system.dwg



PATERSON GROUP

NAUTICAL LANDS GROUP c/o WELLINGS OF STITTSVILLE INC.

PROPOSED MIXED-USE DEVELOPMENT
20 CEDAROW COURT

OTTAWA, ONTARIO

Title:

FOUNDATION DRAINAGE SYSTEM BLIND SIDE TO DOUBLE BLIND SIDE TRANSITION

Scale: N.T.S.	Date: 03/2023
Drawn by: NFRV	Report No.: PG4772
Checked by: NP	Drawing No.: FIGURE 3
Approved by: DJG	Revision No.:

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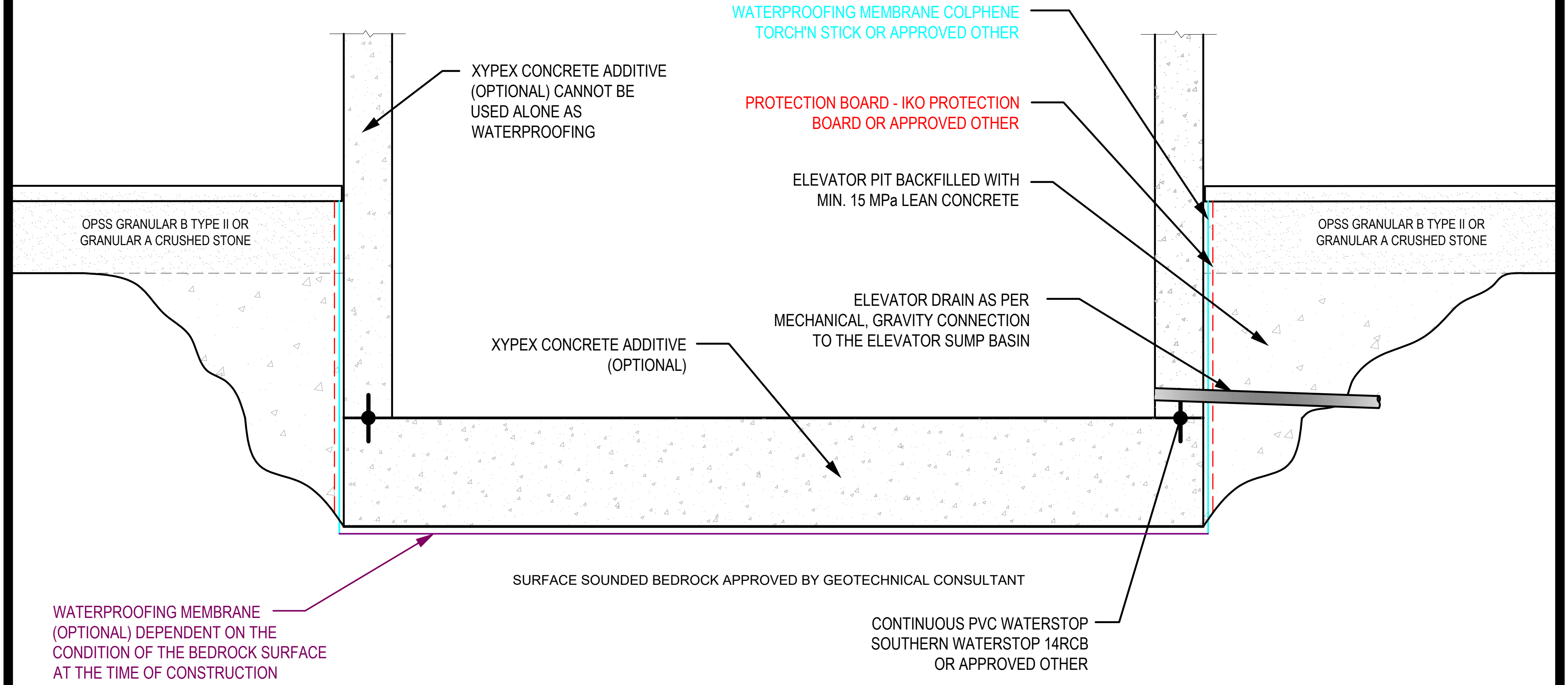

NOTES:

1. IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE ELEVATOR WATERPROOFING MEMBRANE(S).

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NAUTICAL LANDS GROUP c/o WELLINGS OF STITTSVILLE INC.
PROPOSED MIXED-USE DEVELOPMENT
20 CEDAROW COURT
OTTAWA, ONTARIO

Title:

WATERPROOFING SYSTEM FOR ELEVATOR

Scale: N.T.S.
Date: 02/2023
Drawn by: YA
Report No.: PG4772-MEMO.05
Checked by: NP
Drawing No.: FIGURE 4
Approved by: FA
Revision No.: 2

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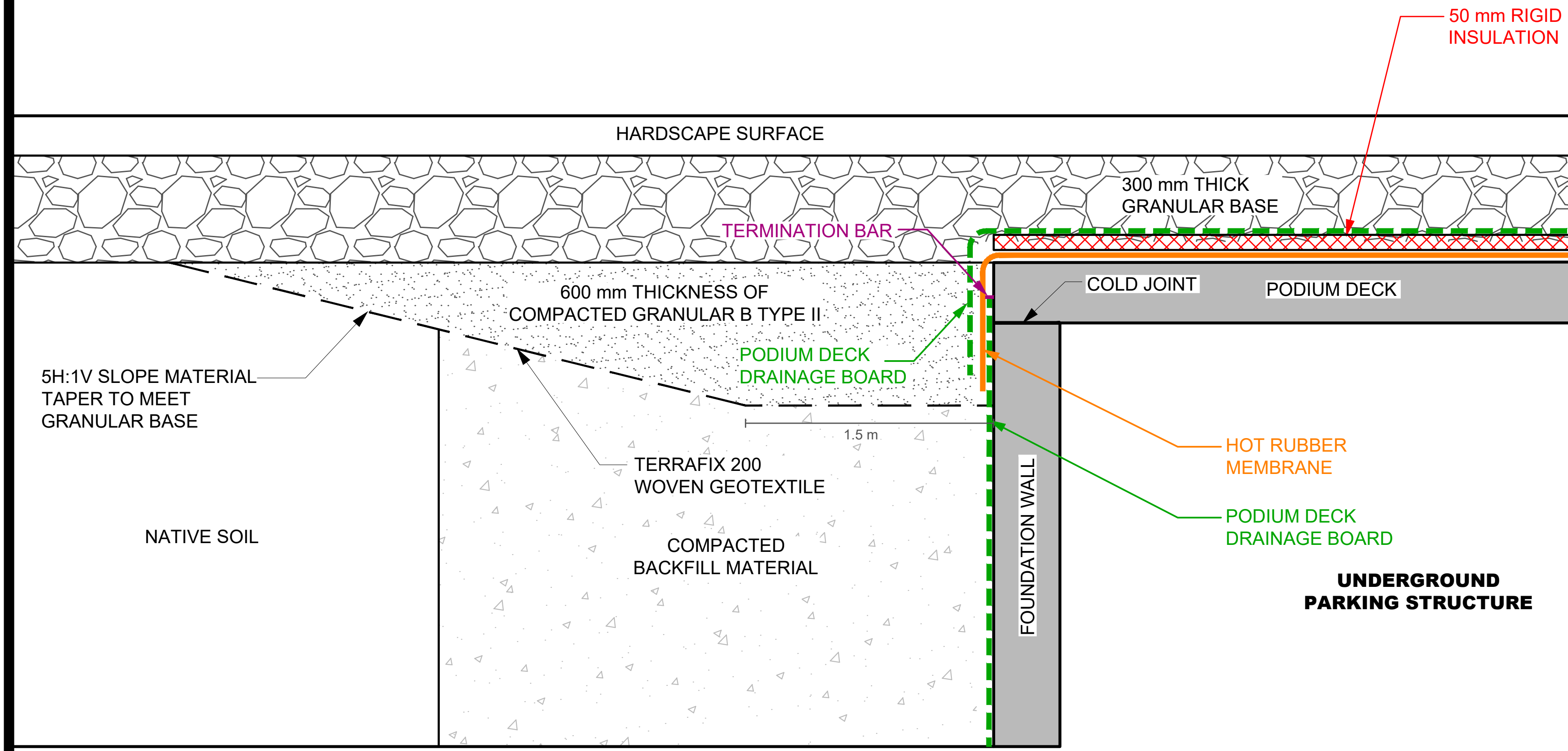

NOTES:

THE ABOVE DETAIL FOR WATER PROOFING MEMBRANE, HOT RUBBER, AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL LOCATIONS WHERE EXCAVATION IS SHORED. BACKFILLING RECOMMENDATIONS PERTAIN ONLY TO HARDSCAPED AREAS.

PATERSON GROUP

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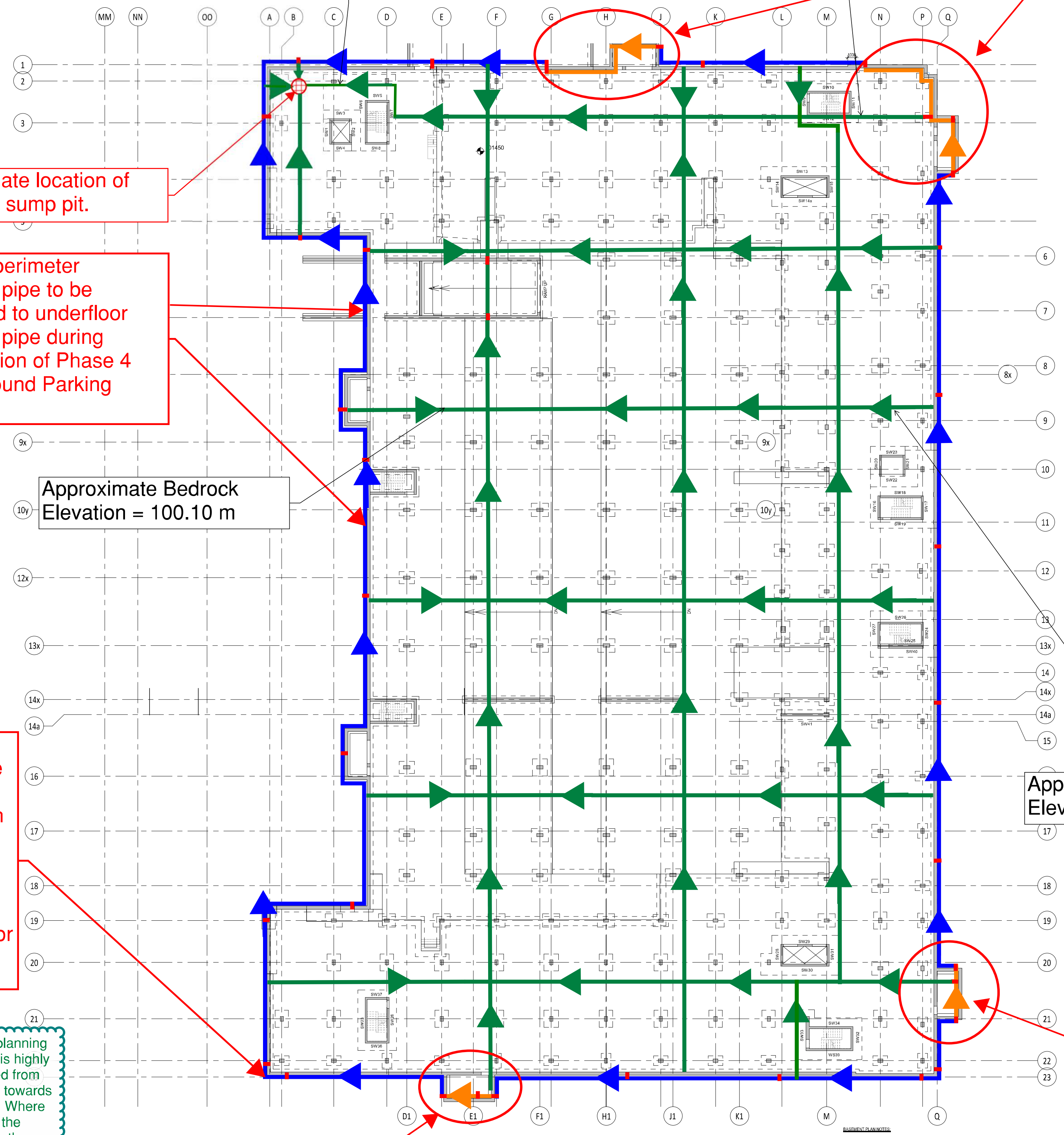
NAUTICAL LANDS GROUP c/o WELLINGS OF STITTSVILLE INC.
PROPOSED MIXED-USE DEVELOPMENT
20 CEDAROW COURT
OTTAWA, ONTARIO

Title:
PODIUM WATERPROOFING TIE IN AND BACKFILL DETAIL FOR HARDSCAPE AREAS

Scale: N.T.S.	Date: 02/2023
Drawn by: YA	Report No.: PG4772-MEMO.05
Checked by: NP	Drawing No.: FIGURE 5
Approved by: FA	Revision No.: 2

p:\autocad drawings\geotechnical\pg47xx\pg4772\pg4772- figure 4- podium waterproofing.dwg

Figure 6 - Perimeter and Underfloor Drainage Plan - By Paterson Group
Paterson Group Memo PG4772-MEMO.05 Rev 3



Approximate Bedrock
Elevation = 99.82 m

Approximate Bedrock
Elevation = 100.37 m

Transition from exterior to interior perimeter drainage pipe.

Approximate location of proposed sump pit.

Exterior perimeter drainage pipe to be converted to underfloor drainage pipe during construction of Phase 4 Underground Parking Level.

Approximate Bedrock
Elevation = 100.10 m

Exterior perimeter drainage pipe at corners of the Phase 2/3 & future Phase 4 Interface to be outfitted with "T" connection.

The unused connection of the "T" should be capped until connection is needed for Phase 4.

Approximate Bedrock
Elevation = 100.37 m

Transition from exterior to interior perimeter drainage pipe.

Transition from exterior to interior perimeter drainage pipe.

150 mm diameter exterior perimeter drainage pipe with positive outlet to sump pit. The drainage pipe is recommended to be corrugated and perforated and placed on the exterior side of the foundation walls. Wrapped with filter cloth and surrounded on all side by 300 mm of clear crushed stone.

Minimum 150 mm diameter PVC drainage sleeves cast into the foundation wall at approximately 12 m spacing and cast into interior walls, as required. Note: Recommended spacing to be confirmed by the Paterson personnel at the time of construction.

150 mm diameter perforated and corrugated interior underfloor drainage pipes with positive outlet to sump pit. Drainage pipes should be spaced at 12 to 15 m spacing.

150 mm diameter perforated and corrugated interior perimeter drainage pipes with positive outlet to sump pit.

Areas conflicting with proposed plumbing layout can be modified upon request

IMPORTANT NOTE - If the contractor is planning on placing the pipe directly on bedrock, it is highly recommended that the bedrock be cleaned from outcrops and be sloped at a minimum 1% towards the sump pit to promote gravity drainage. Where the bedrock dips below the surroundings, the contractor should build up the subgrade to the desired invert level using free draining, non-frost susceptible granular fill such as OPSS Granular A or Granular B Type II placed in max 200 mm thick lifts and compacted to 98% of the material's SPMDD. An As-built is highly recommended of the bedrock surface and be provided to Paterson for review and approval before placement of the underfloor drainage pipes.

BASEMENT PLAN NOTES:

1- REFER TO ARCHITECTURAL DRAWINGS FOR FINISHED FLOOR ELEVATIONS, SLOPES AND DIMENSIONS

2 LOADING
SUPERIMPOSED DEAD LOAD:
PARKING AREAS : 0.5 kPa
SUSPENDED MECH 0.5 kPa
LOCKER ROOMS 0.5kPa
MECH/ELEC ROOMS 6.0kPa
SNOW MELTING SYSTEM 2.3kPa (50mm TOPPING)
LIVE LOAD:
PARKING AREAS: 2.4kPa
CORRIDORS/STAIRS/LOBBIES: 4.8kPa
LOCKER ROOMS: 4.8 kPa
BIKE STORAGE ROOM: 4.8 kPa
MECH/ELEC ROOMS 6.0 kPa

3. PROVIDE CORROSION INHIBITOR FOR ALL EXPOSED REINFORCED CONCRETE AREA

4. SLAB ON GRADE: 150 SLAB
Fc = 32MPa
CLASS C2
R/W : #6 8X8 WWR
5. REFER TO S6.01 FOR FOUNDATION WALL ELEVATIONS

6. ALL BEAMS TO REMAIN SHORED UNTIL CONCRETE HAS REACHED 28DAYS STRENGTH

7. REFER TO TYPICAL DETAILS FOR BUILT-UP SLABS DETAIL AND REINFORCING

8. CONCRETE FOR EXPOSED TERRACES AND EXTERIOR COLUMNS TO BE CLASS F2 EXPOSURE

9. REFER TO GENERAL NOTES, TYPICAL DETAIL FOR TYPICAL SHEAR WALLS AND COLUMNS REINFORCING DETAIL

10. SEE S3.00 - S3.03 FOR COLUMN SCHEDULE AND S5.01 FOR GRAVITY BEAM SCHEDULE

11. SEE S4.01 - S4.04 FOR SHEAR WALL ELEVATION AND S4.0 FOR SHEAR WALL ZONE REINFORCING

12. MAINTAIN SLAB-ON-GRADE THICKNESS AS SHOWN AT ALL LOCATIONS UNLESS NOTED AS RECESS OF PLAN.

13. SEE GENERAL NOTES FOR CONTROL JOINT REQUIREMENTS U.N.O. ON PLAN: C.J. CONTROL JOINTS MAY BE REPLACED WITH CONSTRUCTION JOINTS.

FOUNDATION PLAN NOTES:

1. FOUNDATION DESIGN IS BASED ONT HE GEOTECHNICAL REPORT #PG4772-1 DATED MARCH 7, 2019 PREPARED BY PATERSON GROUP. REPORT TO THIS REPORT FOR SOILS CONDITIONS AND FOUNDATION CONSTRUCTION RECOMMENDATIONS. CONTRACTOR SHALL READ AND UNDERSTAND THIS REPORT BEFORE STARTING WITH THE FOUNDATION WORK

2 ALL FOOTING TO REST ON THE SOUND ROCK. DESIGN IS BASED ON AVERAGE ROCK ELEVATION PER GOEOTECHNICAL INVESTIGATION REPORT. CONTRACTOR TO CARRY ALLOWANCE FOR DIFFERENCES THAT MAY ARISE FROM DIFFERENCE IN ACTUAL ROCK ELEVATION

3. DESIGN ROCK BEARING CAPACITY: ULS: 1500 kPa
SLS: N/A

4. FOR FOOTINGS WHERE BOTTOM ELEVATION IS ABOVE THE BEDROCK ELEVATION, PROVIDE 25MPa LEAN MIX BELOW AT A 1:1 SLOPE (SEE GEOTECHNICAL REPORT FOR DETAIL)

5. FOUNDATION ELEVATIONS AND BEARING STRATA TO BE VERIFIED BY GEOTECHNICAL ENGINEER ON SITE

6. COLUMNS AND WALL TO BE CENTERED ON FOOTINGS UNLESS OTHERWISE NOTED ON PLAN

NOTE: THIS DRAWING IS THE PROPERTY OF THE STRUCTURAL ENGINEER AND MAY NOT BE REPRODUCED OR USED WITHOUT THE EXPRESSED CONSENT OF THE ASTRUCTURAL ENGINEER. THE CONTRACTOR SHALL CHECK AND VERIFY ALL DIMENSIONS AND REPORT ANY OMISSIONS OR DISCREPANCIES TO THE STRUCTURAL ENGINEER BEFORE PROCEEDING WITH THE WORK.

DO NOT SCALE THE DRAWINGS

RELEASE / REVISION RECORD

No.	Description	Date
1	Issued for Coordination	2021-10-22
2	Issued for Building Permit	2021-12-06
3	Issued for Excavation/Rock breaking & Services Tender	2022-01-28
4	Issued for Revision to Building Permit	2022-03-07
5	Issued for Concrete Formwork, Rebar and Structural Steel Tender	2022-03-21
6	Issued for Below Grade Excavation	2022-06-24
7	Issued for Coordination	2022-10-26
8	Phase 2 – Issued for Re-submission for Building Permit	2022-12-02
9	Issued for 93% Coordination	2023-01-13

PROJECT NORTH

MAY NOT BE USED FOR CONSTRUCTION UNTIL SIGNED BY THE ENGINEER

McINTOSH PERRY
115 Walgreen Road, RR3, Carp, ON K0A 1L0
Tel: 613-836-2184 Fax: 613-836-3742
www.mcintoshperry.com

WELLINGS OF STITTSVILLE
PHASE 2

20 CEDAROW COURT, STITTSVILLE, ON

PROJECT NO. 0CP-19-0047-00
DRAWN TE SF dmw
SCALE As indicated
CHECKED AS
DRAWING TITLE P1 PARKING LEVEL - OVERALL
DRAWING NO. S1.00

PATERSON
GROUP

memorandum

re: **Geotechnical Response to City Comments**
Proposed Mixed-Us Development – Wellings of Stittsville Phase 2
20 Cedarow Court - Ottawa

to: Nautical Lands Group - **Ms. Angela Mariani** - angela@nlgc.com

cc: Stantec - **Mr. Mike Sharp** – Mike.Sharp@stantec.com

date: June 22, 2022

file: PG4772-MEMO.08

Following your request and authorization, Paterson Group (Paterson) prepared the current memorandum to provide responses to technical comments from the City of Ottawa regarding potential buoyancy effects on the proposed underground storage tanks to be located at the aforementioned site. The following memorandum should be read in conjunction with Paterson Group Report PG4772-1 Revision 2 dated February 17, 2022 and memorandum PG4772-MEMO.07 dated June 22, 2022.

Technical Comments – Third Submission (June 6, 2022)

Comment 14: *New 3rd Submission Comment: It is noted that subdrains have been proposed around the perimeter of the underground storage tanks to address the concern for buoyancy. The City would like to see additional comment from the Geotechnical Engineer related to buoyancy of the tanks, as well as confirmation that the tank design is generally acceptable from a geotechnical perspective.*

Response: Paterson has provided a response addressing the concerns and potential impacts of buoyancy effects on the underground storage tanks in the aforementioned memorandum, PG4772-MEMO.07 dated June 22, 2022.

We trust that this information satisfies your immediate requirements.

Best Regards,

Paterson Group Inc.

O.Canton

Owen Canton, EIT

LICENSED PROFESSIONAL ENGINEER
June 22-2022
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

PATERSON GROUP

memorandum

re:

Geotechnical Recommendations – Shoring Pad

Proposed Mixed-Use Development - Wellings of Stittsville - Phase 2 & 3
20 Cedarow Court - Ottawa

to: Nautical Lands Group - **Mr. Charles Beaulne** – cb@nlgc.com
cc: Laurin General Contractors - **Mr. Matthew Smith** - matthew.smith@laurin.ca
date: December 20, 2022
file: PG4772-MEMO.09 Revision 1

Further to your request and authorization, Paterson Group (Paterson) prepared the current memorandum to provide geotechnical recommendations for the proposed shoring pad to be constructed prior to the installation of the shoring system. This memorandum should be read in conjunction with Paterson Group report PG4772 Revision 3 dated August 31, 2022.

1.0 Background Information

It is understood that the proposed shoring system will be installed along the northern portion of the site using a mobile Bauer BH28 drill rig. The proposed loading for the subject drill rig along with the location of the drill rig mobility were provided in the following documents:

- ☐ Quick Overview – BG 28 H #5519 – V01_en_12.2021
- ☐ Wellings of Stittsville – Drawing No. SH00 to SH04 – Shoring Plans dated September 1, 2022, prepared by Iron Shoring and Dymont Engineering.

Based on correspondence with Iron Shoring and as indicated on the attached technical data sheets, the maximum ground pressure (P_{MAX}) associated with the current configurations of the drill rig will be 420 kPa. It is understood the maximum casing size of 5 m will add 30 kPa. Therefore a maximum ground pressure of 450 kPa is required for the proposed configuration.

Subsurface Profile

Based on our review of the nearest boreholes within the northern area of the subject site, the subsurface profile consists of a very stiff to hard silty clay, followed by a dense glacial till and/or surface sounded bedrock surface. Groundwater table is expected to be encountered between 2.5 to 3.5 m below existing grade.

2.0 Geotechnical Assessment and Recommendations

Based on our review of the provided information, it is expected that the proposed pad will be placed over a very stiff to hard, brown silty clay bearing medium. Paterson geotechnical report provided a bearing medium of **150 kPa (SLS)** for the subject bearing medium which is not enough for the shoring pad. It should be noted that due to the undrained shear strength values acquired within this portion of the subject site by means of vane readings, a bearing capacity of **200 kPa SLS** can be applicable for the silty clay layer.

Based on the technical data sheets and correspondence with Iron Shoring, a minimum 450 kPa (SLS) will be required for the drill rig to operate. Therefore, to increase the bearing capacity of the ground, it is recommended that a granular pad be constructed to evenly distribute the loading without impacting the native silty clay subgrade as well as increase the bearing capacity of the subsurface soils.

To achieve the desired ground bearing medium, it is recommended that ground be excavated to a minimum 0.5 m below existing grade down to a native very stiff to hard silty clay bearing medium. The ground should be reviewed and approved by Paterson at the time of construction. Once exposed, a biaxial geogrid liner such as Terrafix TBX2500 or equivalent, should be placed over the silty clay followed by a woven geotextile liner such as Terrafix 200W or equivalent. The 500 mm thick granular pad should consist of OPSS Granular A or Granular B Type II placed in maximum 300 mm thick loose lifts and compacted to a minimum 98% of the material's SPMDD.

It is important to note that the granular pad should extend a minimum 1.5 m beyond the edge of the work area to avoid disturbing the soils surrounding the pad area. Also, the geotextile and geogrid liners should extend along the side slopes of the pad's excavated area.

It is expected the drill rig will operating for a short period of time (approximately 1 week+/) Due to the minimal thickness of the engineered pad, rutting is expected to be encountered which may require topping the granular pad and recompacting after completing the work **on a daily basis** until the shoring installation is completed. Therefore, it is highly recommended that Paterson completes a daily inspection of the shoring pad until the completion of the shoring installation to ensure that the performance of the granular pad is in accordance with the current recommendations and expectations.

Provided that the above recommendations are met, the granular pad can resist temporarily a pressure of 450 kPa (SLS) as required by the technical data sheets.

We trust that the current submission meets your requirements.

Best Regards,

Paterson Group Inc.

Faisal I. Abou-Seido, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Dec.20-2022
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

re: **Grading and Site Servicing Plan Review**
Proposed Mixed-Use Development - Wellings of Stittsville - Phase 2 & 3
20 Cedarow Court - Ottawa

to: Nautical Lands Group - **Ms. Angela Mariani** – angel@nlgc.com
to: Chmiel Architects - **Ms. Elzbieta Chylinska** - ElaC@chmielarchitects.com
cc: Laurin General Contractors - **Mr. Matthew Smith** - matthew.smith@laurin.ca
date: May 29, 2023
file: PG4772-MEMO.16

Further to your request and authorization, Paterson Group (Paterson) prepared the current memorandum to complete a review for the grading and servicing plans of the proposed mixed-use development, and to provide associated recommendations from a geotechnical perspective for the aforementioned project. The following memorandum should be read in conjunction with the current Geotechnical Investigation Report PG4772-1 Revision 5 dated May 29, 2023.

Grading Plan Review

Paterson reviewed the following grading plans prepared by Stantec for the aforementioned development:

❐ Grading Plan – Nautical Lands Group, 2962 Carp Road, Wellings of Stittsville PH 2, 20 Cedarow Court – Project No. No. 160401511 – Sheet No. 4 - Revision 8.

Generally, the subsurface profile along the proposed right of ways consists of topsoil underlain by a very stiff to stiff silty clay deposit followed by compact glacial till. The glacial till layer generally consisted of compact silty sand to sandy silt with clay, cobbles and boulders. Practical refusal was encountered on inferred bedrock underlying the glacial till layer.

Based on our review, the proposed grading plans are considered acceptable from a geotechnical perspective and doe not exceed the recommended permissible grade raise provided in the aforementioned geotechnical report. Therefore, no other considerations to accommodate the proposed grades are required at this time.

Site Servicing Plan Review

Paterson reviewed the following site servicing plans prepared by Stantec for the aforementioned development:

- ☐ Site Servicing Plan – Nautical Lands Group, 2962 Carp Road, Wellings of Stittsville PH 2, 20 Cedarow Court – Project No. 160401511 – SSP-1 – Sheet No. 3 - Revision 8.
- ☐ Site Servicing Plan – Nautical Lands Group, 2962 Carp Road, Wellings of Stittsville PH 2, 20 Cedarow Court – Project No. 160401511 – EX-1 – Sheet No. 2 - Revision 8.
- ☐ Erosion Control Plan and Detail Sheet – Nautical Lands Group, 2962 Carp Road, Wellings of Stittsville PH 2, 20 Cedarow Court – Project No. 160401511 –EC/DS-1-1 – Sheet No. 5 - Revision 8.
- ☐ Storm Drainage Plan – Nautical Lands Group, 2962 Carp Road, Wellings of Stittsville PH 2, 20 Cedarow Court – Project No. 160401511 – SD-1 – Sheet No. 6 - Revision 8.
- ☐ Sanitary Drainage Plan – Nautical Lands Group, 2962 Carp Road, Wellings of Stittsville PH 2, 20 Cedarow Court – Project No. 160401511 – SA-1 – Sheet No. 7 –Revision 8.

From a geotechnical perspective, the relevant recommendations including adequate frost protection of services, pipe bedding and backfill recommendations provided by Paterson in the aforementioned geotechnical investigation report have been incorporated satisfactorily into the above plans.

Clay Seal Recommendations

Since clay will be encountered within the trench sidewalls of the site servicing trenches completed along the property boundaries of the subject site, clay seals are recommended to be provided within the service trenches where clay is present within the trench sidewalls. Paterson personnel should confirm the presence of clay within the excavation sidewalls at the subject locations prior to the placement of the clay seals to confirm their applicability at the time of construction.

The clay seals should be a minimum of 1.5 m long (in the trench direction) and should extend from trench wall to trench wall spaced at a maximum 60 m spacing center to centre. The seals should extend from the subgrade for the overlying pavement structure and fully penetrate the bedding, subbedding and cover material. The clay seals 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 placement of clay seals at the subject site should be reviewed and approved by Paterson personnel at the time of construction.

Landscape Plan/Tree Planting Plan Review

Based on our review of the drawing Landscape Plan – Wellings of Stittsville PH 2, 20 Cedarow Court – Project No. 1164 – L1.01, provided by Levstek Consultants, the proposed plans are considered acceptable from a geotechnical perspective and are in general conformance with the aforementioned Geotechnical Report.

We trust that this information satisfies your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 29, 2023
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Puneet Bandi, M.Eng.

Faisal I. Abou-Seido, P.Eng

PATERSON GROUP

memorandum

re: **Geotechnical Response to City Comments**
Proposed Mixed-Use Development - Wellings of Stittsville - Phase 2 & 3
20 Cedarow Court - Ottawa
to: Nautical Lands Group - **Ms. Angela Mariani** – angel@nlgc.com
to: Chmiel Architects - **Ms. Elzbieta Chylinska** - ElaC@chmielarchitects.com
cc: Laurin General Contractors - **Mr. Matthew Smith** - matthew.smith@laurin.ca
date: May 29, 2023
file: PG4772-MEMO.17

Further to your request, Paterson Group (Paterson) has prepared the current memo to provide our responses to the geotechnical-related comments from the City of Ottawa for the proposed development to be located at the aforementioned site. This memo should be read in conjunction with the updated Geotechnical Investigation PG4772-1 Revision 5 dated April 29, 2023.

Geotechnical Investigation Report

Comment B22: *Please provide a stamped and signed memo from the retained Geotechnical Engineer identifying that the most recent site design drawings (i.e. grading plan, servicing plan, tree planting/landscape plan) have been reviewed from a geotechnical perspective and are in keeping with the geotechnical recommendations provided in the Geotechnical Investigation.*

Response: Reference should be made to Paterson Group Memorandum PG4772-MEMO.16 dated May 26, 2023.

Comment B23: *Geotechnical report to make recommendations related to suitability of storm chambers and infiltration BMP system proposed beneath storm chambers (e.g., infiltration rate, separation from groundwater, separation from bedrock, etc.). Section 6.10 of the geotechnical report still references formerly proposed setup of concrete storage tanks.*

Response: Please refer to Section 6.10 of the revised Geotechnical Investigation Report, referenced above. In summary the chambers are suitable to function as a stormwater detention system for the proposed development. It is recommended that the proposed system be founded with a minimum of 1 m of separation between the base of the system and the static, long-term groundwater table be maintained in order to comply with the LID guidelines. The infiltration rates are as assumed to be 10-30 mm per hour, any LID's being considered should incorporate within the above-mentioned range of infiltration.

Due to the recommended separation between the groundwater table and the base of the system, uplift forces will be negligible.

Comment B24: *Geotechnical report to make necessary recommendations related to tunnel installation between phase 1 and 2.*

Response: Reference should be made to Paterson Group Memorandum PG4772-MEMO.05 Revision 1 dated July 7, 2022.

Comment B25: *Please compile all geotechnical related memos and report into one document.*

Response: Reference should be made to our revised geotechnical Report PG4772-1 Revision 5 dated April 29, 2023

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

Faisal I. Abou-Seido, P.Eng.

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May 29., 2023
F. I. ABOU-SEIDO
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