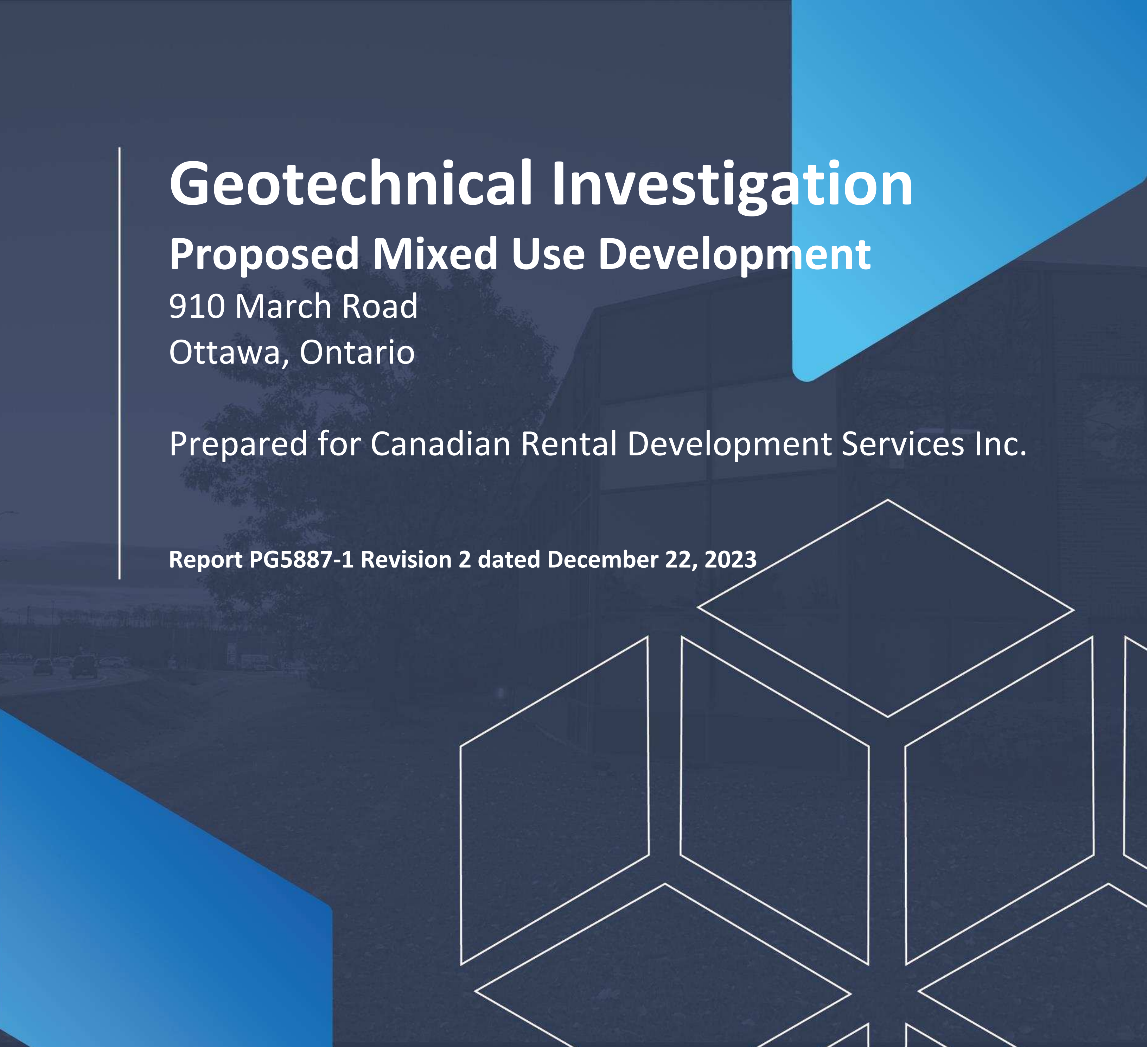
Geotechnical Investigation

Proposed Mixed Use Development

910 March Road
Ottawa, Ontario

Prepared for Canadian Rental Development Services Inc.

Report PG5887-1 Revision 2 dated December 22, 2023



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1.0 Introduction

Paterson Group (Paterson) was commissioned by Canadian Rental Development Services Inc. to conduct a geotechnical investigation for the proposed mixed use development to be located at 910 March Road in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

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

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of two (2) multi-storey buildings sharing two (2) underground parking levels. Associated access lanes, at-grade parking and hardscaped areas, and walkways are also anticipated as part of the proposed development. It is further anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out between July 14 and 19, 2021 and consisted of advancing a total of 16 boreholes to a maximum depth of 7.0 m below existing ground surface. A previous geotechnical investigation was also completed on subject site on October 15, 2019. At that time, a total of 9 boreholes were advanced to a maximum depth of 4.7m or auger refusal below existing ground surface. The test hole locations were determined by Paterson personnel and distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG5887-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a low clearance 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 drilling to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Rock samples were recovered from all boreholes drilled during the current investigation (BH 1-21 through BH 16-21) 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 Quality 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.

Subsurface conditions observed in the test pits were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test pits locations.

Groundwater

Boreholes BH 1-21, BH 6-21 and BH 11-21 were fitted with 51 mm diameter PVC groundwater monitoring wells. Typical monitoring well construction details are described below:

- ❒ 3.0 m of slotted 51 mm diameter PVC screen at the base of the boreholes.
- ❒ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ❒ No. 3 silica sand backfill within annular space around screen.
- ❒ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ❒ Clean backfill from top of bentonite plug to the ground surface.

The other boreholes were fitted with flexible piezometers to allow groundwater level monitoring. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision handheld GPS and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5887-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is located on the east side of March Road, north of the intersection of Maxwell Bridge Road and March Road in the City of Ottawa, Ontario. The site is bordered to the north by vacant agricultural land, to the east by Shirley's Brook followed by newly constructed single family residential dwellings and to the south by a tributary (ditch) to Shirley's Brook followed by a commercial property. The north and north-east boundaries of the site are occupied by a shallow tributary to Shirley's Brook which bisects the northeast corner of the site before traveling south along the east property boundary. The site is generally flat with a slight downward slope toward the north, east and south property boundaries toward Shirley's Brook and its tributaries. The west portion of the site was observed to be approximately at grade with March Road.

The site was previously occupied by a single family residential dwelling having a basement level and an attached garage. The single dwelling was recently demolished. Several outbuildings and sheds of either slab-on-grade or wood pier construction occupy the central portion of the site. Several storage containers, trailers, sheet metal and farm equipment occupy the remainder of the site.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of topsoil and fill overlying a hard to stiff brown silty clay deposit, followed by compact to very dense glacial till, underlain by bedrock. The fill material was found to generally consist of brown silty sand with gravel, crushed stone, and trace clay. The fill layer thickness ranged approximately between 0.5 to 2.4 m. The glacial till layer was encountered underlying the silty clay deposit or fill at the location of most boreholes except for BH 1-21, BH 10-21, BH 11-21, BH 12-21, and BH 16-21 where glacial till was not encountered. The glacial till deposit was generally observed to consist of compact to dense brown silty clay with sand, gravel and cobbles. The bedrock was cored in the locations of all the boreholes at depths between 2.3 to 4.0 m below existing ground surface, with an average RQD value ranging from 40 to 100%. This is indicative of a poor to excellent quality bedrock within the footprint of the proposed building. 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.

Bedrock

Based on available geological mapping, the bedrock in this area consists of interbedded quartz sandstone and sandy dolostone of the March Formation with an overburden drift thickness of 2 to 10 m depth.

4.3 Groundwater

The groundwater levels were recorded within the monitoring wells and piezometers installed within the boreholes during the current and previous investigations on July 16, 2021 and September 18, 2019, respectively. The recorded groundwater levels are presented in Table 1 below and are further noted on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Measured Groundwater Levels – Current Investigation

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	78.36	2.93	75.43	July 26, 2021
BH 2-21	78.71	3.29	75.42	
BH 3-21	78.16	2.90	75.26	
BH 4-21	78.24	3.09	75.15	
BH 5-21	77.88	2.79	75.09	
BH 6-21	77.22	2.15	75.07	
BH 7-21	77.17	2.02	75.15	
BH 8-21	76.63	1.59	75.04	
BH 9-21	76.19	2.07	74.12	
BH 10-21	75.45	1.62	73.83	
BH 11-21	75.62	1.32	74.3	
BH 12-21	75.91	1.09	74.82	
BH 13-21	76.84	1.85	74.99	
BH 14-21	76.72	2.42	74.3	
BH 15-21	77.21	1.80	75.41	
BH 16-21	77.47	2.25	75.22	
Measured Groundwater Levels – Previous Investigation – PG5119				
Test Hole Number	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Dated Recorded**
		Depth (m)	**Elevation (m)**	
BH 1	78.39	Blocked at 0.83	-	September 18, 2019
BH 2	77.63	3.25	74.38	
BH 3	77.04	2.98	74.06	
BH 4	76.82	Blocked at 1.80	-	
BH 5	77.27	1.57	75.70	
BH 6	77.83	2.22	75.61	
BH 7	78.80	2.05	76.75	
BH 8	77.99	Block at 0.78	-	
BH 9	77.55	Destroyed	-	

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

It is important to note that groundwater readings can be influenced by surface water perched within the borehole backfill material. Long-term groundwater conditions can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, it is estimated that long-term groundwater level can be expected between geodetic elevations 74.0 m to 75.5 m. However, groundwater levels are subject to seasonal fluctuations and therefore could vary during time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed mixed-use development. Due to the presence of two levels of underground parking, it is anticipated that conventional shallow foundations placed on the clean surface sounded bedrock bearing surface will be suitable to support the proposed buildings.

It is anticipated that bedrock removal will be required for the majority of the excavation to complete the underground parking levels of the proposed structure. Line drilling and controlled blasting could be used where large quantities of bedrock are needed to be removed. The blasting operations should be planned and conducted under the guidance of a professional engineer with experience in blasting operations.

Based on the founding level of the proposed building with two underground levels, it is anticipated that the silty clay material will be removed entirely from below the footprint of the proposed building, Therefore, footings of the proposed building founded on bedrock will not be subjected to a permissible grade raise restriction. However, other settlement sensitive structures liked paved areas and site servicing alignments within the site will be subjected to a grade raise restriction of **2.0m**. If higher than the permissible grade raise is required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the anticipated founding level for the proposed building, all overburden material will be excavated from within the proposed building footprint. Bedrock removal will be required for portions of the underground parking levels.

Topsoil and deleterious fill, such as those containing significant organic materials, or construction debris/remnants should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only a small quantity of 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 the proximity of the blasting operations should be carried out prior to commencing site.

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

Excavation side slopes in sound bedrock can be carried out using near vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing. The 1 m horizontal ledge set back can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipments. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations 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). It should be noted that these guidelines are for today’s construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines.

Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Bedrock Excavation Face Reinforcement

A bedrock stabilization system consisting of a combination of horizontal rock anchors and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs. This system is usually considered where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations.

Fill Placement

Fill placed for grading beneath the building 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. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. This material should be used structurally only to build up the subgrade for pavements. Where the fill is open-graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction.

5.3 Foundation Design

Conventional Shallow Foundation

Footings placed on clean, surface sounded bedrock can be designed using a factored bearing resistance value at ULS of **2,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

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.

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Lateral Support

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

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

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit at the subject site, a **permissible grade raise restriction of 2.0 m** is recommended for settlement sensitive structures founded within the clay deposit, including paved areas (e.g., access lanes and heavy traffic access areas, and walkway areas) and pipe bedding areas. However, a permissible grade raise restriction will not be required within the proposed two muti-storey building footprint due to the anticipated founding depth, since all the overburden soil, including the silty clay deposit will be excavated entirely to accommodate for the underground parking structure.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements. Provided sufficient time is available to induce the required settlements, consideration could be given to surcharging the subject site.

5.4 Design for Earthquakes

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

Field Program

The seismic array testing location was placed within the central area of the site in an approximate east-west direction as presented in Drawing PG5887-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 18 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

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

The shot locations are also completed in forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 15, 3, and 2 m away from the first geophone 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, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

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

Based on our testing results, the average overburden shear wave velocity is 304 m/s, while the bedrock shear wave velocity is 2,438 m/s. Further, the testing results indicate the average overburden thickness to be approximately 4.0 m, which is considered to be a conservative estimate based on the results of our test hole information. Provided that the building will be founded on the bedrock surface, the overburden shear wave velocity does not need to be considered for the calculation of V_{S30}. The V_{S30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012, and as presented below.

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

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{2{,}438\ m/s}\right)}$$

$$V_{s30=}\ 2{,}438\ \ m/s$$

Based on the results of the seismic shear wave velocity testing, V_{s30}, for foundations at the subject site is 2,438 m/s. Therefore, a Site **Class A** is applicable for design of the proposed development 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

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native soils and bedrock surface will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

The upper 200 mm of sub-slab fill should consist of an OPSS Granular A material for slab-on-grade construction. All backfill material within the proposed building footprint should be placed in maximum 300 mm lifts and compacted to a minimum of 98% of the SPMDD.

Any soft subgrade areas should be removed and backfilled with appropriate backfill material, such as OPSS Granular B Type II, with a maximum particle size of 50 mm.

If the floor slab is constructed in the areas of shallow bedrock, it is recommended that a minimum 300 mm thick layer (native soil plus crushed stone layer) be present between the floor slab and the bedrock surface to reduce the risks of bending stresses developing in the concrete slab. The bending stress could lead to cracking of the concrete slab. This requirement could be waived in areas where the bedrock surface is relatively flat within the footprint of the building. This recommendation does not refer to potential concrete shrinkage cracking which should be controlled in the usual manner.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. However, if a full drainage system is being implemented and approved by Paterson at the time of construction, hydrostatic pressure can be omitted in the structural design.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the site area is 0.30 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using

$$P_o = 0.5\ K_o\ \gamma\ H^2\text{, where } K_o = 0.5 \text{ for the soil conditions noted above.}$$

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

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

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

5.7 Pavement Design

Car only parking areas, access and heavy traffic access areas are expected at this site. The subgrade material will consist of native soil and possibly bedrock. The proposed pavement structures are presented in Tables 2 and 3.

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

Table 2 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 3 – Recommended 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	**Wear Course –** HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or concrete fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) could be reduced to 300 mm. The upper 300 mm of the bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised to ensure that the bedrock subgrade does not have depressions that will trap water.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Based on the preliminary information provided, it is expected that a portion of the proposed building foundation walls will be located below the long-term groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for the proposed building. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater, which breaches the primary ground infiltration control system.

The groundwater infiltration control system should extend at least 1 m above the long-term groundwater level and the following is suggested for preliminary design purposes:

- Place a suitable waterproofing membrane against the temporary shoring surface, such as a bentomat liner system or equivalent. The membrane liner should extend down to footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at underside of footing level.
- Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- Pour foundation wall against the composite drainage system.

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 6-9 m centres be cast in the footing or at the foundation wall/footing interface 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.

It is important to note that the building's sump pit and elevator pit be considered for waterproofing in a similar fashion. A detail can be provided by Paterson once the design drawings are available for the elevator and sump pits.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration for the lower basement area. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at every bay opening. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided to the elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

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

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

The footings located along parking garage entrance may require protection against frost action depending on 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

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Side Slopes

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

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open 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 depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system or steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

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

Table 4 – Soils Parameter for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight 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. At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. 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 or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay and silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement. Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

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/building) 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. Based on the anticipated 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.

Due to the proposed waterproofing to be installed along the perimeter of the proposed building, no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

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

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

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

6.7 Corrosion Potential and Sulphate

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

6.8 Slope Stability Analysis

Slope Conditions

Based on our field observations and available topographic mapping, the subject slopes in the vicinity of the watercourse and in the eastern portion of the site are stable with no signs of active erosion and are sloped at 8H:1V slope or less. Boreholes in close proximity to the existing slopes were analyzed to determine the subsurface soil conditions for our analysis.

Slope Stability Analysis

The analysis of the stability of the slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

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

Two (2) slope cross-sections (Sections A and B) were studied as the worst case scenarios. The cross section locations are presented on Drawing PG5887-1 - Test Hole Location Plan in Appendix 2. It should be noted that details of the slope height and slope angle at the cross-section locations are presented in Figures 4 through 7 in Appendix 2 from the topographic data identified on Drawing PG5887-3 – Limit of Hazard Lands Plan in Appendix 2.

Stable Slope Allowance

The results of the slope stability analysis for static conditions at Section A and B are presented in Figure 4 and 6 in Appendix 2. Based on the analysis, the factor of safety for the slopes was greater than 1.5 for the slope section analysed.

The results of the analyses with seismic loading are shown in Figures 5 and 7 presented in Appendix 2. The results indicate that the factor of safety for the sections are greater than 1.1. Based on these results, the slopes are considered to be stable under seismic loading.

As the slopes were determined to be stable under static and seismic conditions for the sections analysed, a stable slope allowance is not required for the subject site.

Toe Erosion and Erosion Access Allowances

The slopes were generally observed to be vegetated with trees and brush. Further, flow from the creek in the watercourse at the base of the slopes was observed to be minimal, with no signs of active erosion observed at the toe of the slopes. In consideration of these observations, a toe erosion allowance is not considered to be required for these slopes. Therefore, a 6 m erosion access allowance is not required for the subject slopes adjacent to the watercourse alignment within the subject site.

Limit of Hazard Lands

The limit of hazard lands setback lines for the proposed development are presented on Drawing PG5887-3 - Limit of Hazard Lands in Appendix 2. The limit of hazard lands line is running parallel (no setback) to the identified top of slope and along the top of slope.

It is important to note that the existing vegetation on the slope faces should not be removed as it contributes to the long-term stability of the slope and reduces surficial erosion. If the existing vegetation needs to be removed along the slope faces, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed or an erosional control blanket be placed across the exposed slope face.

It should also be noted that Paterson reviewed the preliminary constraints plan Figure 4, Job No. 121186, dated March 2023, prepared by Novatech for the proposed development at the subject site. Based on our review, it is understood that the footprint of the proposed development slightly conflicts with the latest estimated MVCA meander belt line. It is further understood that the erosion hazard limit adopted by Novatech complies with the calculated meander belt limit provided by the Fluvial Geomorphic Assessment prepared by Parish based on site-specific study, which specifies a meander belt of 34m for Tributary 3 (Reach SPT-6) and 38m for Tributary 2 (Reach SBT-7B). Based on the above discussion, adopting the site-specific meander belt calculated from the approved fluvial geomorphic assessment referenced above is considered conservative, from a geotechnical perspective. Furthermore, the adopted erosion hazard limit reflected in Novatech’s study based on the calculated meander belt is considered acceptable from a geotechnical perspective.

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 of the geotechnical aspects of the excavation contractor's shoring design, prior to construction.
- Review the bedrock stabilization and excavation requirements.
- Observation of all bearing surfaces prior to the placement of concrete.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Sampling and testing of the concrete and fill materials used.
- 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.

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

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Canadian Rental Development Services Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Zubaida Al-Moselly, P.Eng.

Maha K. Saleh, M.A.Sc., P.Eng.

LICENSED PROFESSIONAL ENGINEER
December 22, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

Report Distribution:

- ☐ Canadian Rental Development Services Inc. (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

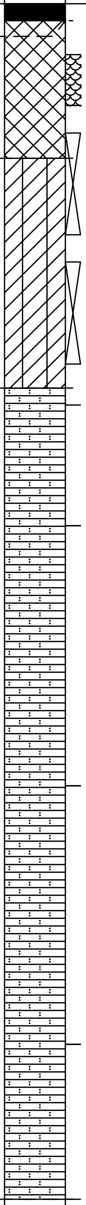
SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE July 14, 2021
FILE NO. PG5887
HOLE NO. BH 1-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.10
FILL: Brown silty sand with gravel, trace clay	0.91
Hard, brown **SILTY CLAY**	2.26
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with grey limestone	7.03
End of Borehole (GWL @ 2.93m - July 26, 2021)	

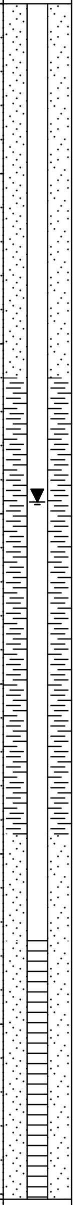


SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	22
SS	3	58	11
RC	1	100	86
RC	2	100	87
RC	3	100	100
RC	4	100	94

DEPTH (m)	ELEV. (m)
0	78.36
1	77.36
2	76.36
3	75.36
4	74.36
5	73.36
6	72.36
7	71.36

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

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



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SOIL PROFILE AND TEST DATA

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Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

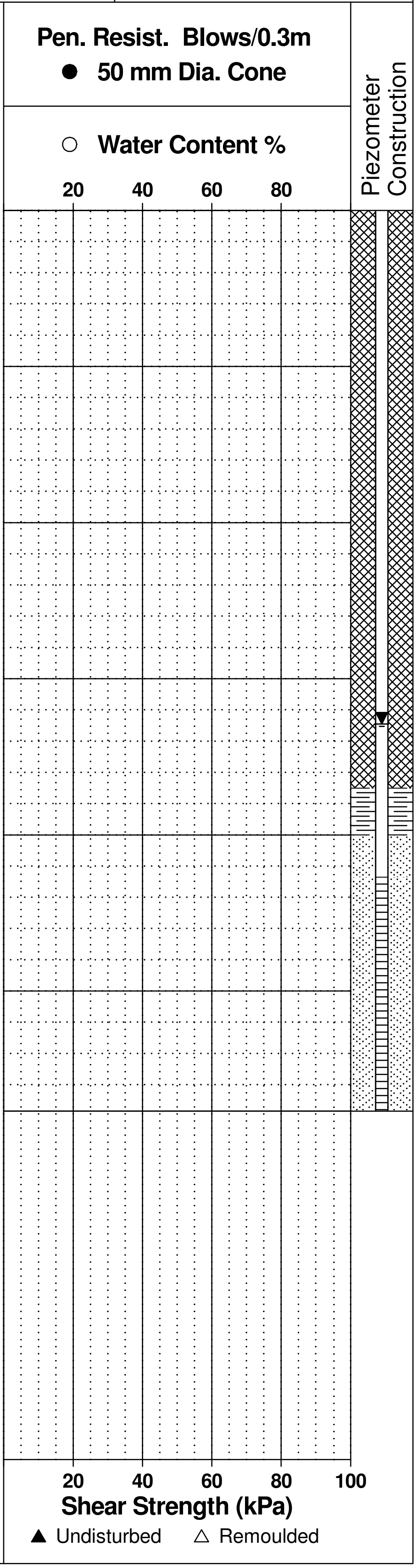
DATE July 14, 2021

FILE NO. PG5887

HOLE NO. BH 2-21

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Brown silty clay with gravel, trace topsoil	0.76	AU	1		
Hard, brown **SILTY CLAY**		SS	2	75	17
	2.44	SS	3	83	16
GLACIAL TILL: Brown silty clay with sand and gravel		SS	4	83	6
	3.50	SS	5	44	50+
BEDROCK: Excellent quality, grey silty dolostone interbedded with grey limestone		RC	1	100	90
	5.77	RC	2	100	100
End of Borehole					
(GWL @ 3.29m - July 26, 2021)					

DEPTH (m)	ELEV. (m)
0	78.71
1	77.71
2	76.71
3	75.71
4	74.71
5	73.71



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Proposed Mixed-Use Development - 910 March Rd.
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REMARKS

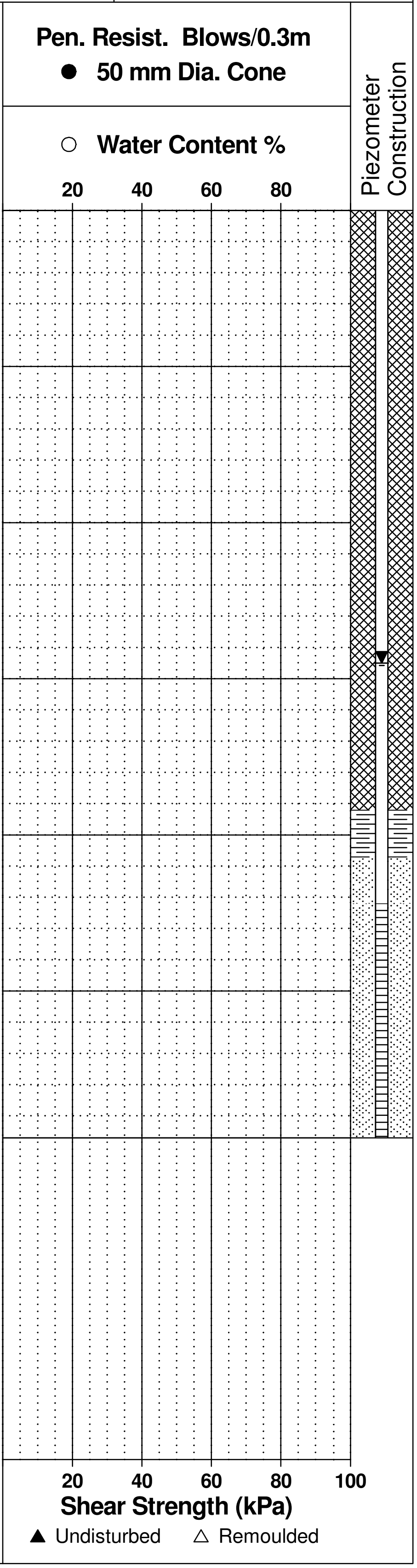
BORINGS BY CME-55 Low Clearance Drill

DATE July 14, 2021

FILE NO. PG5887

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	78.16
FILL: 25mm Organics over brown silty sand with gravel 0.91		AU	1				
Hard to very stiff, brown **SILTY CLAY** 1.68		SS	2	50	23	1	77.16
GLACIAL TILL: Brown silty clay with sand and gravel 2.30		SS	3	83	12	2	76.16
GLACIAL TILL: Very dense, brown silty sand with gravel, trace clay		SS	4	83	50+		
3.12		SS	5	100	50+	3	75.16
BEDROCK: Excellent quality, grey silty dolostone interbedded with grey limestone		RC	1	100	96	4	74.16
5.94		RC	2	100	100	5	73.16
End of Borehole (GWL @ 2.90m - July 26, 2021)							



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SOIL PROFILE AND TEST DATA

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Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

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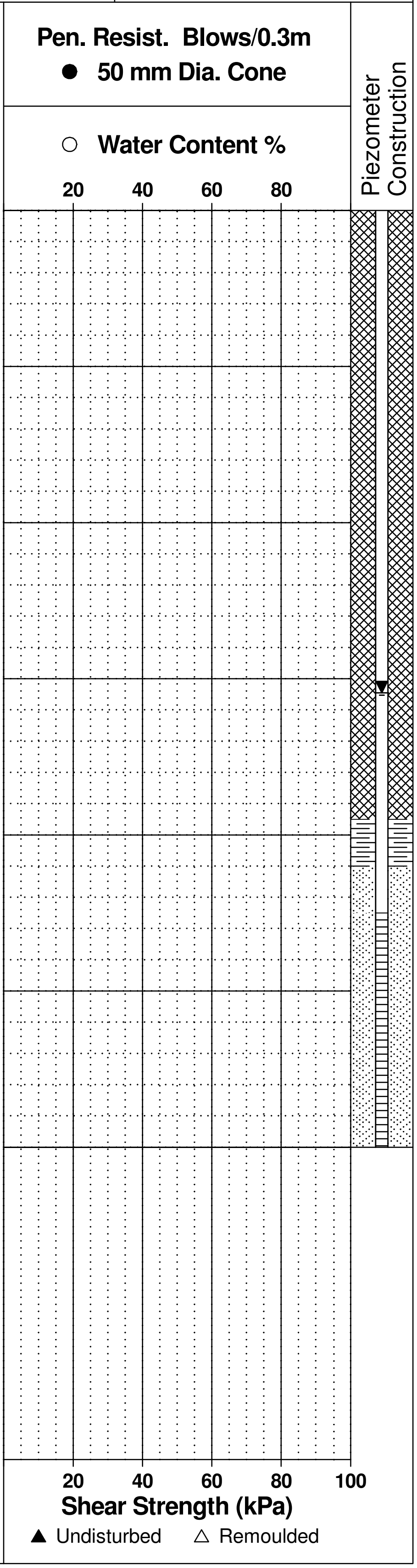
DATE July 14, 2021

FILE NO. PG5887

HOLE NO. BH 4-21

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25				
FILL: Brown silty sand with gravel and occasional boulder	0.76	AU	1		
Hard to very stiff, brown **SILTY CLAY**	1.83	SS SS	2 3	33 50	13 20
GLACIAL TILL: Brown silty clay, some sand, trace gravel	3.35	SS SS	4 5	75 82	18 14
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders, some clay	3.66				
BEDROCK: Poor to fair quality, grey silty dolostone interbedded with grey limestone - vertical quartz seam from 4.5 to 5.4m depth	6.00	RC RC	1 2	100 100	44 53
End of Borehole (GWL @ 3.09m - July 26, 2021)					

DEPTH (m)	ELEV. (m)
0	78.24
1	77.24
2	76.24
3	75.24
4	74.24
5	73.24
6	72.24



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SOIL PROFILE AND TEST DATA

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DATE July 15, 2021

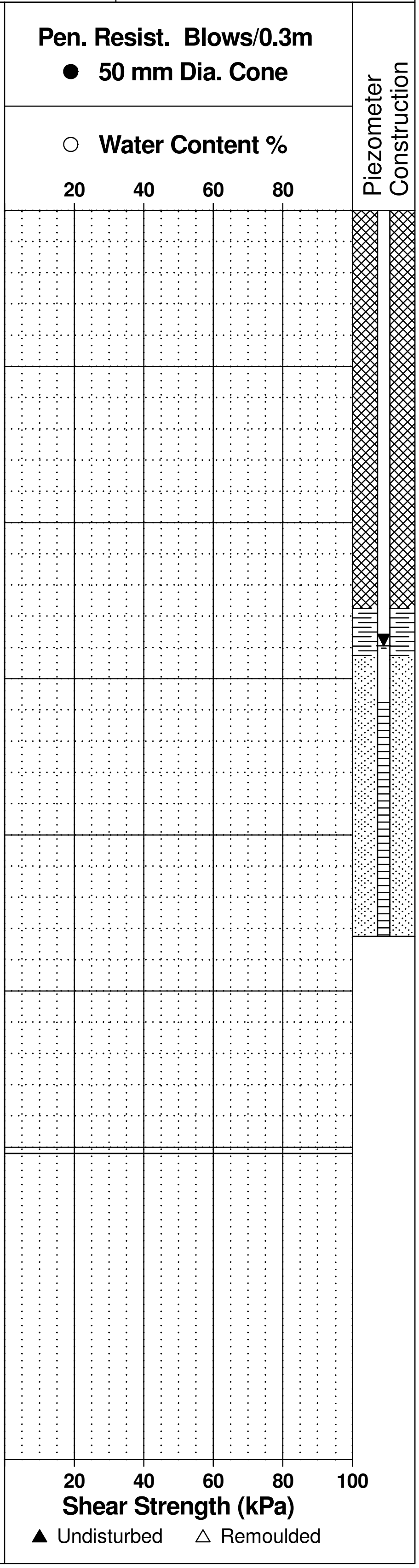
FILE NO. PG5887

HOLE NO. BH 5-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.05
FILL: Brown silty sand, some clay, trace gravel	0.76
Hard to very stiff, brown **SILTY CLAY**	1.83
GLACIAL TILL: Brown silty clay, some sand and gravel	2.95
BEDROCK: Poor to fair quality, grey silty dolostone interbedded with grey limestone - vertical quartz seam from 3.6 to 4.0m depth	6.04
End of Borehole (GWL @ 2.79m - July 26, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	83	18
SS	3	83	7
SS	4	75	10
RC	1	100	46
RC	2	100	67
RC	3	100	72

DEPTH (m)	ELEV. (m)
0	77.88
1	76.88
2	75.88
3	74.88
4	73.88
5	72.88
6	71.88



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Ottawa, Ontario

DATUM Geodetic

REMARKS

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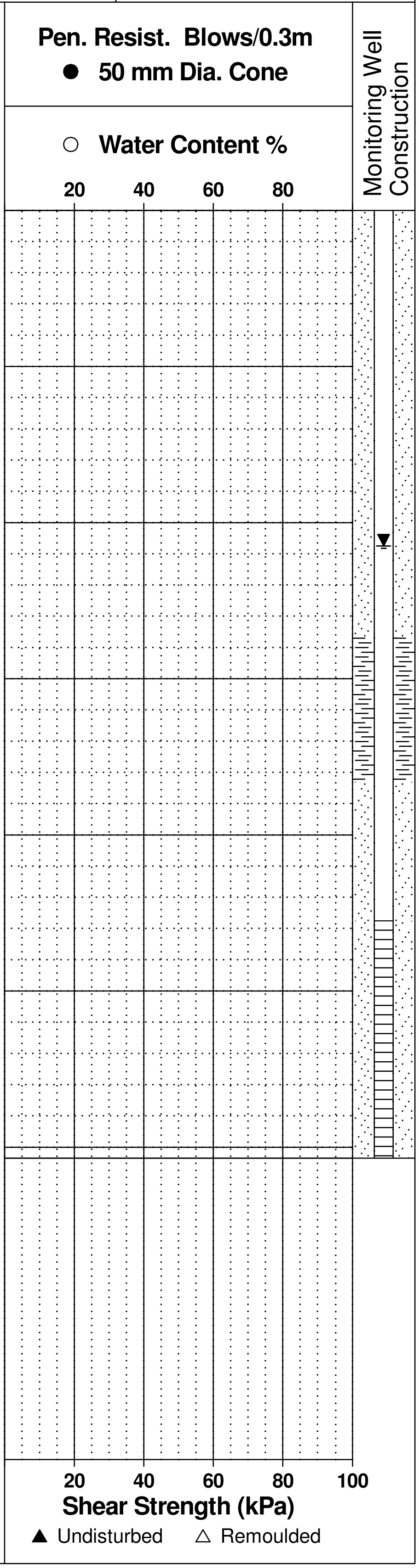
DATE July 15, 2021

FILE NO. PG5887

HOLE NO. BH 6-21

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with crushed stone and gravel	0.84	AU	1		
Hard to very stiff, brown **SILTY CLAY**	3.05	SS	2	83	13
		SS	3	83	12
		SS	4	83	6
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles	3.15	SS	5	100	50+
BEDROCK: Poor to good quality, grey silty dolostone interbedded with grey limestone - vertical seam from 3.25 to 4.0m depth	6.07	RC	1	100	40
		RC	2	100	83
End of Borehole (GWL @ 2.15m - July 26, 2021)					

DEPTH (m)	ELEV. (m)
0	77.22
1	76.22
2	75.22
3	74.22
4	73.22
5	72.22
6	71.22



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5887**

REMARKS

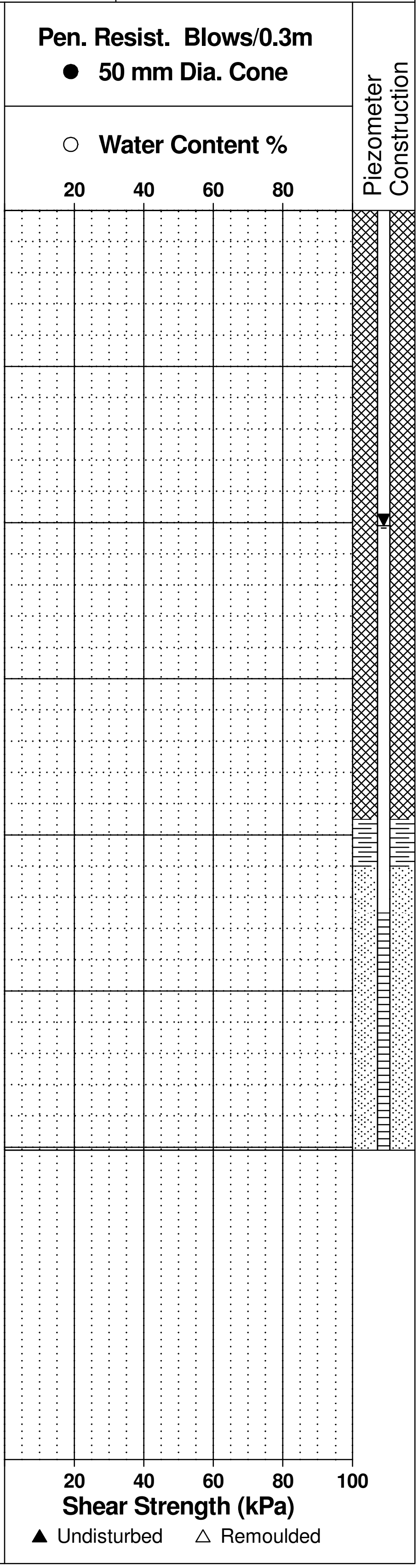
HOLE NO. **BH 7-21**

BORINGS BY CME-55 Low Clearance Drill

DATE July 15, 2021

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.30				
FILL: Brown silty sand, trace gravel 0.76	AU	1		
Hard to very stiff, brown **SILTY CLAY**	SS	2	83	17
1.98	SS	3	83	12
GLACIAL TILL: Brown silty clay, some sand and gravel	SS	4	83	5
- grey by 3.0m depth	SS	5	40	50+
3.99				
BEDROCK: Excellent quality, grey limestone	RC	1	100	100
	RC	2	100	98
6.02				
End of Borehole				
(GWL @ 2.02m - July 26, 2021)				

DEPTH (m)	ELEV. (m)
0	77.17
1	76.17
2	75.17
3	74.17
4	73.17
5	72.17
6	71.17



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

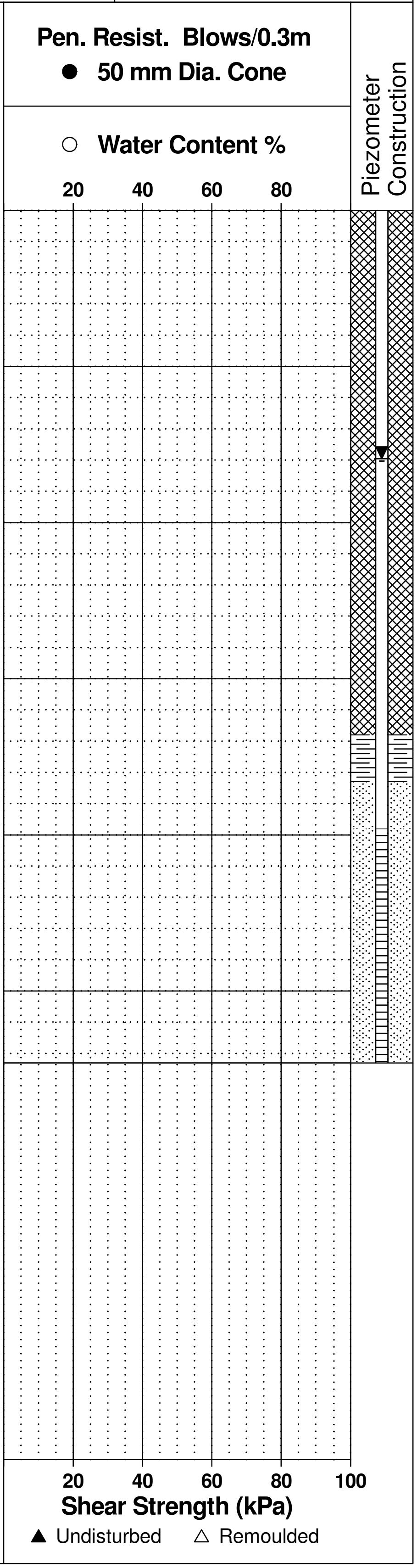
BORINGS BY CME-55 Low Clearance Drill

DATE July 15, 2021

FILE NO. PG5887

HOLE NO. BH 8-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	76.63
TOPSOIL 0.30						
FILL: Brown silty sand with gravel 0.60	AU	1				
Hard to very stiff, brown **SILTY CLAY** - trace sand and gravel by 2.0m depth	SS	2	79	14	1	75.63
	SS	3	83	6	2	74.63
3.02	SS	4	83	2	3	73.63
BEDROCK: Excellent to good quality, grey silty dolostone interbedded with grey limestone	RC	1	100	92	4	72.63
5.46	RC	2	100	79	5	71.63
End of Borehole (GWL @ 1.59m - July 26, 2021)						



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

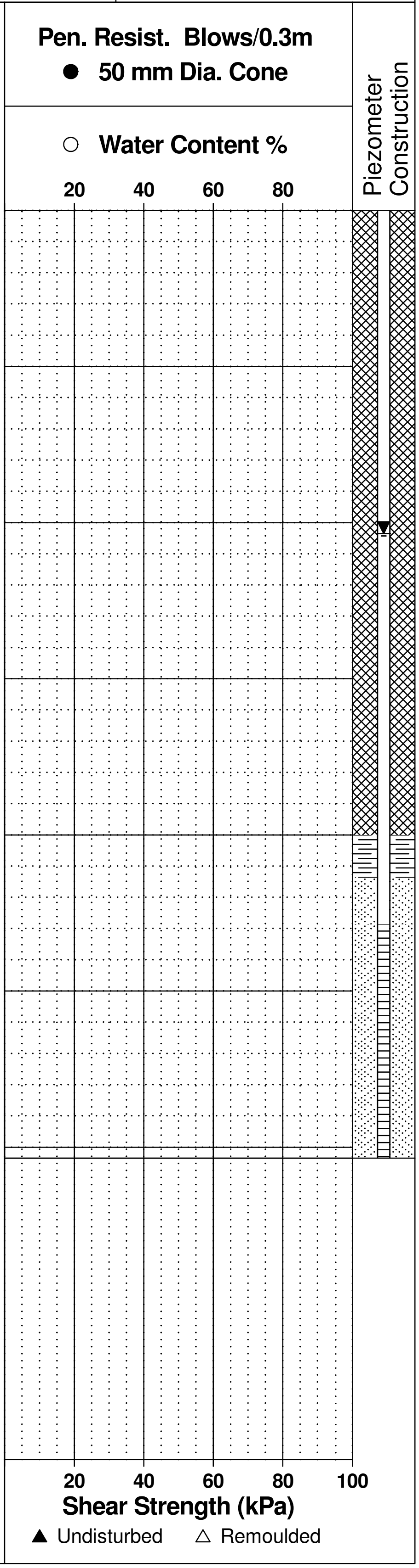
BORINGS BY CME-55 Low Clearance Drill

DATE July 16, 2021

FILE NO. PG5887

HOLE NO. BH 9-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.19
FILL: Brown silty sand with organics, some gravel — 0.53		AU	1				
FILL: Brown silty clay, some topsoil, trace gravel — 0.91							
Hard to very stiff, brown **SILTY CLAY**		SS	2	12	21	1	75.19
- some sand, trace gravel by 2.6m depth		SS	3	75	8	2	74.19
— 2.97		SS	4	83	7	3	73.19
GLACIAL TILL: Compact, brown silty sand with gravel, trace clay — 3.50		SS	5	94	22		
BEDROCK: Excellent to good quality, grey silty dolostone interbedded with grey limestone		RC	1	100	98	4	72.19
						5	71.19
— 6.07		RC	2	100	88	6	70.19
End of Borehole							
(GWL @ 2.07m - July 26, 2021)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5887

REMARKS

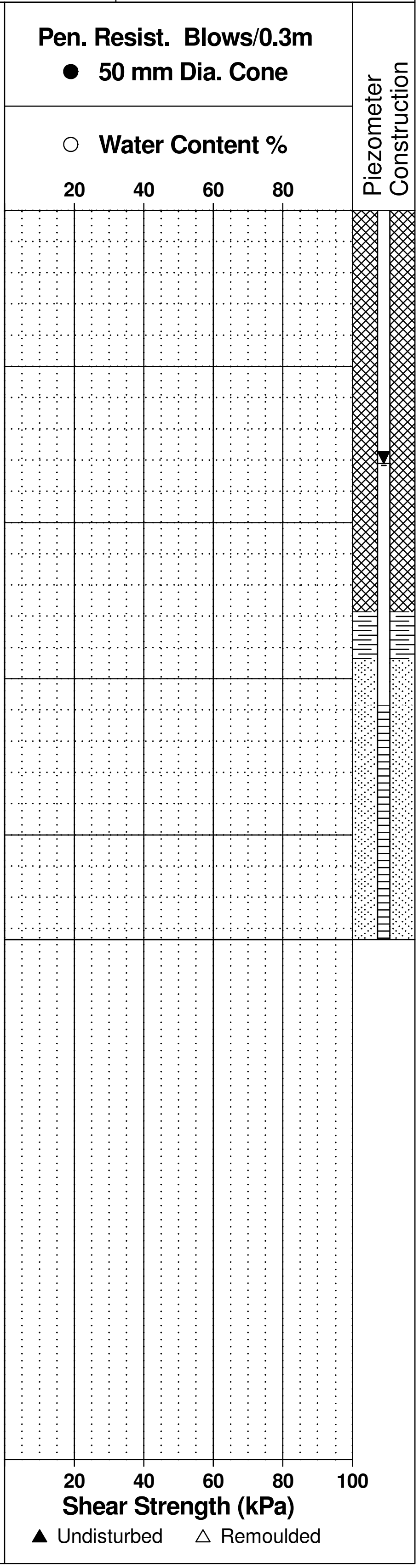
HOLE NO. BH10-21

BORINGS BY CME-55 Low Clearance Drill

DATE July 16, 2021

SOIL DESCRIPTION GROUND SURFACE	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
FILL: Brown silty sand with crushed stone, some topsoil and gravel	AU	1		
	SS	2	83	34
1.68	SS	3	33	50+
FILL: Brown silty sand with crushed stone 2.36				
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with grey limestone	RC	1	100	87
	RC	2	100	97
4.67 End of Borehole (GWL @ 1.62m - July 26, 2021)				

DEPTH (m)	ELEV. (m)
0	75.45
1	74.45
2	73.45
3	72.45
4	71.45



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 16, 2021

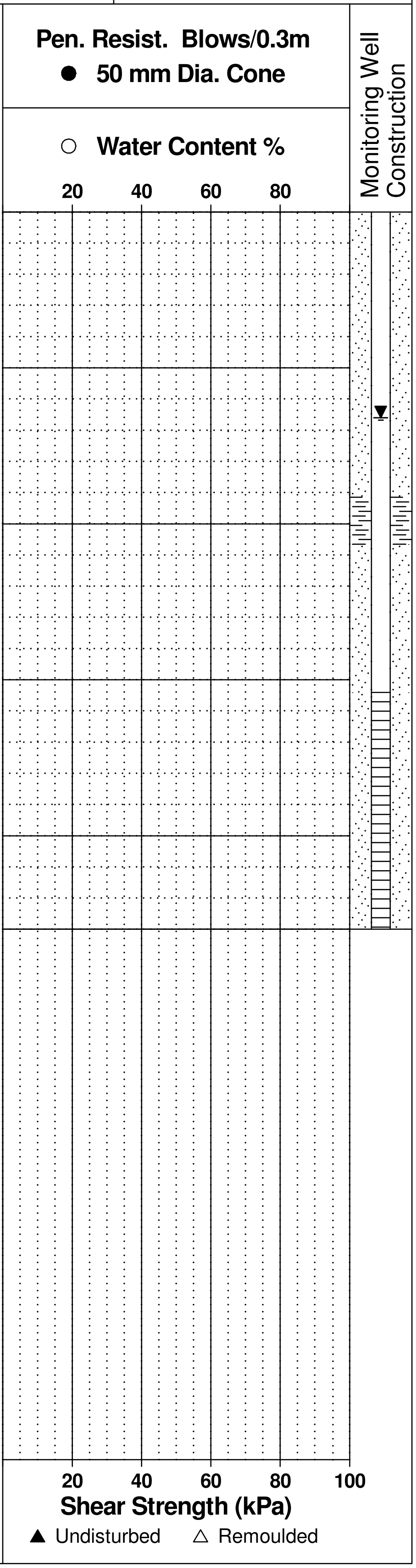
FILE NO. PG5887

HOLE NO. BH11-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
FILL: Brown silty sand with gravel	1.14
FILL: Brown silty sand with clay, trace gravel and brick	1.52
Grey **SILTY CLAY** with sand, trace gravel	2.34
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with grey limestone	4.60
End of Borehole	
(GWL @ 1.32m - July 26, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	22
SS	3	33	6
SS	4	100	50+
RC	1	100	79
RC	2	100	97

DEPTH (m)	ELEV. (m)
0	75.62
1	74.62
2	73.62
3	72.62
4	71.62



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 16, 2021

FILE NO. PG5887

HOLE NO. BH12-21

GROUND SURFACE

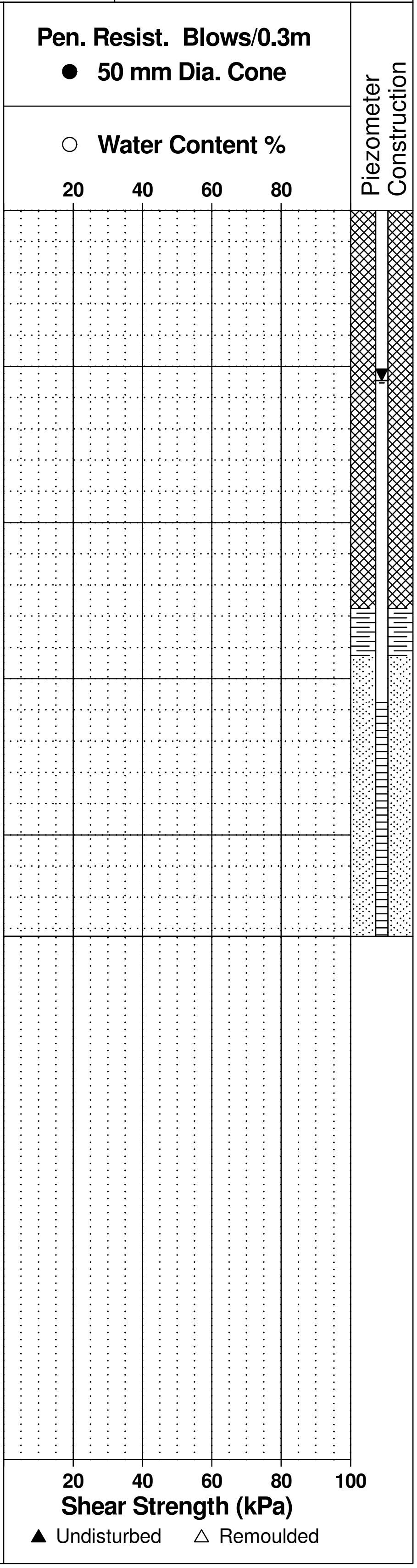
SOIL DESCRIPTION	Depth to base (m)
TOPSOIL	0.30
FILL: Brown silty sand with gravel, trace topsoil	0.91
FILL: Brown silty clay, trace brick and gravel	1.83
Hard to very stiff, brown **SILTY CLAY** - some sand, trace gravel by 2.4m depth	2.67
BEDROCK: Good to excellent quality, grey silty dolostone interbedded with grey limestone	4.65

End of Borehole

(GWL @ 1.09m - July 26, 2021)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	15
SS	3	75	12
SS	4	15	50+
RC	1	100	89
RC	2	100	100

DEPTH (m)	ELEV. (m)
0	75.91
1	74.91
2	73.91
3	72.91
4	71.91



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

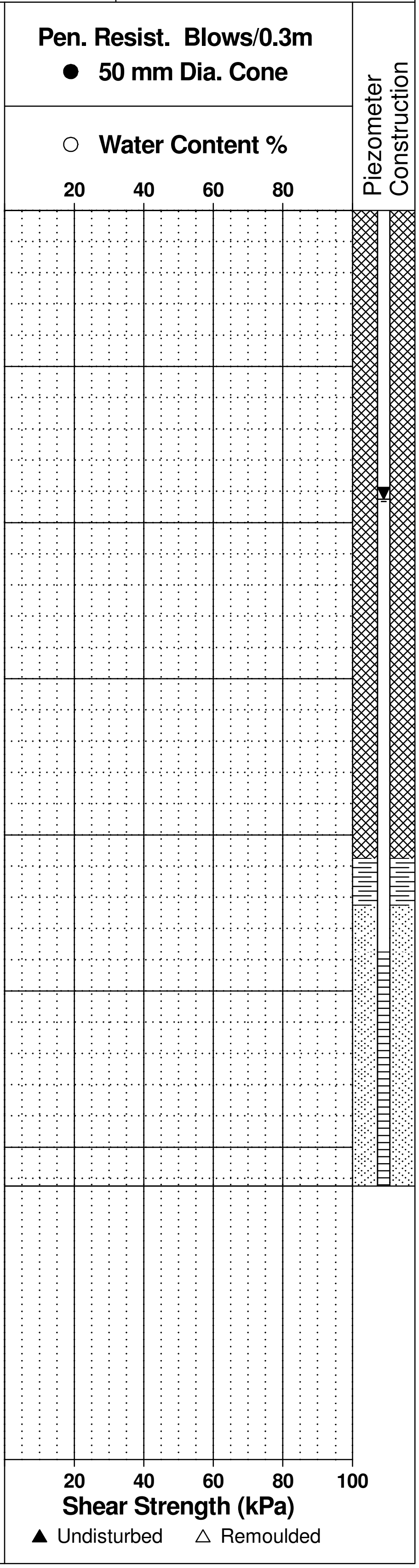
BORINGS BY CME-55 Low Clearance Drill

DATE July 19, 2021

FILE NO. PG5887

HOLE NO. BH13-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	76.84
FILL: Brown silty sand with clay and gravel		AU	1				
1.07		SS	2	0	16	1	75.84
GLACIAL TILL: Brown silty clay with sand, some gravel		SS	3	33	4		
						2	74.84
		SS	4	100	50+		
2.90		RC	1	100	100	3	73.84
BEDROCK: Good quality, grey silty dolostone interbedded with grey limestone		RC	2	100	77	4	72.84
						5	71.84
		RC	3	100	75		
						6	70.84
6.25							
End of Borehole (GWL @ 1.85m - July 26, 2021)							



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

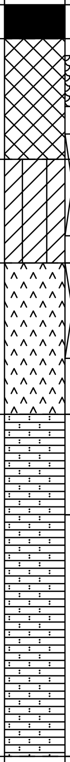
DATE July 19, 2021

FILE NO. PG5887

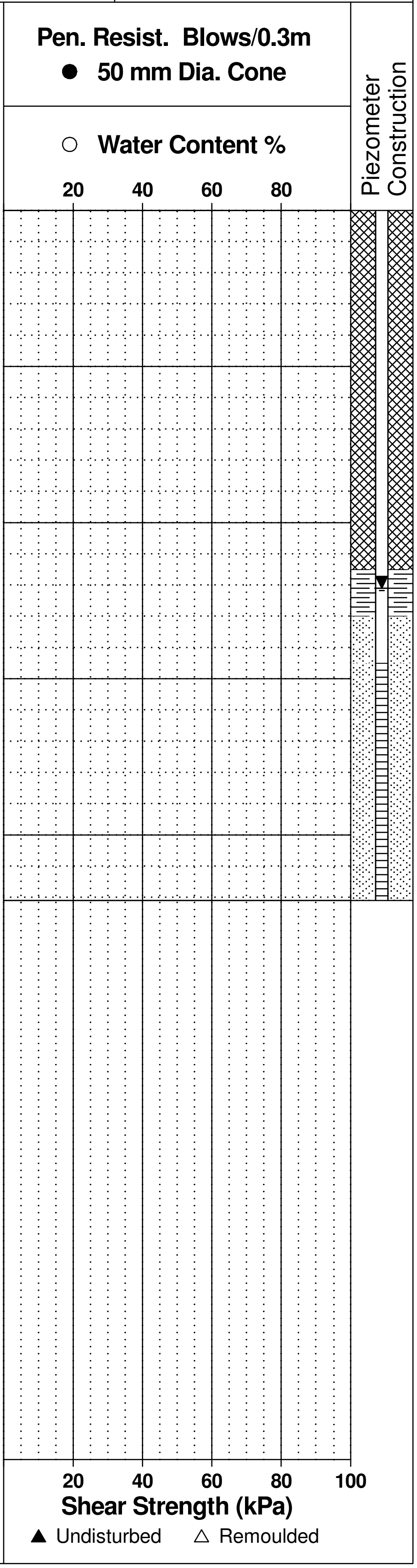
HOLE NO. BH14-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.20				
FILL: Brown silty sand with gravel, trace clay 0.91	AU	1		
Hard to very stiff, brown **SILTY CLAY** 1.52	SS	2	100	24
GLACIAL TILL: Brown silty clay with silty sand and gravel 2.41	SS	3	91	14
BEDROCK: Fair to good quality, grey silty dolostone interbedded with grey limestone 4.42	RC RC	1 2	100 100	70 80
End of Borehole				
(GWL @ 2.42m - July 26, 2021)				

STRATA PLOT



DEPTH (m)	ELEV. (m)
0	76.72
1	75.72
2	74.72
3	73.72
4	72.72



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

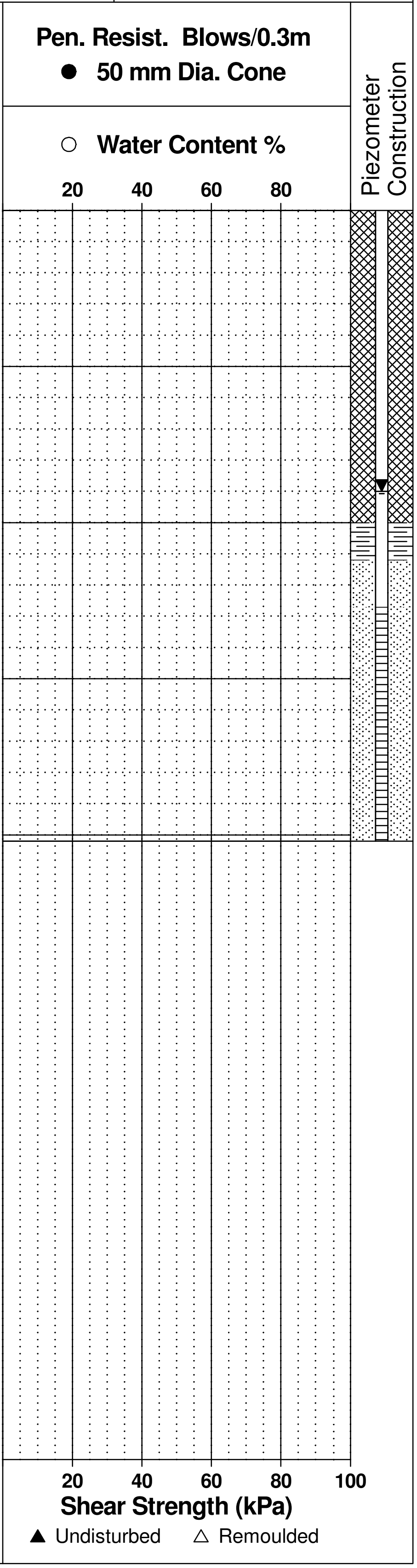
DATE July 19, 2021

FILE NO. PG5887

HOLE NO. BH15-21

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20				
FILL: Brown silty sand with crushed stone and gravel	0.60	AU	1		
Hard to very stiff, brown **SILTY CLAY**	1.52	SS	2	83	14
GLACIAL TILL: Brown silty clay with sand, some gravel, cobbles and boulders	2.03	SS	3	90	13
BEDROCK: Excellent quality, grey silty dolostone interbedded with grey limestone		RC	1	100	100
	4.04	RC	2	100	90
End of Borehole (GWL @ 1.80m - July 26, 2021)					

DEPTH (m)	ELEV. (m)
0	77.21
1	76.21
2	75.21
3	74.21
4	73.21



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Mixed-Use Development - 910 March Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

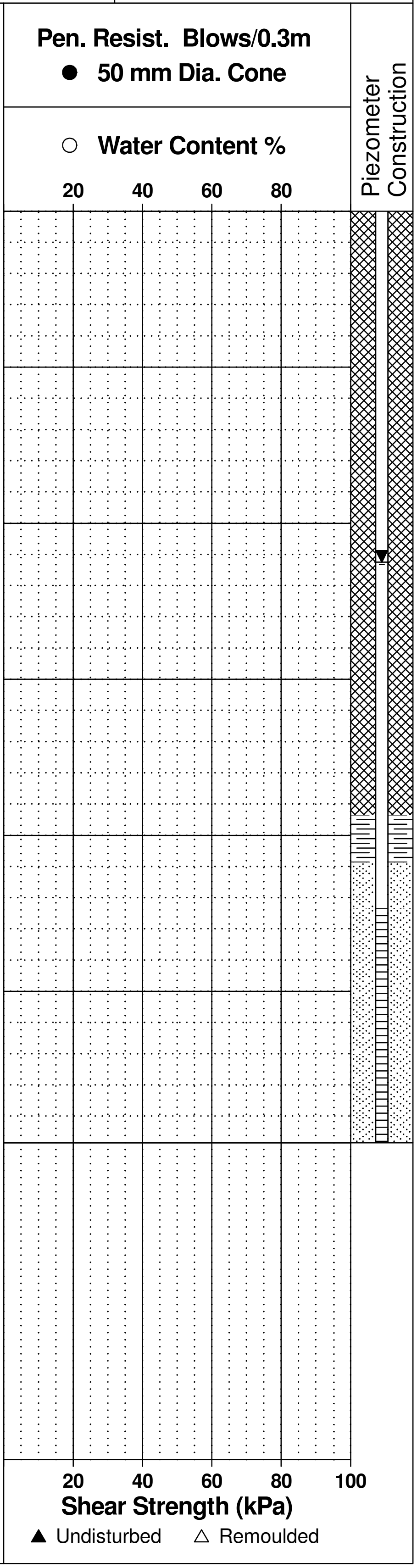
BORINGS BY CME-55 Low Clearance Drill

DATE July 19, 2021

FILE NO. PG5887

HOLE NO. BH16-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.47
FILL: 25mm Topsoil over brown silty sand with crushed stone and gravel		AU	1				
0.76							
		SS	2	83	20	1	76.47
Hard to very stiff, brown **SILTY CLAY**		SS	3	75	12	2	75.47
		SS	4	83	8		
- trace sand and gravel by 3.6m depth						3	74.47
		SS	5	83	4		
3.68							
		RC	1	100	71	4	73.47
BEDROCK: Fair to excellent quality, grey silty dolostone interbedded with grey limestone						5	72.47
		RC	2	100	98		
5.97							
End of Borehole							
(GWL @ 2.25m - July 26, 2021)							



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 strength of cohesionless soils is the relative density, 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.

Relative Density	‘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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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 is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture 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 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 x 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)

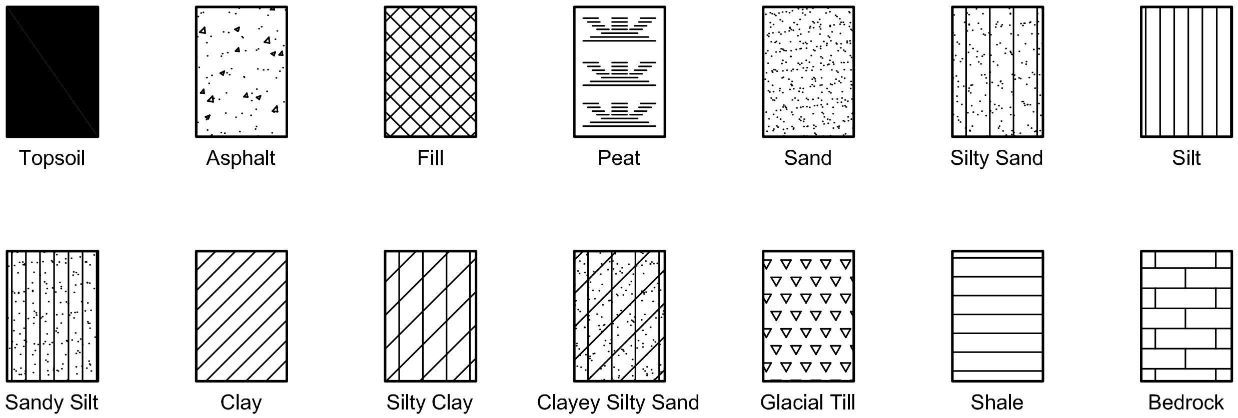
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

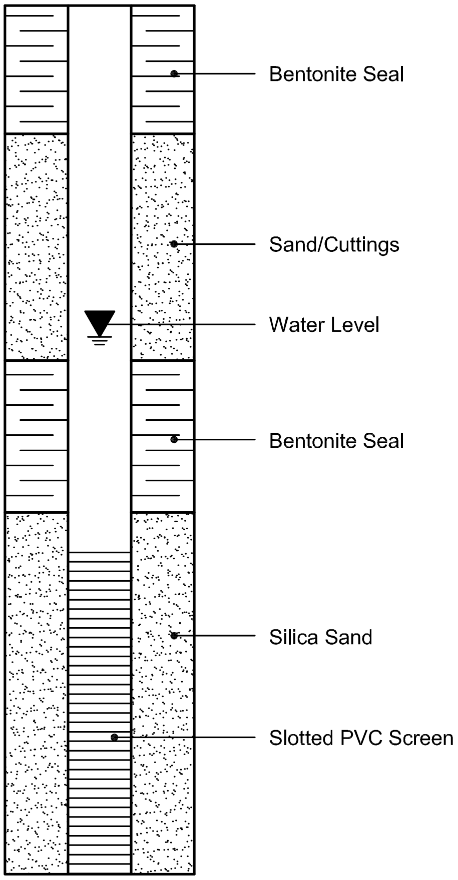
SYMBOLS AND TERMS (continued)

STRATA PLOT

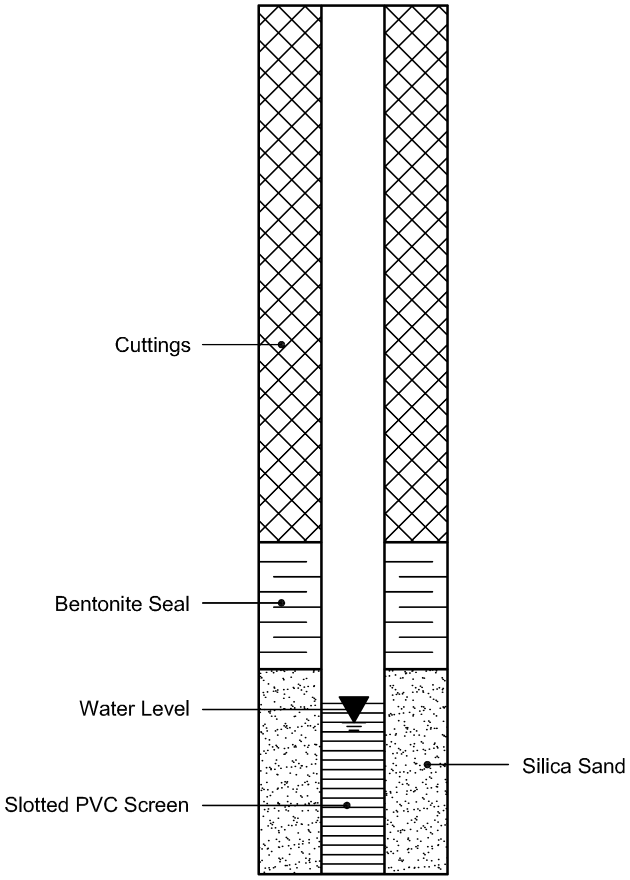


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32460

Report Date: 23-Jul-2021

Order Date: 20-Jul-2021

Project Description: PG5887

Client ID:		BH16-21 SS5	-	-	-
Sample Date:		19-Jul-21 09:00	-	-	-
Sample ID:		2130233-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	70.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.33	-	-	-
Resistivity	0.10 Ohm.m	58.2	-	-	-
Anions					
Chloride	5 ug/g dry	22	-	-	-
Sulphate	5 ug/g dry	74	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURES 4 TO 7 – SLOPE STABILITY CROSS SECTIONS

DRAWING PG5887-1 – TEST HOLE LOCATION PLAN

DRAWING PG5887-2 – BEDROCK CONTOUR PLAN

DRAWING PG5887-3 – LIMIT OF HAZARD LANDS PLAN

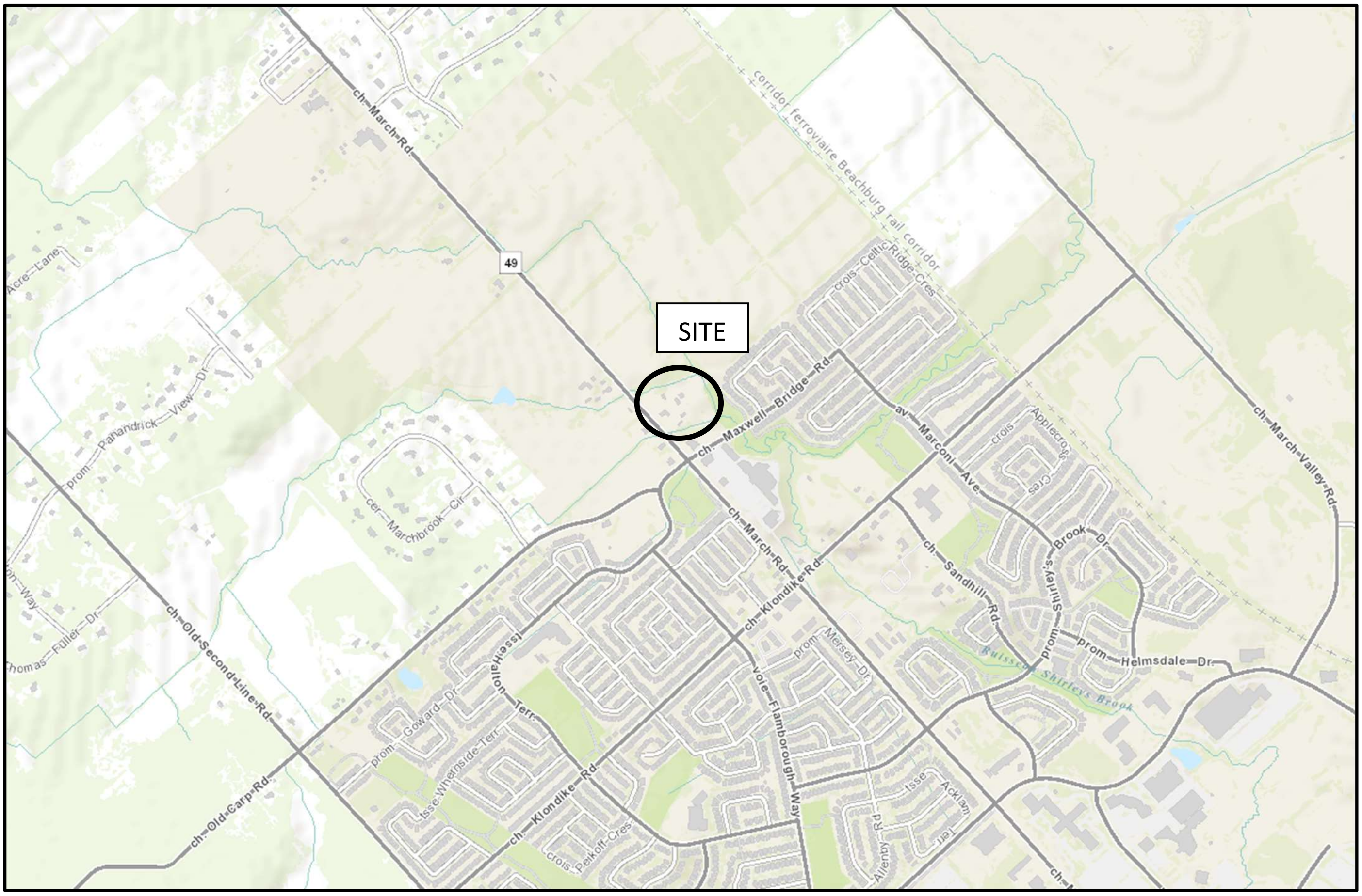


FIGURE 1

KEY PLAN

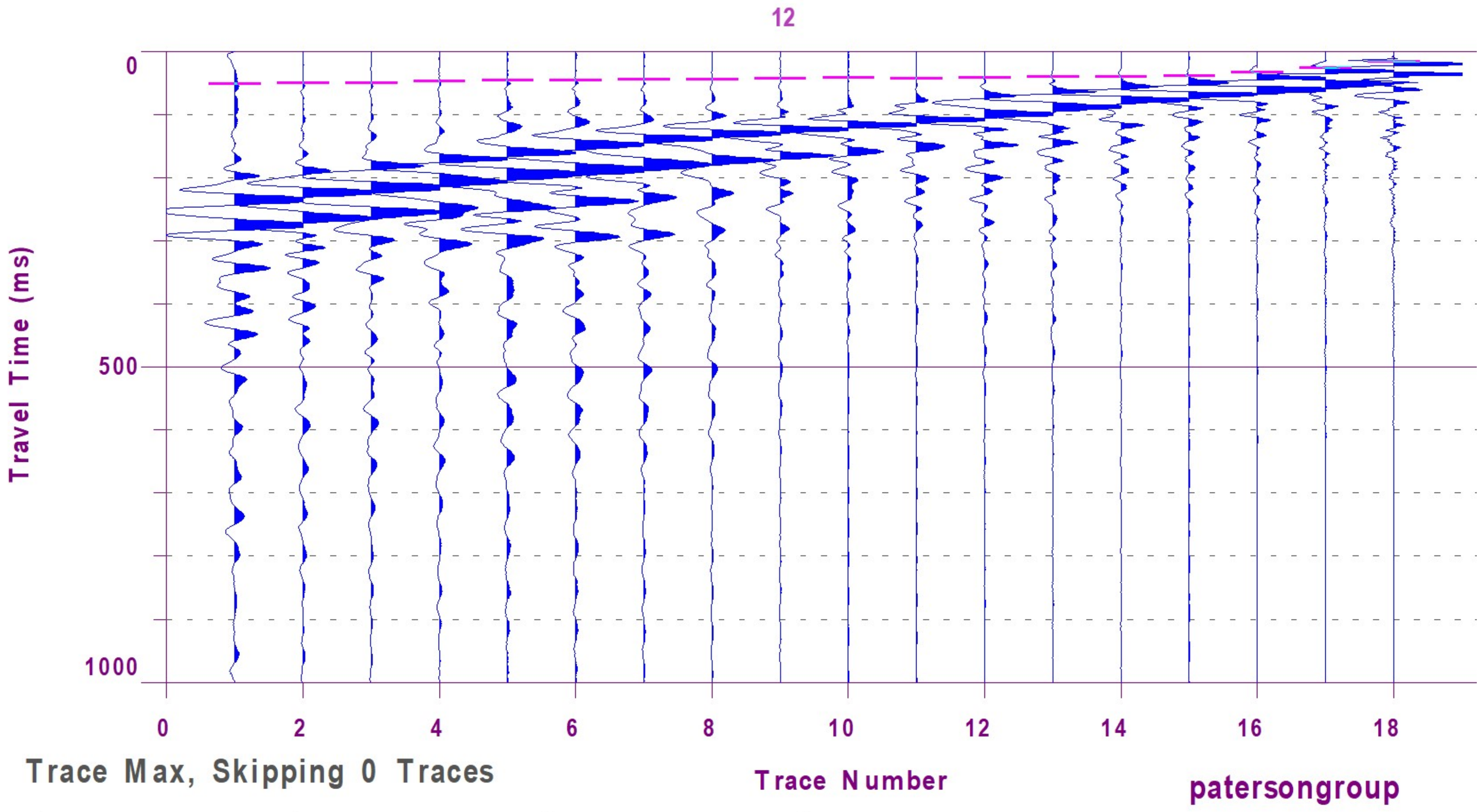


Figure 2 – Shear Wave Velocity Profile at Shot Location 36 m

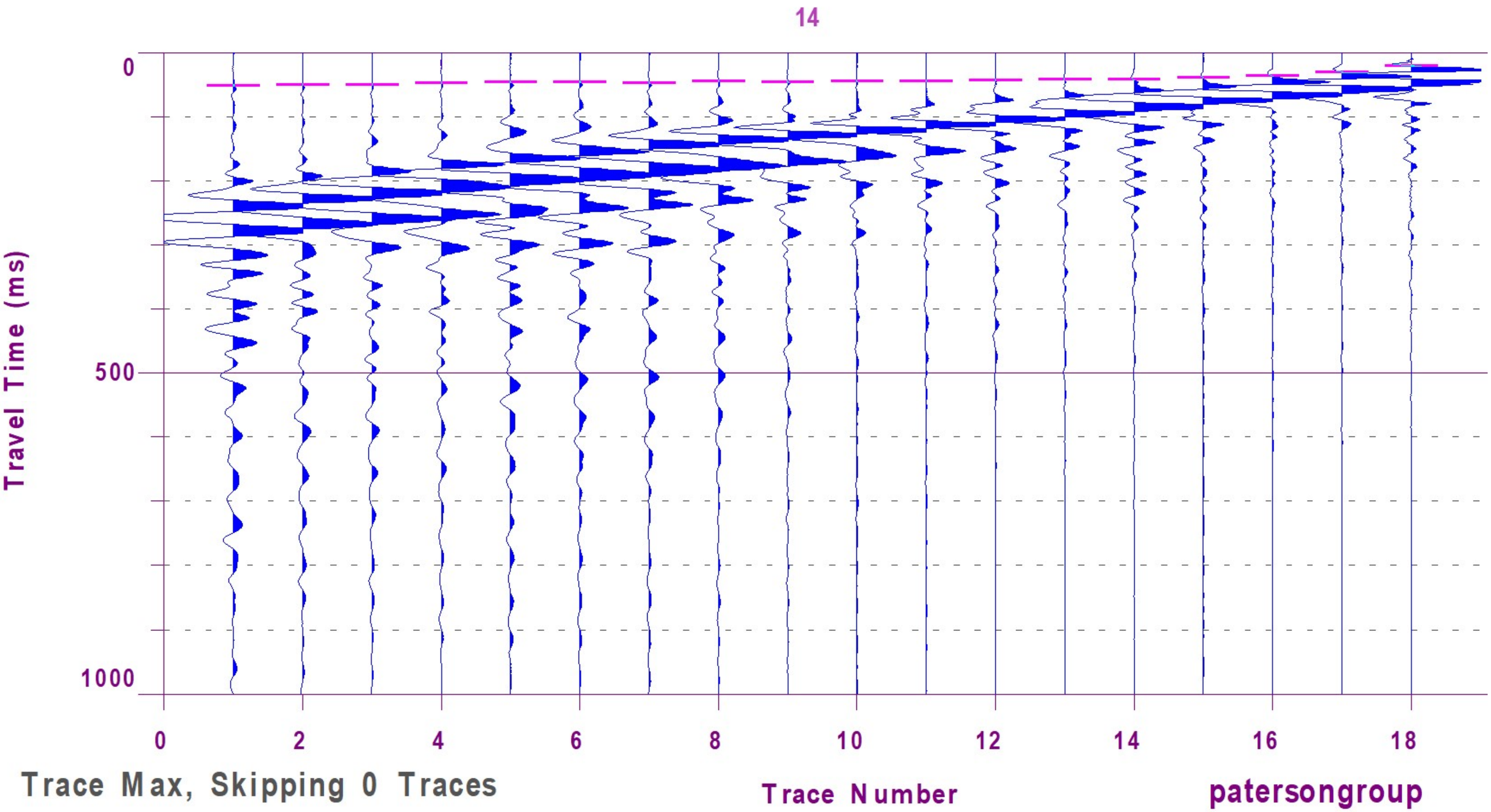


Figure 3 – Shear Wave Velocity Profile at Shot Location 37 m

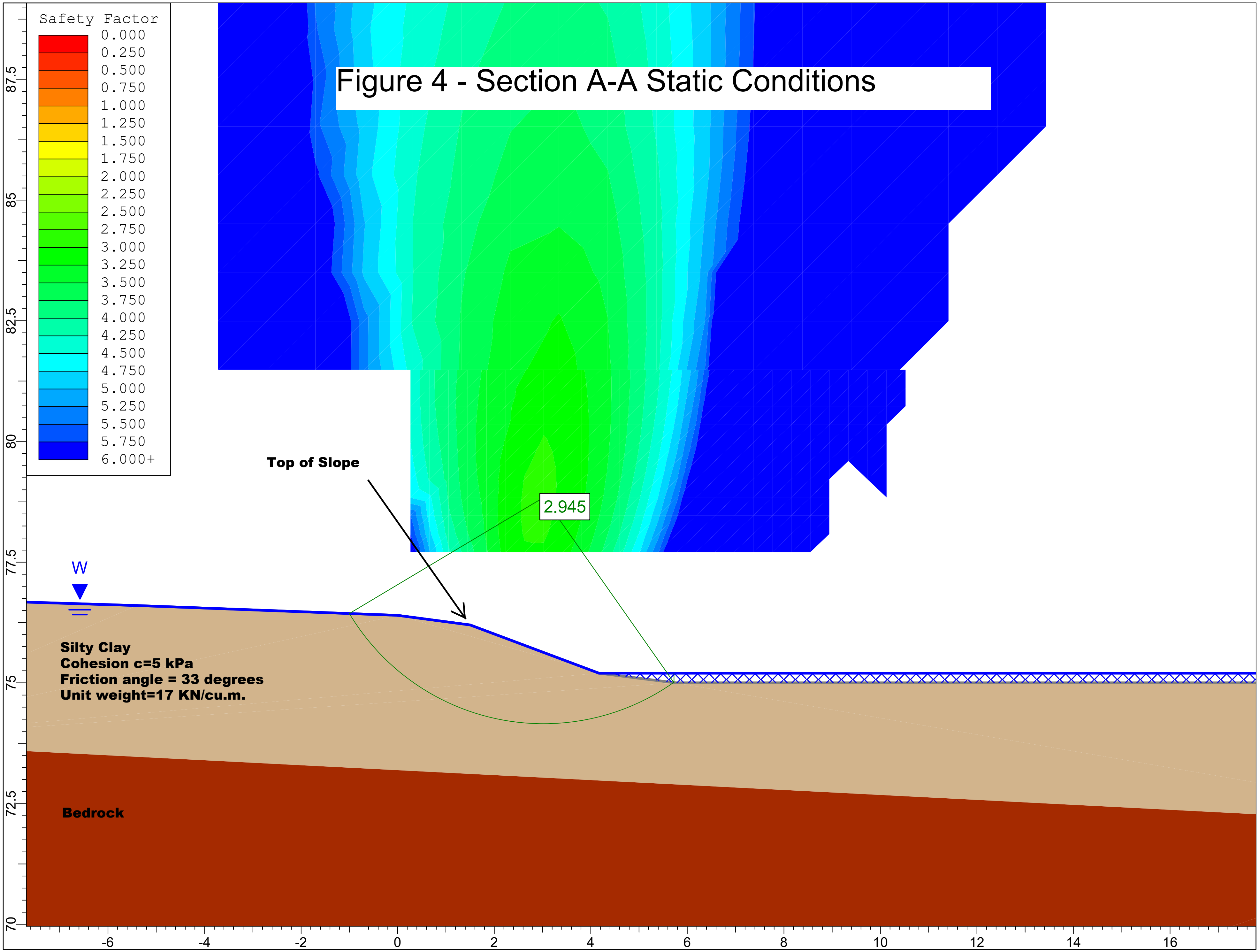


Figure 4 - Section A-A Static Conditions

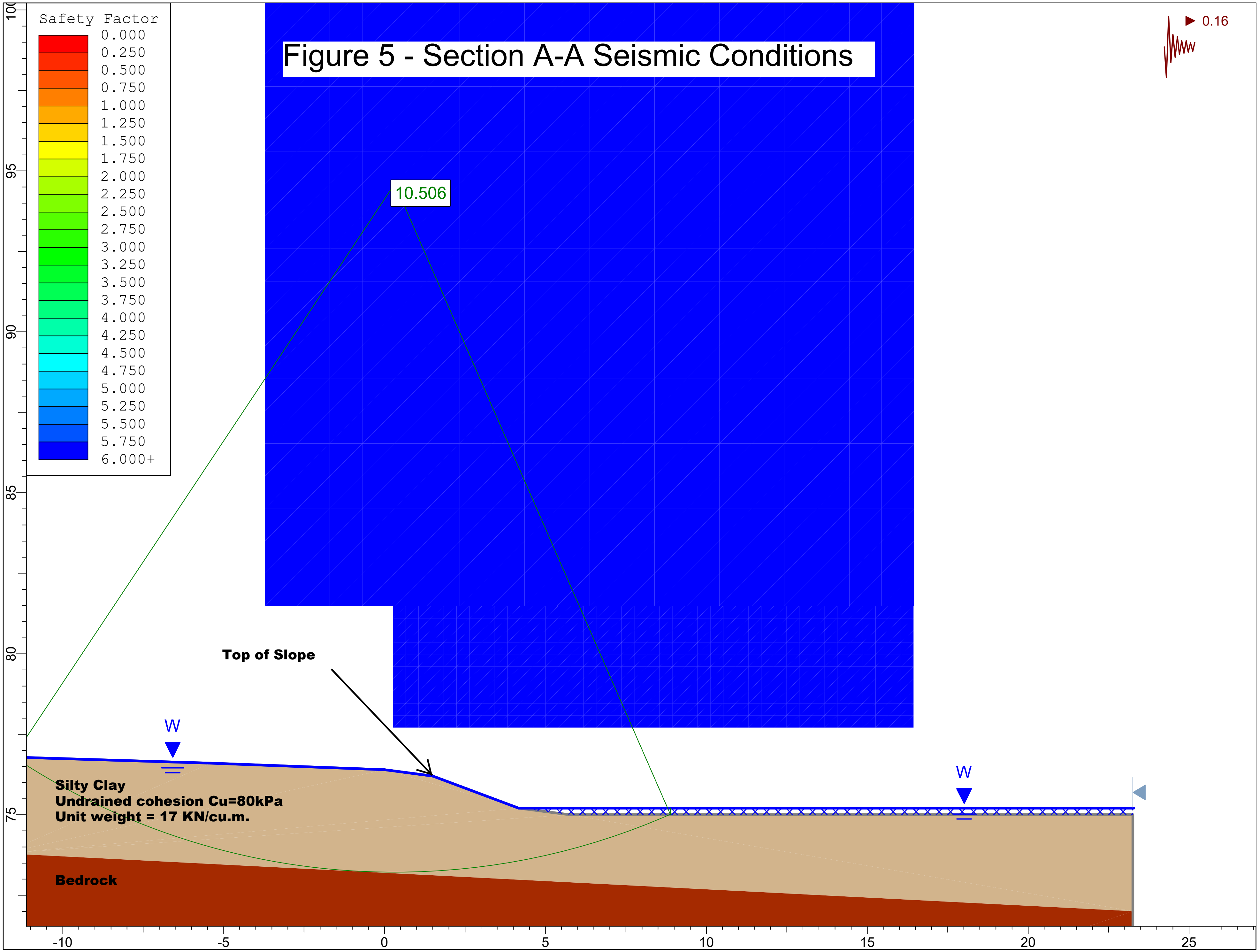


Figure 5 - Section A-A Seismic Conditions

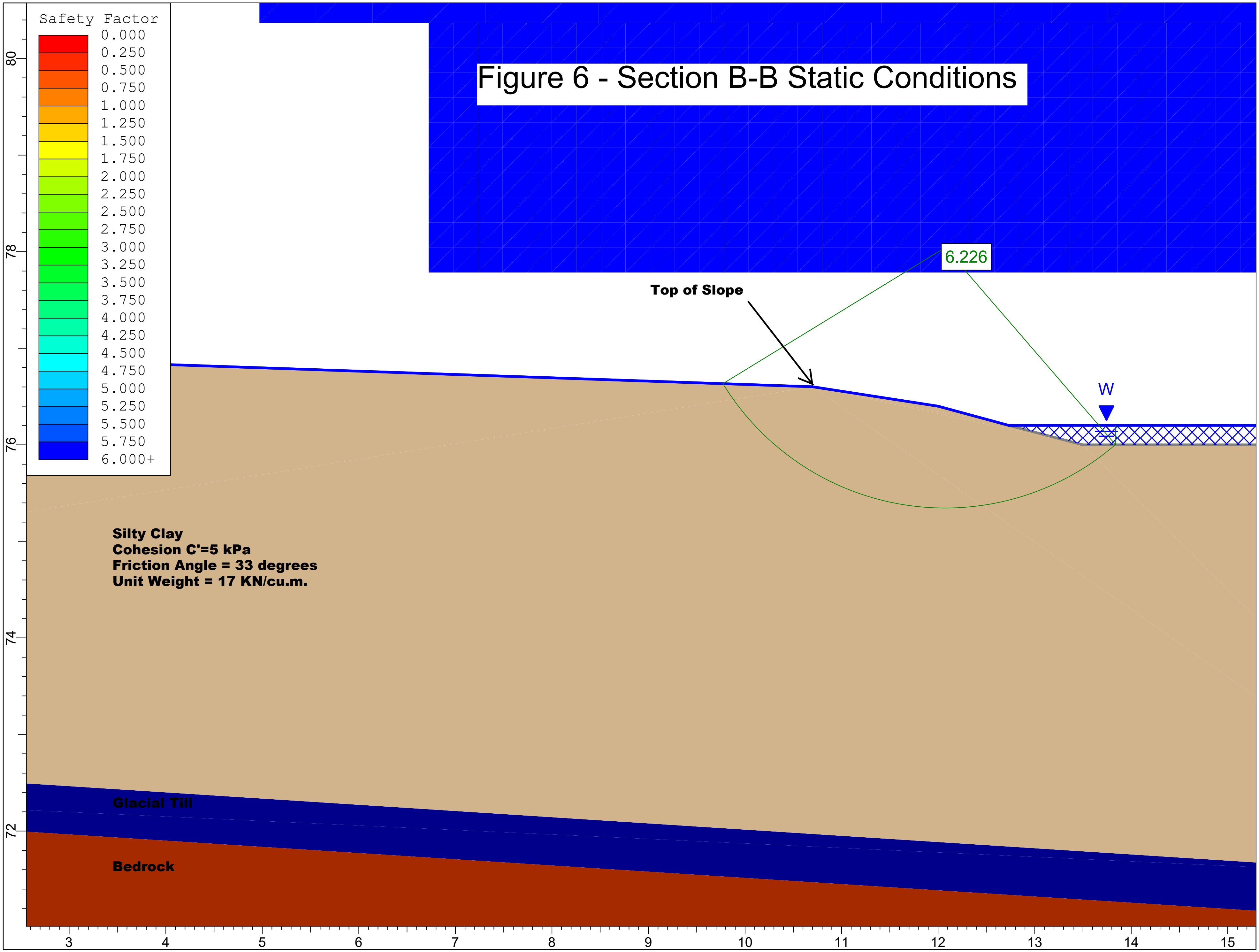


Figure 6 - Section B-B Static Conditions

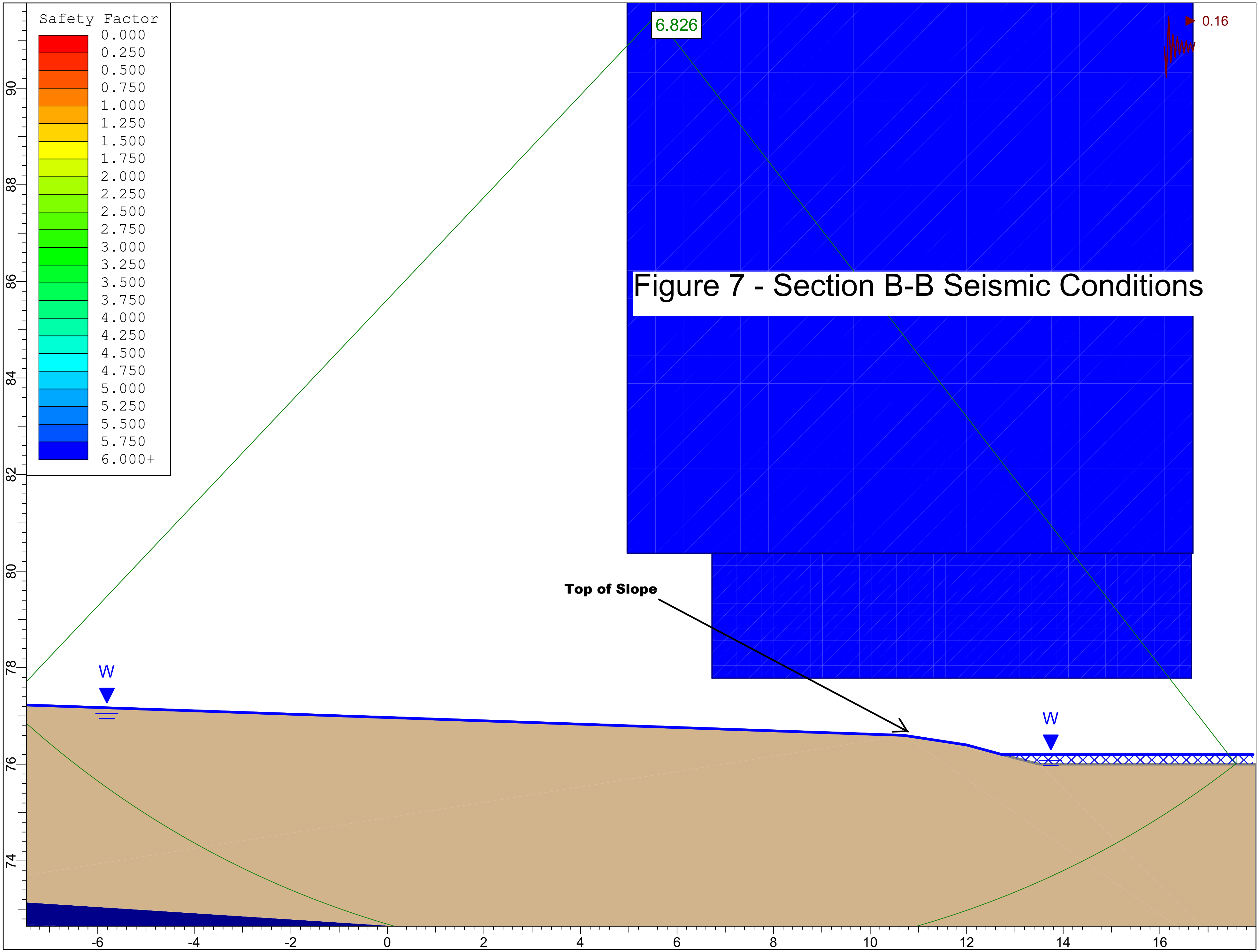
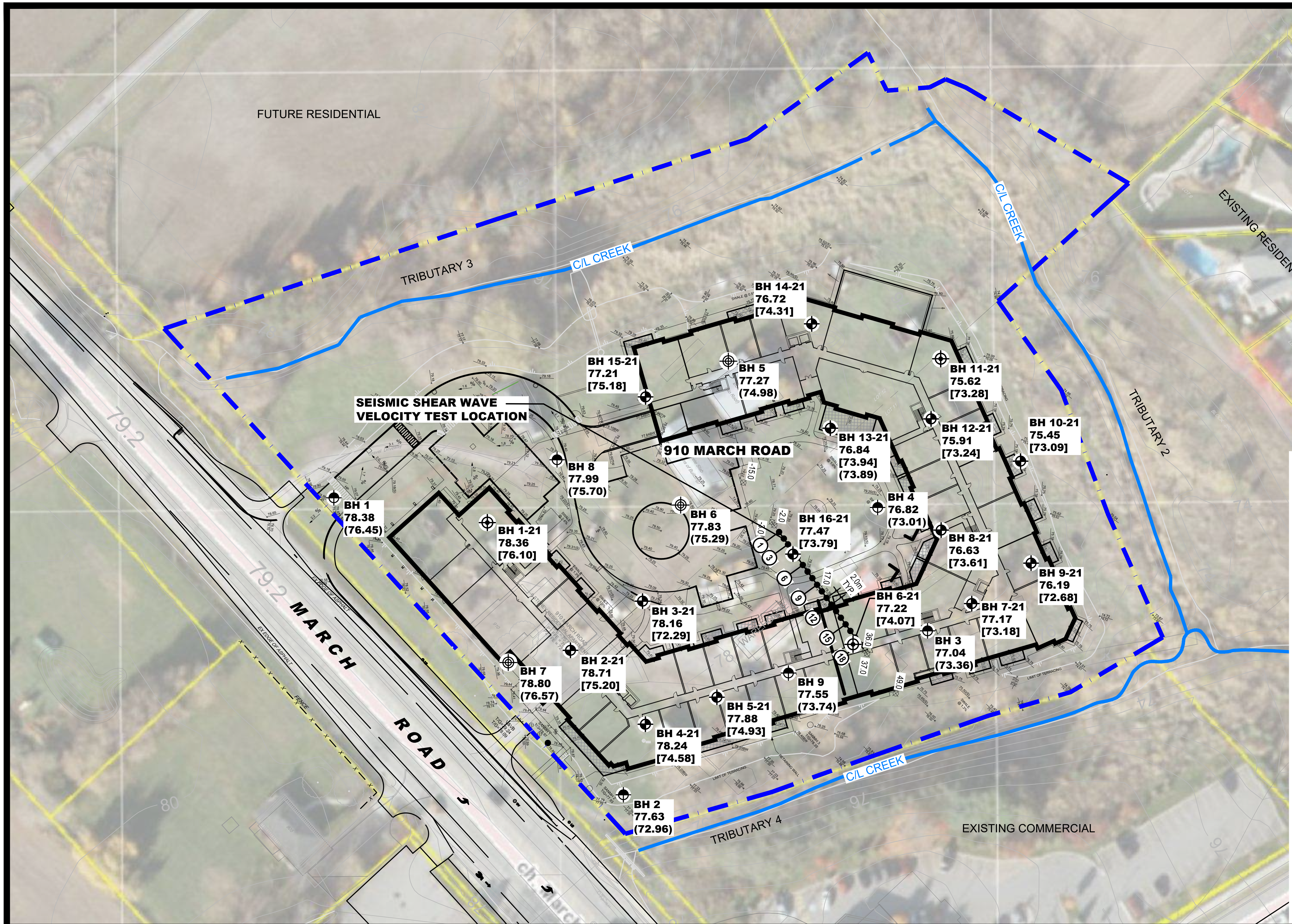


Figure 7 - Section B-B Seismic Conditions



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT; PG5119, 2019)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; PG5119, 2019)
• • •	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
+ +10.0	SHOT LOCATION
76.63	GROUND SURFACE ELEVATION (m)
[73.61]	BEDROCK SURFACE ELEVATION (m)
(73.36)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NOVATECH

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
2	UPDATED TO NEW CONCEPTUAL PLAN	21/12/2023	ZM
1	UPDATED TO NEW BASE PLAN	16/11/2023	ZM

CANADIAN RENTAL DEVELOPMENT SERVICES INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
910 MARCH ROAD

OTTAWA, ONTARIO

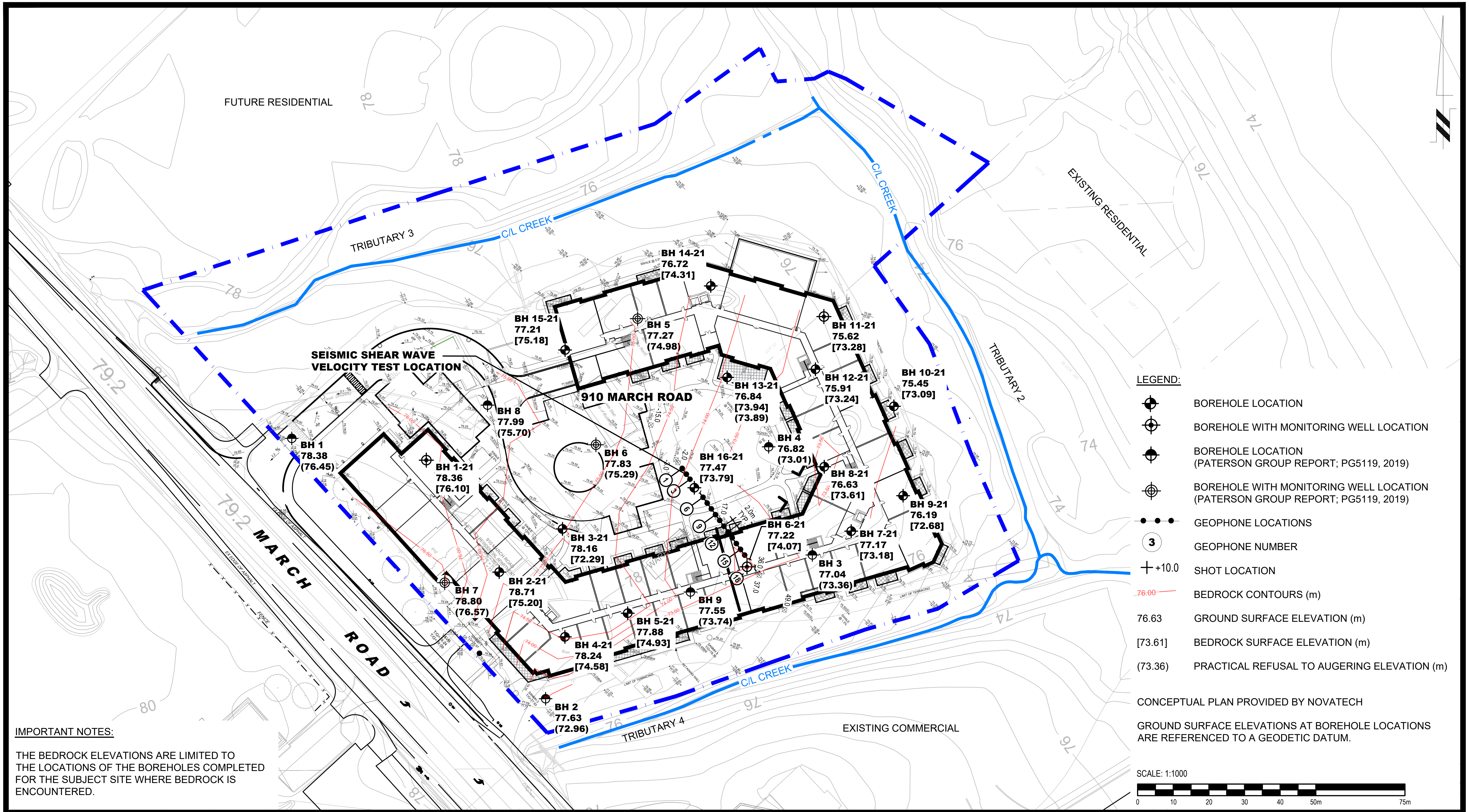
Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 07/2021
Drawn by: JM	Report No.: PG5887-1
Checked by: MS	Dwg. No.: PG5887-1
Approved by: DJG	Revision No.: 2

11x17

p:\autocad drawings\geotechnical\pg58xx\pg5887\pg5887-3 limits of hazard lands plan (rev.03) (21-12-2023).dwg



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⌖	BOREHOLE LOCATION (PATERSON GROUP REPORT; PG5119, 2019)
⌖	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; PG5119, 2019)
● ● ●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
+ +10.0	SHOT LOCATION
76.00	BEDROCK CONTOURS (m)
76.63	GROUND SURFACE ELEVATION (m)
[73.61]	BEDROCK SURFACE ELEVATION (m)
(73.36)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NOVATECH

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

IMPORTANT NOTES:

THE BEDROCK ELEVATIONS ARE LIMITED TO THE LOCATIONS OF THE BOREHOLES COMPLETED FOR THE SUBJECT SITE WHERE BEDROCK IS ENCOUNTERED.

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
2	UPDATED TO NEW CONCEPTUAL PLAN	21/12/2023	ZM
1	UPDATED TO NEW BASE PLAN	16/11/2023	ZM

CANADIAN RENTAL DEVELOPMENT SERVICES INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
910 MARCH ROAD

OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:1000	Date: 07/2021
Drawn by: JM	Report No.: PG5887-1
Checked by: MS	Dwg. No.: PG5887-2
Approved by: DJG	Revision No.: 2

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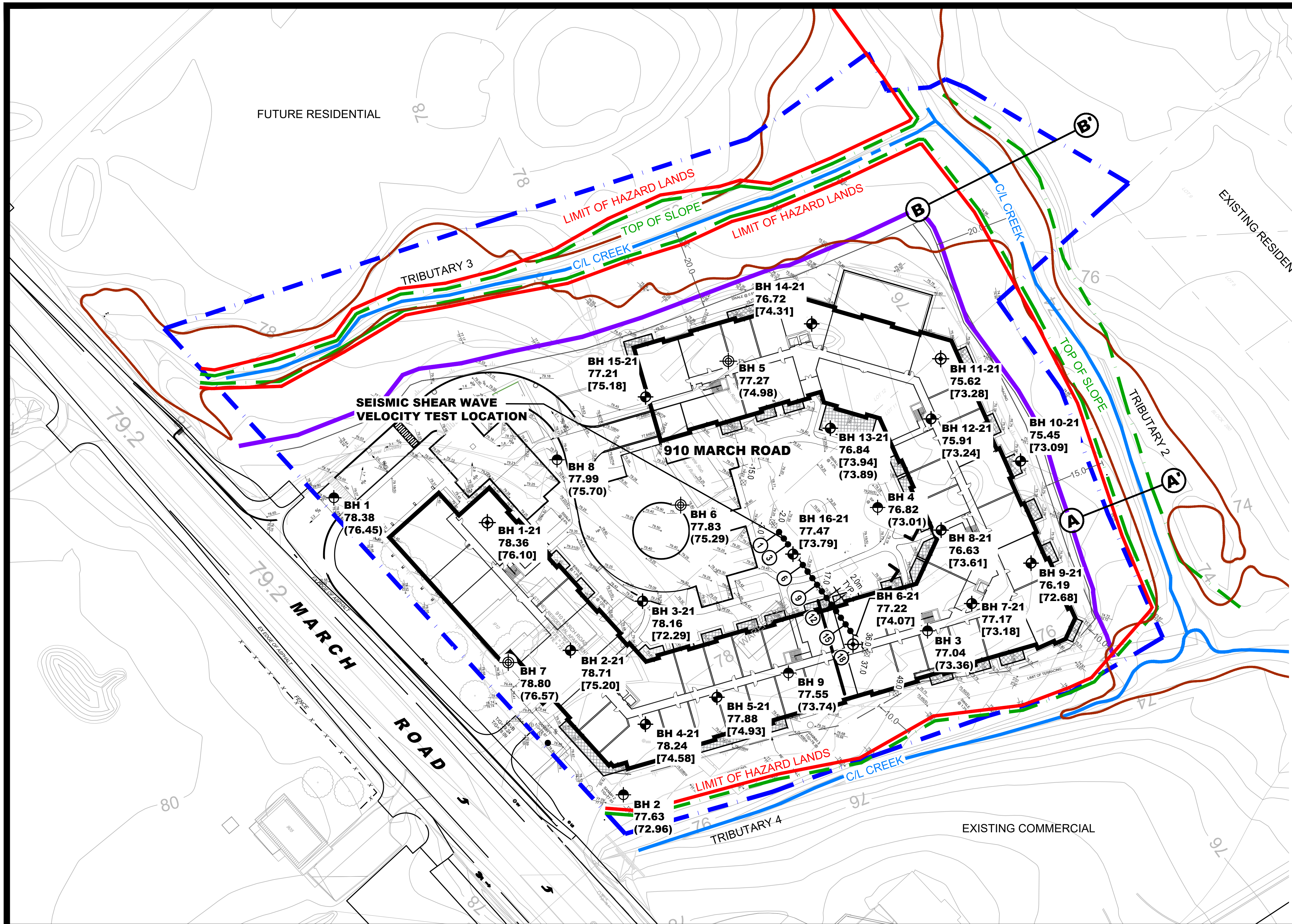
11x17



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT; PG5119, 2019)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT; PG5119, 2019)
● ● ●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
+ +10.0	SHOT LOCATION
Ⓐ—Ⓐ'	SLOPE STABILITY CROSS-SECTION LOCATION
—	MEANDER BELT LIMIT (KANATA NORTH CDP, ENVIRONMENTAL MASTER P[LAN, NOVATECH, DATED: JUNE 28,2016)
—	1:100 YEARS FLOOD PLAIN - MVCA (INTERPOLATED FROM MVCA MAPPING DATED: DECEMBER 06, 2017)
76.63	GROUND SURFACE ELEVATION (m)
[73.61]	BEDROCK SURFACE ELEVATION (m)
(73.36)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NOVATECH

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
3	UPDATED TO NEW CONCEPTUAL PLAN	22/12/2023	ZM
2	UPDATED TO NEW BASE PLAN	16/11/2023	ZM
1	ADDED MVCA LIMITS TO THE PLAN	03/05/2023	MS

CANADIAN RENTAL DEVELOPMENT SERVICES INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
910 MARCH ROAD

OTTAWA, ONTARIO

Title:

LIMIT OF HAZARD LANDS PLAN

Scale: 1:1000	Date: 07/2021
Drawn by: JM	Report No.: PG5887-1
Checked by: MS	Dwg. No.: PG5887-3
Approved by: DJG	Revision No.: 3

p:\autocad drawings\geotechnical\pg58xx\pg5887\pg5887-3 limits of hazard lands plan (rev.03) (21-12-2023).dwg

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