Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

patersongroup

Geotechnical Investigation

Proposed Multi-Storey Building
180 Kanata Avenue
Ottawa, Ontario

Prepared For

Kanata Woods Inc.

December 1, 2021

Report: PG5758-1
Revision 1

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

Table of Contents

Appendices

Appendix 1 Soil Profile and Test Data Sheets
Symbols and Terms
Analytical Test Results

Appendix 2 Figure 1 - Key Plan
Figures 2 & 3 - Seismic Shear Wave Velocity Profiles
Figures 4 & 5 – Slope Stability Cross Sections
Drawing PG5758-1 - Test Hole Location Plan
Drawing PG5758-2 – Rock Safety Setbacks

1.0 Introduction

Paterson Group (Paterson) was commissioned by Kanata Woods Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 180 Kanata Avenue in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.

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

The following report has been prepared specifically and solely for the aforementioned project which is described herein. 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 a mixed-use, multi-storey structure which will have up to 2 levels of underground parking (P1 & P2). The P1 underground parking level will occupy most of the site footprint, with the exception of the northwest quadrant of the site, and will extend beyond the footprint of the above-grade levels. The deeper P2 level will only be located under the eastern portion of the proposed multi-storey structure. It is also understood that the ground floor level of the proposed building, and finished grades surrounding the proposed building, will be approximately coincident with the grade of Kanata Avenue to the south at geodetic elevation 100 m. A vertical bedrock face, with a varying height of up to approximately 8 m, will be present along the north and west boundaries of the site.

The proposed building will generally be surrounded by asphalt-paved access lanes, vehicles parking areas, and landscaped areas. A paved access road is also proposed to the east of the proposed building, which will link to Kanata Avenue. It is also expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out during the period of March 29 through April 6, 2021, consisting of 8 boreholes which were advanced to a maximum depth of 10.8 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5758-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a 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 advancing each test hole to the required depths at the selected locations 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 classified on site, placed in sealed plastic bags and transported to our laboratory. The depths at which the auger, split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, 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 the boreholes 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.

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

Groundwater

All boreholes were fitted with flexible standpipe piezometers to allow for groundwater level monitoring. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

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

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 handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5758-1 - 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 Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently densely forested, with the exception of a few areas which were cleared to allow access for the geotechnical investigation. Bedrock outcroppings were also observed in the northwest portion of the site.

The site is bordered by undeveloped, forested lands to the north, east, and west, and by Kanata Avenue to the south. The existing ground surface slopes moderately from the northwest end of the site at approximate geodetic elevation 108 m, downward to the southeast end of the site at approximate geodetic elevation 100 m.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of topsoil overlying silty clay and/or silty sand.

A glacial till deposit was encountered underlying the silty clay and/or silty sand. The glacial till deposit generally consists of compact to very dense, brown to grey silty sand with gravel, cobbles, and boulders. Specifically, boulders were cored in the glacial till deposit at BH 2-21, BH 3-21, BH 5-21, BH 6-21, BH 7-21, and BH 8-21 in order to penetrate the glacial till deposit.

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

Bedrock

Bedrock was encountered underlying the glacial till deposit at approximate geodetic elevations ranging from 108.1 m at the northwest end of the site to 92.7 m at the southeast end of the site. Based on the recovered bedrock cores, the bedrock consists of gneiss to granitic gneiss. Further, based on the RQD values of the recovered bedrock cores, the bedrock varies from poor to excellent in quality, generally increasing in quality with depth.

4.3 Groundwater

Groundwater levels were measured on April 9, 2021 within the installed piezometers. The measured groundwater levels are presented in Table 1 below:

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	105.54	2.28	103.26	April 9, 2021
BH 2-21	100.14	1.30	98.84	April 9, 2021
BH 3-21	99.64	1.86	97.78	April 9, 2021
BH 4-21	99.55	1.48	98.07	April 9, 2021
BH 5-21	103.19	0.73	102.46	April 9, 2021
BH 6-21	108.12	2.58	105.54	April 9, 2021
BH 7-21	103.20	2.45	100.75	April 9, 2021
BH 8-21	103.77	3.70	100.07	April 9, 2021

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

It should be noted that long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximate depths of 2 to 3 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed multi-storey building, from a geotechnical perspective. It is recommended that the proposed multi-storey building be founded on conventional spread footings placed on clean, surface sounded bedrock.

Where the bedrock surface is encountered below the proposed underside of footing elevation, it is recommended that lean concrete be placed from the clean, surface sounded bedrock to the underside of footing elevation for support of the conventional spread footings.

Bedrock removal will be required to complete construction of the proposed building. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures.

Due to the depth of the bedrock at the subject site and the anticipated founding level for the proposed building, it is expected that all existing overburden material will be excavated from within the footprint of the proposed building.

Bedrock Removal

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

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

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 into 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. Vibrations, whether they are caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby 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.

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

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

Lean Concrete Filled Trenches

Where the proposed footings are to be founded on bedrock which is located below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. 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. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock, or on lean concrete which is placed directly over clean, surface sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa**, incorporating a geotechnical resistance factor of 0.5.

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.

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.

A factored bearing resistance value at ULS of **4,500 kPa** can be used for footings founded on bedrock at the proposed founding elevation of the underground parking levels provided the bedrock is free of seams, fractures and voids within 1.5 m below the founding level.

This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint(s) of the footing(s). At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant.

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

Settlement

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 settlement.

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 building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2.

Field Program

The seismic array testing location was placed on the southern end of the site in an approximate east-west direction as presented in Drawing PG5758-1, attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

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

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 2, 3 and 15 m away from the first and last geophones, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct 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 proposed building foundations. 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 shear wave velocity due to the increasing quality of bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

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

$$V_{S30} = \frac{Depth_{ofinterest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{S_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{S_{Layer2}}(m/s)}\right)}$$

$$V_{S30} = \frac{30m}{\left(\frac{30m}{2{,}540m/s}\right)}$$

$$V_{S30} = 2{,}540m/s$$

Based on the results of the seismic shear wave velocity testing, the average shear wave velocity, V_{s30}, for foundations placed on bedrock is **2,540 m/s**. Therefore, for the anticipated underside of footing elevation, a **Site Class A** is applicable for the design of the proposed building as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject sire are not susceptible to liquefaction.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. It is expected that the basement area will be mostly parking and the recommended pavement structure noted in Section 5.8 will be applicable.

However, if storage or other uses of the lower level where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of the geotechnical investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone layer under the lowest level floor slab.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed 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/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face.

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

Static 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 material
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g 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 $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Rock Anchor Design

Overview of Anchor Features

The geotechnical design of grouted rock anchors bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60-to-90-degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchor should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Generally, the unconfined compressive strength of granitic gneiss ranges between 50 and 100 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of 1.0 MPa, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing subsoils information, a Rock Mass Rating (RMR) of 65 was assigned to the bedrock, and Hoek and Brown parameters (m and s) were taken as 2.052 and 0.00293, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented in Table 2. Load specified rock anchor lengths can be provided, if required.

For our calculations, the following parameters were used:

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Granitic Gneiss Hoek and Brown parameters	65 m=2.052 and s=0.00293
Unconfined compressive strength – Granitic Gneiss	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75- and 125-mm diameter hole are provided in Table 3.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.1	1.6	2.7	250
	2.2	2.0	4.2	500
	4.3	2.2	6.5	1000
125	1.1	2.2	3.3	400
	1.5	2.4	3.9	600
	2.6	2.8	5.4	1000

Other considerations

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lowest level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below. The flexible pavement structure presented in Table 5 should be used for the proposed access road located at the east of the subject site, and heavy loading parking areas overlying the podium deck.

Table 4 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 5 - Recommended Asphalt Pavement Structure - Access Road and Heavy Loading Parking Areas	
Thickness (mm)	**Material Description**
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the Concrete Podium Deck.	

Minimum Performance Graded (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 excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

It is recommended that the portion of the proposed building foundation walls located below the long-term groundwater table (approximate geodetic elevation 97 m) be placed against a groundwater infiltration control system which is fastened to the temporary shoring system or vertical bedrock face. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which comes in contact with the proposed building's foundation walls.

For the portion of the groundwater infiltration control system installed against the vertical bedrock face, the following is recommended:

❑ Line drill the excavation perimeter.

❑ Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface, as required based on site inspection by Paterson.

❑ Place a suitable membrane against the prepared bedrock surface, such as Tremco Paraseal, or equivalent. The membrane liner should extend from geodetic elevation 97 m down to the 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.

Sub-slab Drainage

Subfloor drainage is recommended to control water infiltration below the lowest underground parking level slab. For design purposes, it is recommended that a 150 mm diameter perforated pipe be placed in at approximate 6 m spacing underlying the lowest level floor slab. The final 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

Where sufficient space is available for conventional backfilling, 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 are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection. These should be provided with a minimum 2.1 m thick soil cover and foundation insulation.

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

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.

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.

Bedrock stabilization

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. 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 or to provide a stable base for the overburden shoring system.

Horizontal rock anchors and chainlink fencing may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface.

A permanent vertical bedrock face is also proposed as part of the proposed site development. The permanently exposed vertical bedrock should be grinded. Paterson will then make a review to determine if localized bedrock stabilization measures are required. This is discussed further below in Section 6.8.

Temporary Shoring

Temporary shoring may be required to support the overburden soil 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 approval of these temporary systems will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor.

It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. The designer should also 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.

For design purposes, the temporary shoring system may consist of a 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 added to the earth pressures described below. These systems can 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. It is further recommended that the toe of the shoring be adequately supported to resist toe failure by means of rock bolts or extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is used.

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

Table 6 – 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 used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used 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 when placed on soil subgrade. 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 (concrete or PSM PVC pipes) or sand (concrete pipe). 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.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finish grade) should match the soils exposed at the trench walls to reduce the potential differential frost having. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater infiltration

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.

The rate of flow of groundwater into the excavation through the overburden and bedrock should be moderate for the expected subsurface conditions at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

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.

Adverse Effects of Dewatering on Adjacent Structures

Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Given the distance to neighbouring buildings and the minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures in the vicinity of 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 should be carried in a manner to avoid the introduction of frozen materials, snow, or ice into the trenches. Precautions must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, 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 soils. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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 moderate to slightly aggressive corrosive environment.

6.8 Slope Stability Assessment

It is proposed that the north and west boundaries of the site will have an approximate 4 to 8 m vertical bedrock face between the finished site grades, and the adjacent grades at Bill Teron Park. The proposed proposed grade differential and bedrock were reviewed by Paterson. Two (2) slope cross-sections were studied as the worst-case scenarios for the slope stability analysis. The cross-section locations are presented on Drawing PG5758-1 – Test Hole Location Plan in Appendix 2.

Slope Stability Analysis

The analysis of the stability of the proposed vertical bedrock 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.

Subsurface conditions at the cross-sections were inferred based on nearby boreholes. Proposed grades were modelled based on the available Grading Plan, prepared by others. The groundwater table throughout the subject was based on a combination of our piezometer readings and general knowledge of the area’s geology.

Static Loading Analysis

The results of the slope stability analysis for static conditions at Sections A and B are presented in Figures 4A and 5A in Appendix 2. The results indicate factors of safety exceeding 1.5 beyond at both analyzed section.

Seismic Loading Analysis

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

The results of the analyses including seismic loading are shown in Figures 4B and 5B in Appendix 2. The results indicate factors of safety exceeding 1.1 at both analyzed sections.

Based on these results, 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

Based on the subsurface profiles encountered at the borehole locations, shallow bedrock is present in the vicinity of the northern and western site boundaries. Further, given that no watercourse is present near the toe of the proposed bedrock face, toe erosion and erosion access allowances are not required at the subject site.

Limit of Hazard Lands

Based on the slope stability analyses, no hazard lands were identified along the top or bottom of the proposed vertical bedrock face.

However, once the vertical bedrock face is exposed, a varying safety setback will be established from the bottom of the vertical bedrock face. Reference should be made to Drawing PG5758-2 – Rock Safety Setbacks in Appendix 2, which illustrates the rock safety setbacks from the vertical bedrock face, and which vary with the varying height of the proposed vertical bedrock face.

The rock safety setbacks will be demarcated on-site using a fence, and signage will be posted on the fence indicating that no persons are to enter the space between the fence and the vertical bedrock face, or attempt to climb the vertical bedrock face.

A fence will also be built on the top of the vertical bedrock face, inset approximately 150 mm from the site boundaries.

7.0 Recommendations

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

- Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- Review proposed waterproofing and foundation drainage design and requirements.
- Observe and approve the installation of the water suppression system.
- Review the bedrock stabilization and excavation requirements.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- 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 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 Kanata Woods 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.

LICENSED PROFESSIONAL ENGINEER
Dec 1, 2021
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Fernanda Carozzi, PhD. Geoph.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❏ Kanata Woods Inc. (email copy)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building
Kanata Ave. and Canadian Shield Ave., Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 29, 2021

FILE NO. PG5758

HOLE NO. BH 1-21

SOIL DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.54
TOPSOIL	0.25	SS	1	50	50+		
GLACIAL TILL: Brown silty sand with clay, some gravel	0.51	RC	1	100	47	1	104.54
BEDROCK: Poor to excellent quality, reddish grey gneiss to granitic gneiss		RC	2	100	89	2	103.54
		RC	3	100	88	3	102.54
		RC	4	100	100	4	101.54
		RC	5	100	94	5	100.54
		RC	6	100	95	6	99.54
		RC	7	100	98	7	98.54
		RC	8	100	100	8	97.54
						9	96.54
						10	95.54
	11.61					11	94.54

End of Borehole

(GWL @ 1.19m - April 9, 2021)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building
Kanata Ave. and Canadian Shield Ave., Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5758

REMARKS

HOLE NO. BH 2-21

BORINGS BY Track-Mount Power Auger

DATE March 30, 2021

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.14
TOPSOIL	0.56	SS	1	50	W		
Compact, brown **SILTY SAND,** trace gravel	1.37	SS	2	100	18	1	99.14
GLACIAL TILL: Very dense, brown silty sand to sandy silt with gravel, cobbles and boulders Auger refusal at 2.9m depth. Cored through boulders from 2.9 to 4.5m depth.		SS	3	75	74	2	98.14
		SS	4	42	66	3	97.14
		RC	1	24		4	96.14
		SS	5	100	50	5	95.14
		SS	6	67	50+	6	94.14
		SS	7	54	66	7	93.14
	7.44	SS	8	50	50+		
BEDROCK: Fair quality, dark grey to red gneiss to granitic gneiss - vertical seam from 8.4 to 8.8m depth		RC	2	100	53	8	92.14
						9	91.14
		RC	3	100	75	10	90.14
End of Borehole (GWL @ 0.33m - April 9, 2021)	10.49						

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building
Kanata Ave. and Canadian Shield Ave., Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

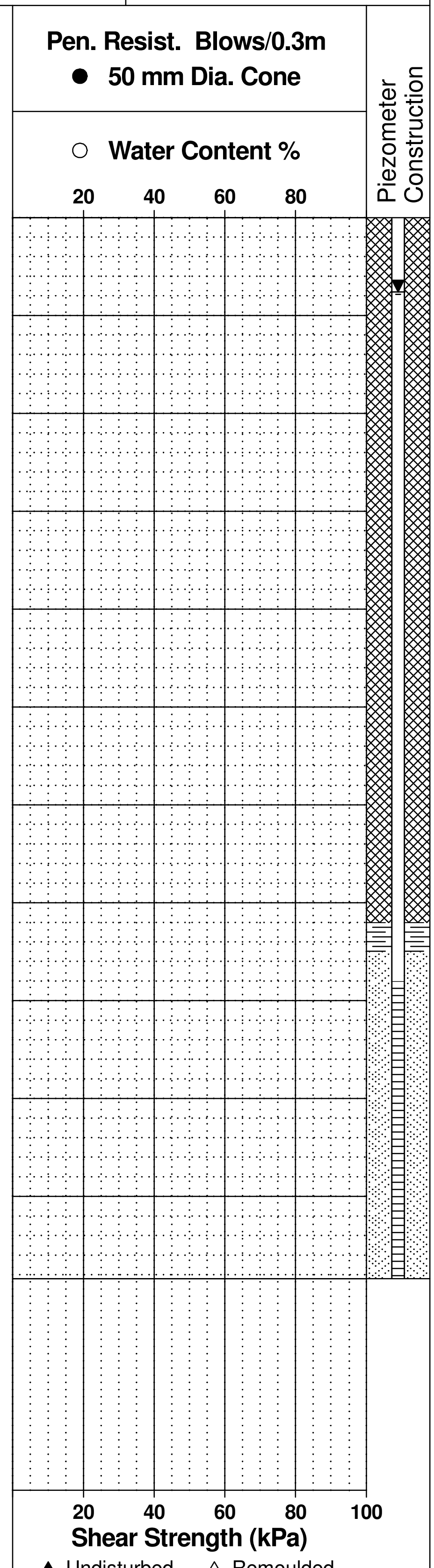
DATE March 31, 2021

FILE NO. PG5758

HOLE NO. BH 3-21

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20	AU	1		
Compact, brown **SILTY SAND** with clay and organics	0.60	AU	2		
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders, trace clay Auger refusal at 2.3m depth. Cored through boulders from 2.3 to 4.0m depth.	4.57	SS	3	42	26
		SS	4	100	50+
		SS	5	0	50+
		RC	1	83	
		RC	2	69	
BEDROCK: Poor quality gneiss to granitic gneiss - 50mm thick silty sand seam at 4.9m depth - mud seams from 5.1 to 5.5m, 6.4 to 6.55m, 7.0 to 7.1m, 9.9 to 10.3m and 10.55 to 10.6m depths	10.84	RC	3	100	0
		RC	4	100	30
		RC	5	100	40
		RC	6	100	52
		RC	7	100	40
		RC	8	100	0
End of Borehole (GWL @ 0.76m - April 9, 2021)					

DEPTH (m)	ELEV. (m)
0	99.64
1	98.64
2	97.64
3	96.64
4	95.64
5	94.64
6	93.64
7	92.64
8	91.64
9	90.64
10	89.64



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building
Kanata Ave. and Canadian Shield Ave., Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 31, 2021

FILE NO. PG5758

HOLE NO. BH 4-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20	AU	1		
Brown **SILTY CLAY,** trace sand and organics	0.60	AU	2		
Compact, brown **SILTY SAND** with gravel, trace clay		SS	3	50	21
	2.13	SS	4	42	20
Dense, brown **SILTY SAND to SANDY SILT,** some clay	2.59	SS	5	100	34
GLACIAL TILL: Compact to very dense, grey silty sand to sandy silt with gravel, cobbles and boulders, trace clay		SS	6	67	22
		SS	7	83	52
	4.65	SS	8	100	50+
BEDROCK: Fair quality, grey gneiss to granitic gneiss		RC	1	100	40
		RC	2	100	76
		RC	3	100	73
		RC	4	100	52
	10.87	RC	5	100	58
End of Borehole					
(GWL @ 0.43m - April 9, 2021)					

DEPTH (m)	ELEV. (m)
0	99.55
1	98.55
2	97.55
3	96.55
4	95.55
5	94.55
6	93.55
7	92.55
8	91.55
9	90.55
10	89.55

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building
Kanata Ave. and Canadian Shield Ave., Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 1, 2021

FILE NO. PG5758

HOLE NO. BH 5-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.19
TOPSOIL 0.20		AU	1				
Brown **SILTY CLAY** with silty sand 1.07		AU	2				
		SS	2	71	17	1	102.19
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace clay Auger refusal at 2.3m depth. Cored through boulders from 2.3 to 5.7m depth.		SS	3	62	42	2	101.19
		RC	1	94		3	100.19
		RC	2	82		4	99.19
5.79		RC	3	70		5	98.19
BEDROCK: Fair to good quality, grey gneiss to granitic gneiss		RC	4	80	60	6	97.19
		RC	5	88	80	7	96.19
						8	95.19
		RC	6	100	87	9	94.19
10.39						10	93.19
End of Borehole							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building
Kanata Ave. and Canadian Shield Ave., Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 5, 2021

FILE NO. PG5758

HOLE NO. BH 6-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	108.12
TOPSOIL	0.20	AU	1				
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders	0.97	RC	1	82		1	107.12
BEDROCK: Poor to good quality, grey to red gneiss to granitic gneiss		RC	2	100	36	2	106.12
						3	105.12
	4.14	RC	3	100	83	4	104.12
End of Borehole							
(GWL @ 1.12m - April 9, 2021)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building
Kanata Ave. and Canadian Shield Ave., Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 6, 2021

FILE NO. PG5758

HOLE NO. BH 7-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20	AU	1		
Brown **SILTY CLAY** with organics, trace silty sand	0.91	AU	2		
Very loose to dense, brown **SILTY SAND,** trace gravel	2.74	SS	3	4	2
		SS	4	33	3
		SS	5		50+
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders Cored through cobbles and boulders from 2.7 to 4.7m depth.	4.75	RC	1	33	
		RC	2	30	
BEDROCK: Fair to excellent quality, reddish grey gneiss - vertical seam from 6.5 to 6.9m depth	7.80	RC	3	94	94
		RC	4	100	52
		RC	5	100	83
End of Borehole (GWL @ 1.50m - April 9, 2021)					

DEPTH (m)	ELEV. (m)
0	103.20
1	102.20
2	101.20
3	100.20
4	99.20
5	98.20
6	97.20
7	96.20

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building
Kanata Ave. and Canadian Shield Ave., Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5758

REMARKS

HOLE NO. BH 8-21

BORINGS BY Track-Mount Power Auger **DATE** April 6, 2021

SOIL DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.77
TOPSOIL	0.20	AU	1				
Brown **SILTY CLAY** with organics and silty sand	0.76	AU	2				
Loose to dense, brown **SILTY SAND,** some gravel, trace clay		SS	3		6	1	102.77
	1.98	SS	4		50+	2	101.77
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders	2.39	RC	1	72			
BEDROCK: Good to excellent quality, reddish grey gneiss to granitic gneiss		RC	2	100	90	3	100.77
		RC	3	98	98	4	99.77
						5	98.77
		RC	4	100	98	6	97.77
	7.44					7	96.77
End of Borehole							

(GWL @ 1.89m - April 9, 2021)

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

Piezometer Construction

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

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

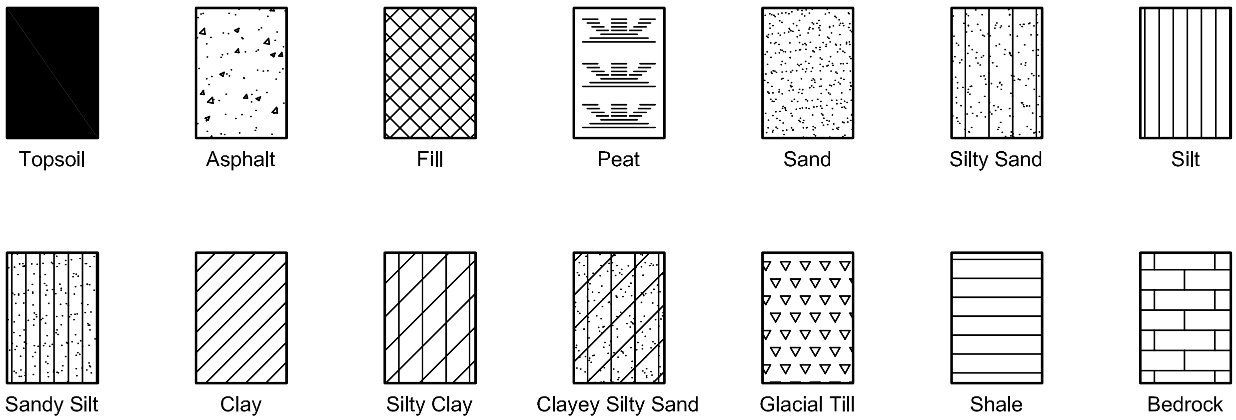
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

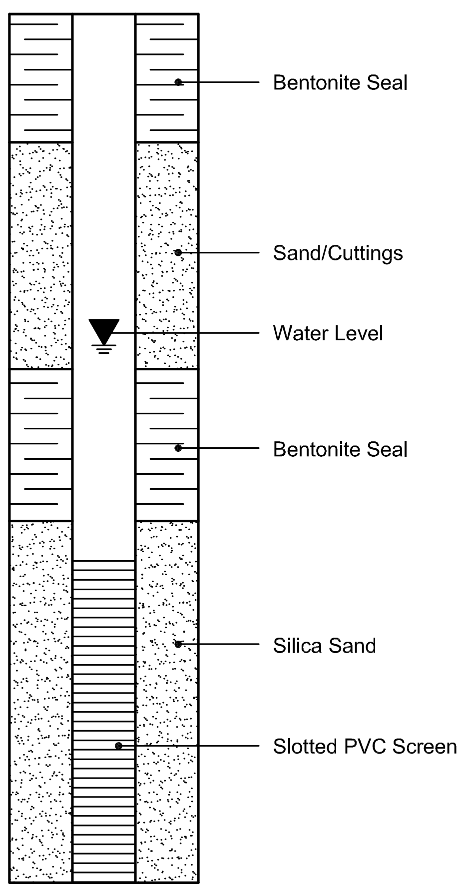
SYMBOLS AND TERMS (continued)

STRATA PLOT

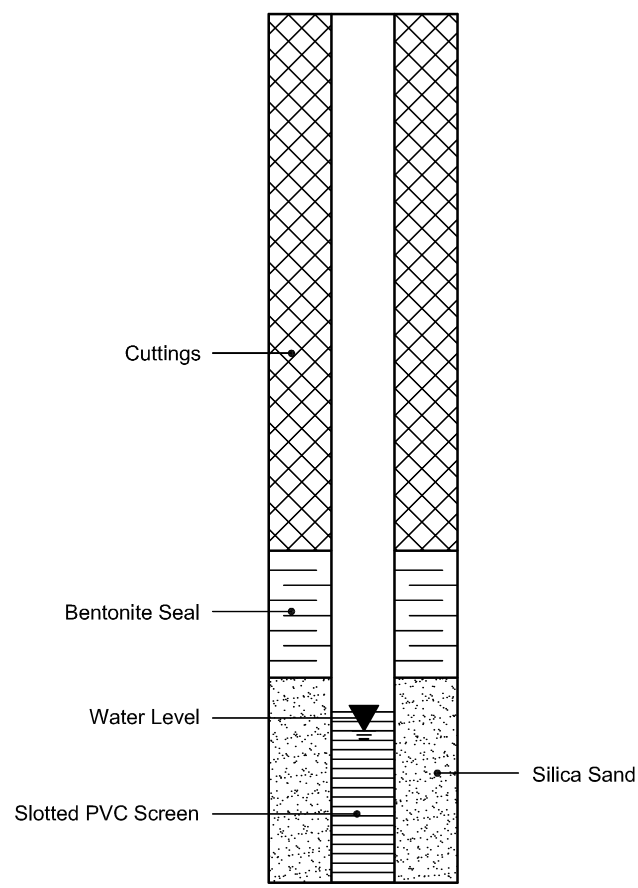


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29750

Report Date: 07-Apr-2021

Order Date: 1-Apr-2021

Project Description: PG5758

	Client ID:	BH4-21 SS5	-	-	-
	Sample Date:	31-Mar-21 09:00	-	-	-
	Sample ID:	2114513-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	92.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.75	-	-	-
Resistivity	0.10 Ohm.m	94.6	-	-	-
Anions					
Chloride	5 ug/g dry	8	-	-	-
Sulphate	5 ug/g dry	7	-	-	-

APPENDIX 2

FIGURE 1

KEY PLAN

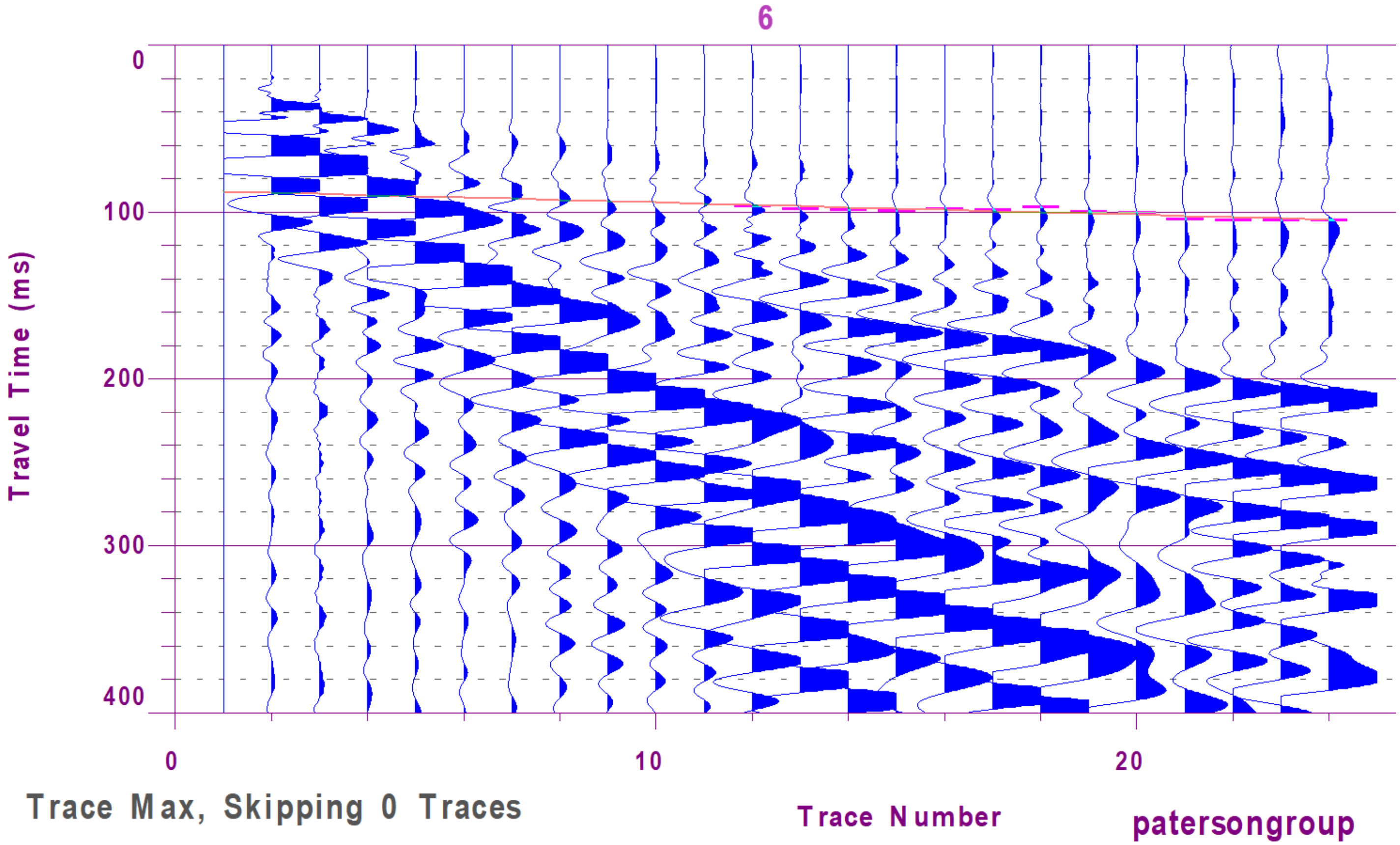


Figure 2 – Shear Wave Velocity Profile at Shot Location -3 m

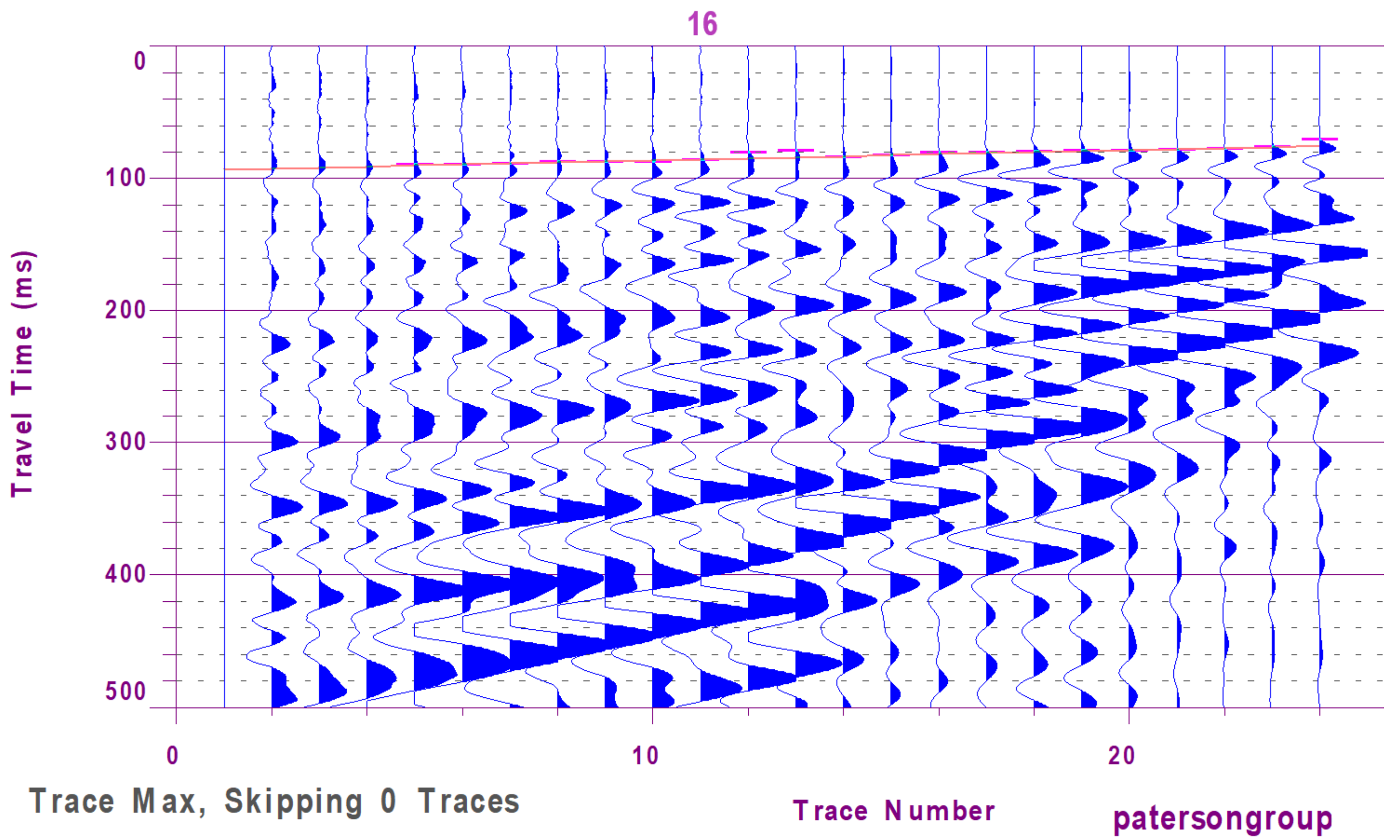


Figure 3 – Shear Wave Velocity Profile at Shot Location +61 m

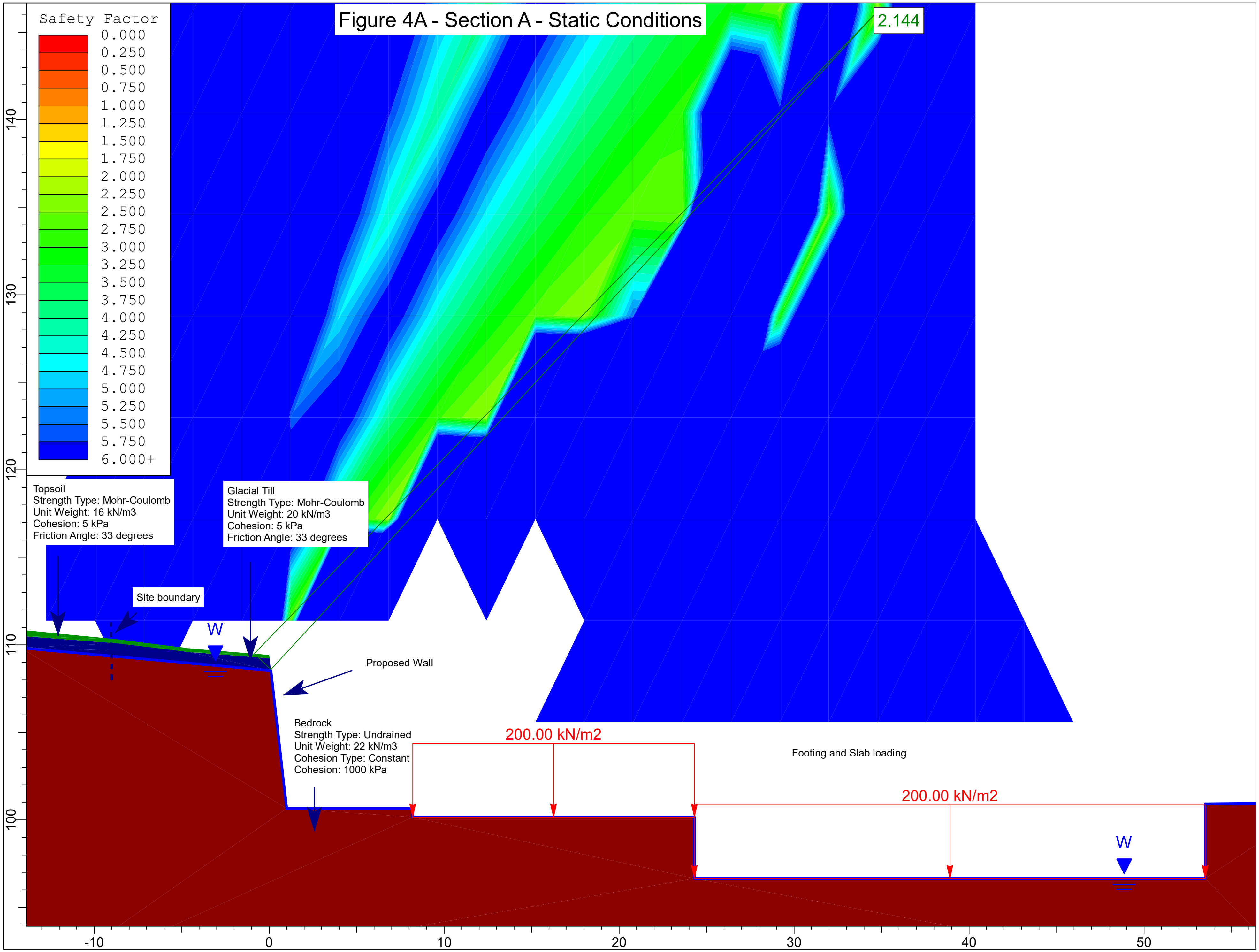


Figure 4A - Section A - Static Conditions
2.144
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Topsoil
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Glacial Till
Strength Type: Mohr-Coulomb
Unit Weight: 20 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Site boundary
W
Proposed Wall
Bedrock
Strength Type: Undrained
Unit Weight: 22 kN/m3
Cohesion Type: Constant
Cohesion: 1000 kPa
200.00 kN/m2
Footing and Slab loading
200.00 kN/m2
W
140
130
120
110
100
-10
0
10
20
30
40
50

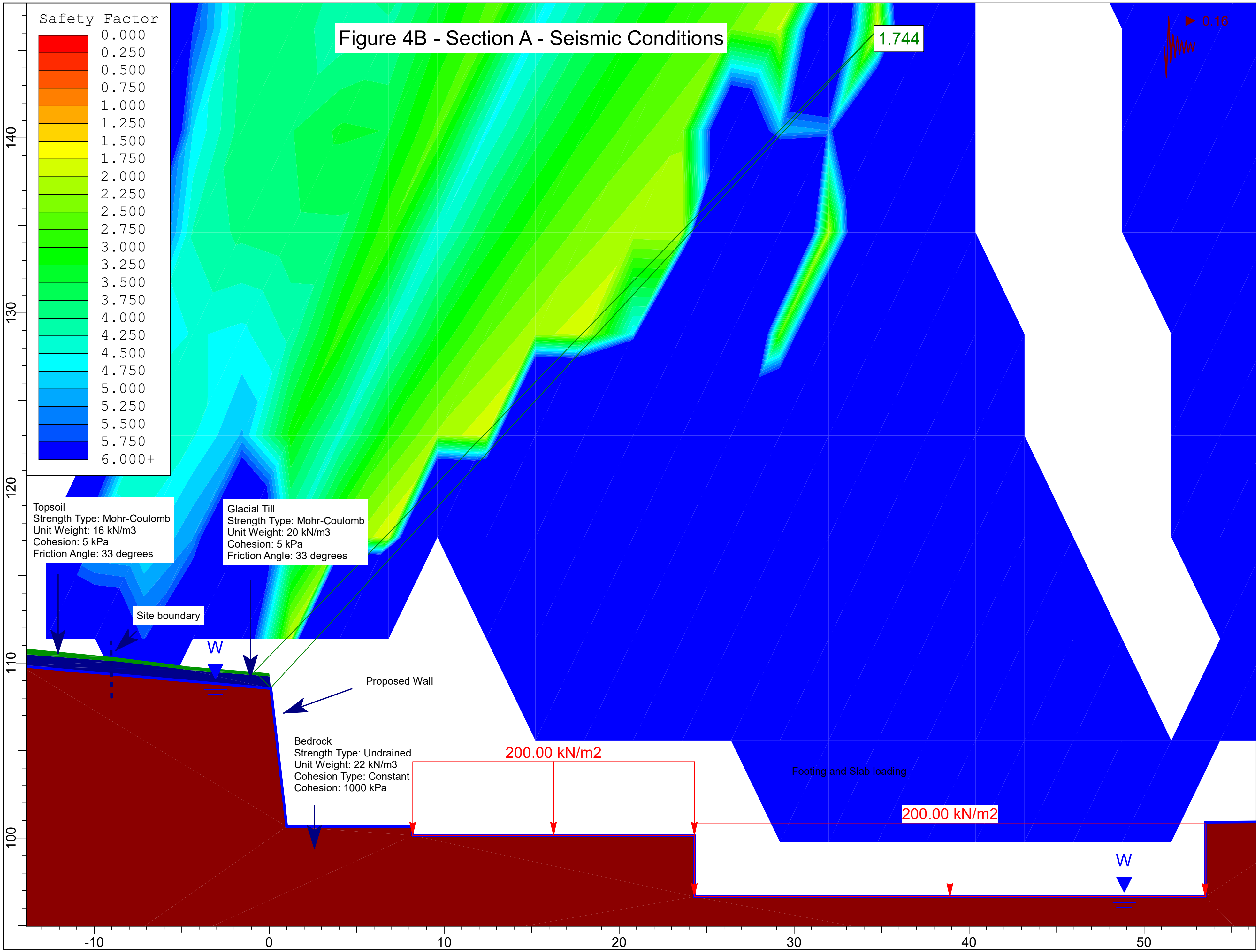


Figure 4B - Section A - Seismic Conditions
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.744
0.16
Topsoil
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Glacial Till
Strength Type: Mohr-Coulomb
Unit Weight: 20 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Site boundary
W
Proposed Wall
Bedrock
Strength Type: Undrained
Unit Weight: 22 kN/m3
Cohesion Type: Constant
Cohesion: 1000 kPa
200.00 kN/m2
Footing and Slab loading
200.00 kN/m2
W
140
130
120
110
100
-10
0
10
20
30
40
50

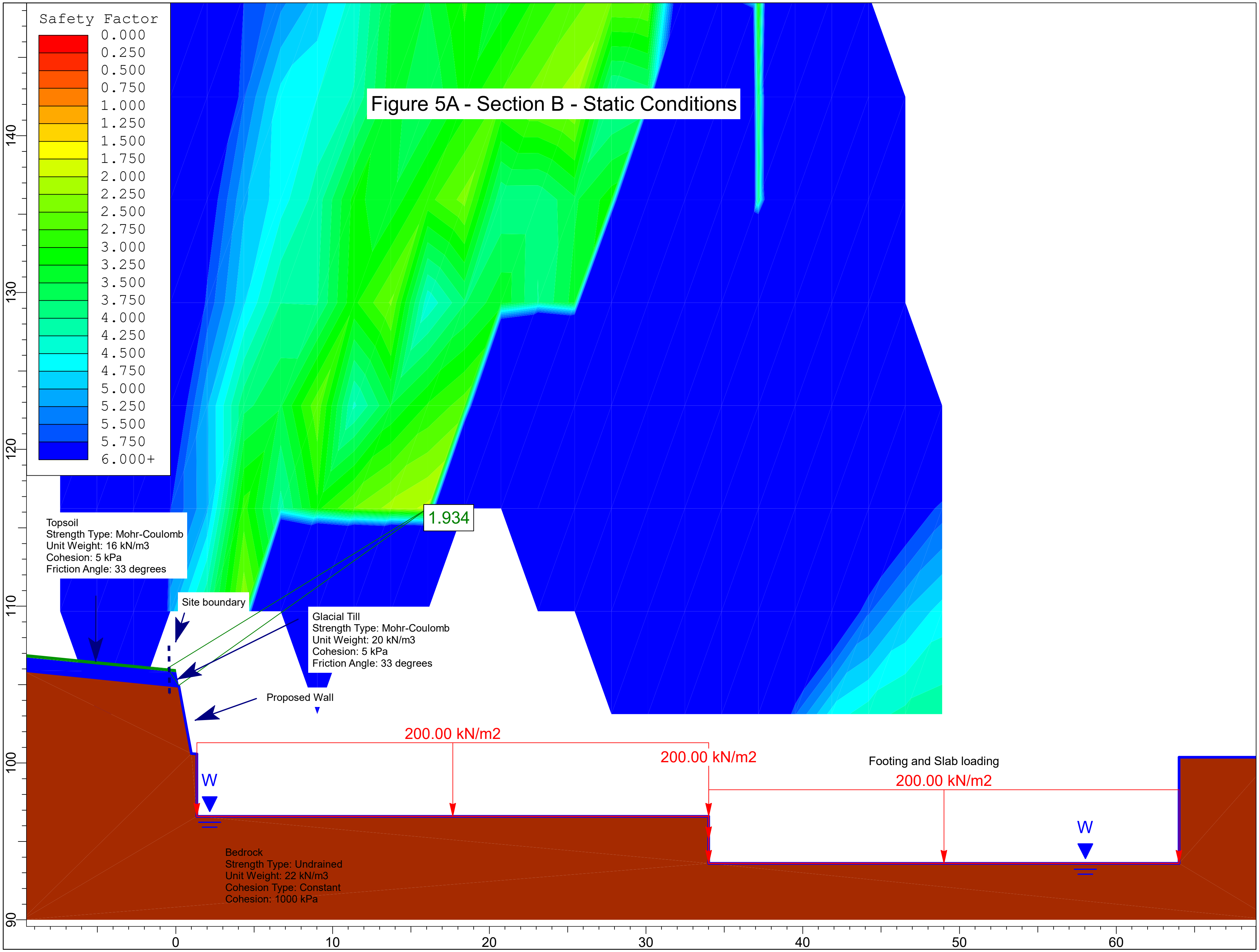


Figure 5A - Section B - Static Conditions
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.934
Topsoil
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Site boundary
Glacial Till
Strength Type: Mohr-Coulomb
Unit Weight: 20 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Proposed Wall
200.00 kN/m2
200.00 kN/m2
Footing and Slab loading
200.00 kN/m2
W
W
Bedrock
Strength Type: Undrained
Unit Weight: 22 kN/m3
Cohesion Type: Constant
Cohesion: 1000 kPa
140
130
120
110
100
90
0
10
20
30
40
50
60

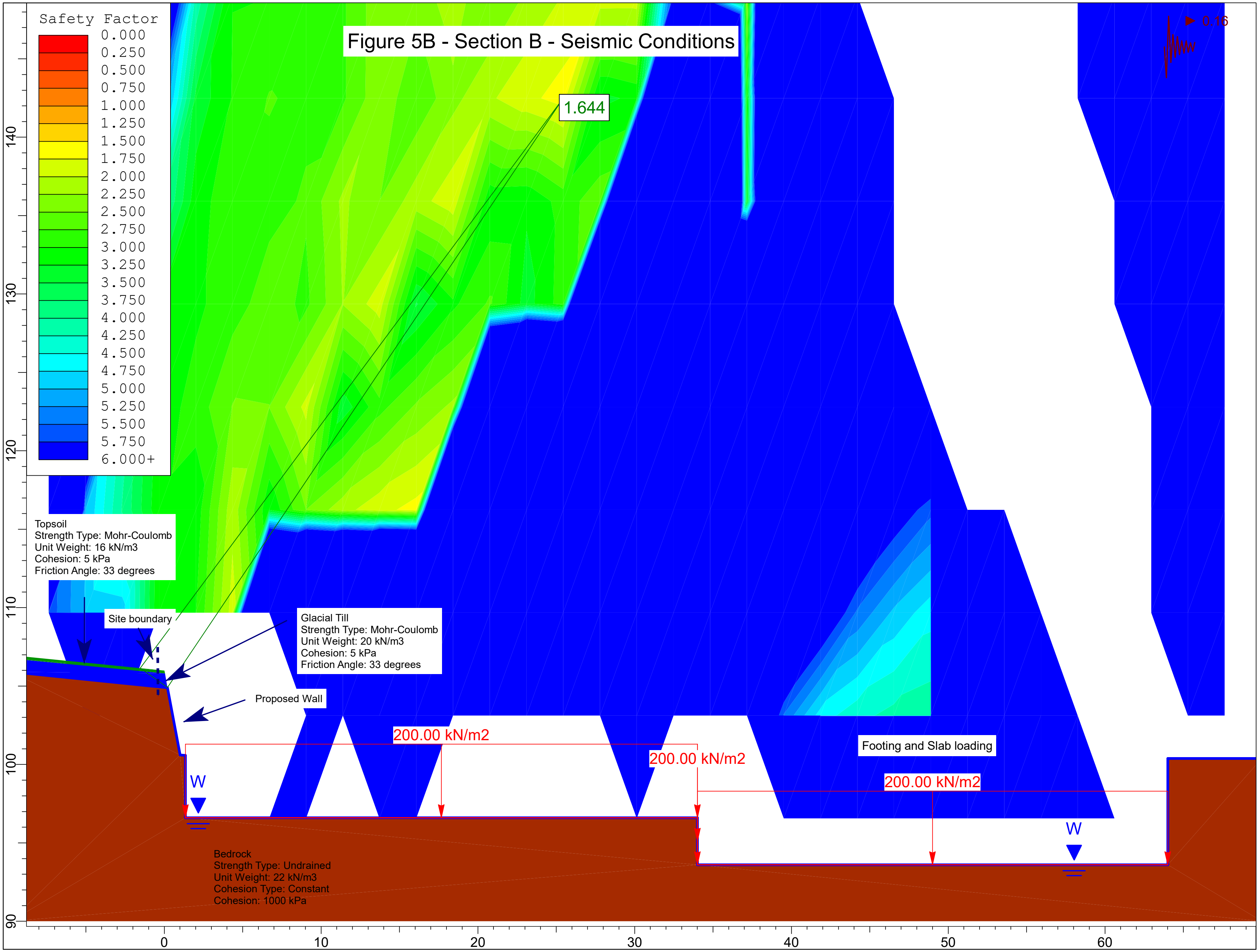
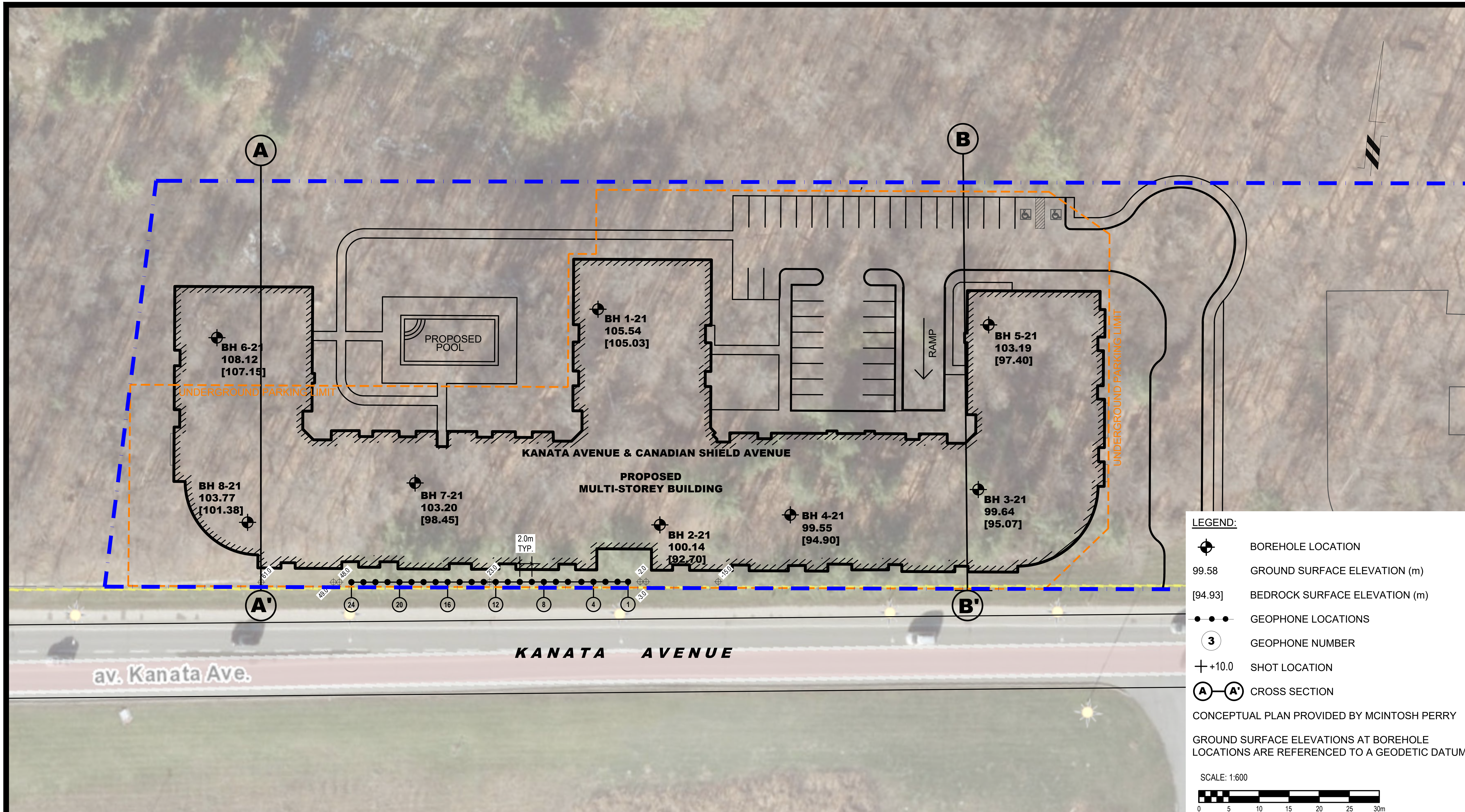


Figure 5B - Section B - Seismic Conditions
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.644
0.16
Topsoil
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Site boundary
Glacial Till
Strength Type: Mohr-Coulomb
Unit Weight: 20 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Proposed Wall
200.00 kN/m2
200.00 kN/m2
200.00 kN/m2
Footing and Slab loading
W
W
Bedrock
Strength Type: Undrained
Unit Weight: 22 kN/m3
Cohesion Type: Constant
Cohesion: 1000 kPa
90
100
110
120
130
140
0
10
20
30
40
50
60



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
1	UPDATED LATEST CONCEPTUAL PLAN TO DRAWING	12/01/2021	FC

KANATA WOODS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY RESIDENTIAL BUILDING
KANATA AVENUE AND CANADIAN SHIELD AVENUE

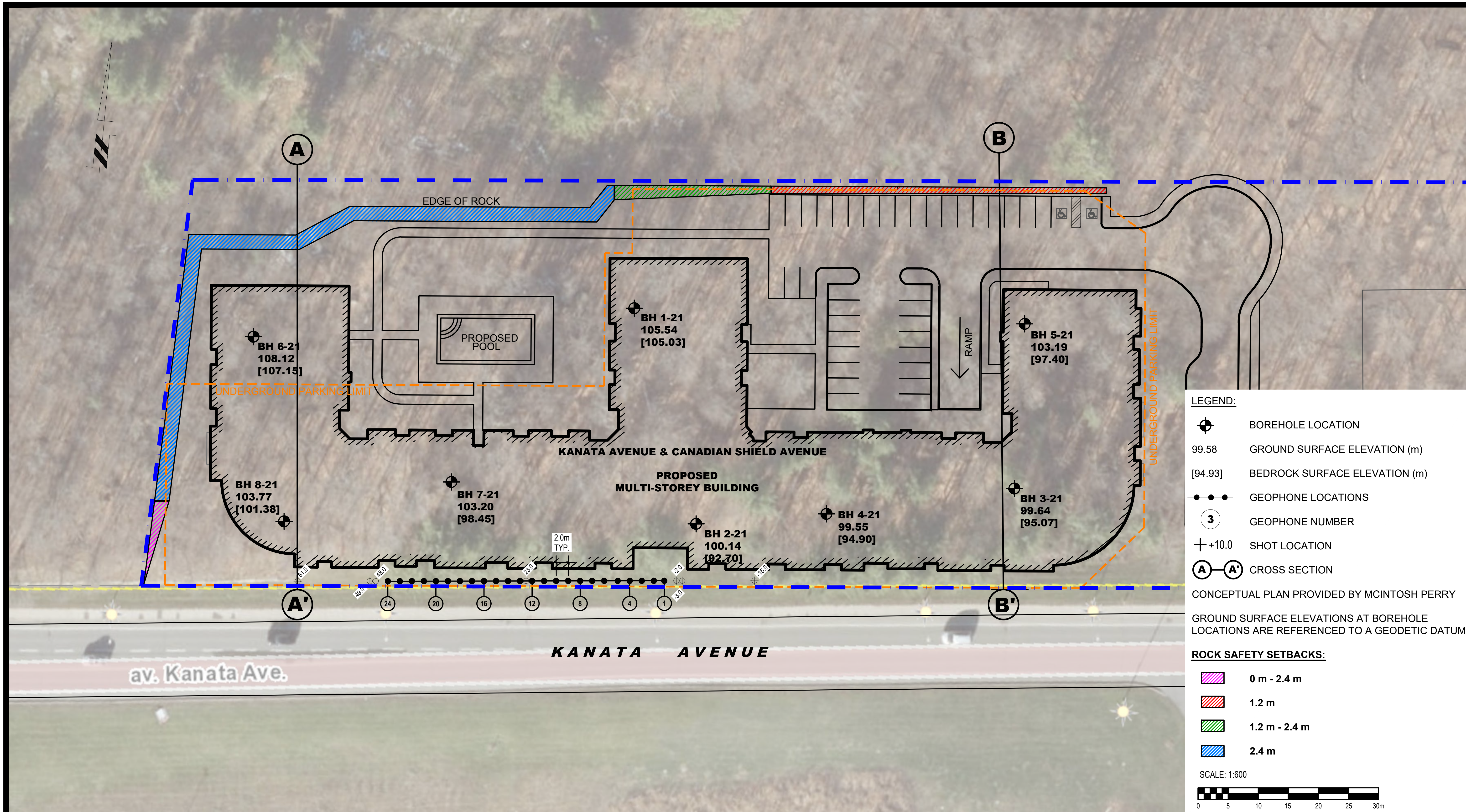
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600	Date: 11/2021
Drawn by: NFRV	Report No.: PG5758-1
Checked by: SD	Dwg. No.: **PG5758-1**
Approved by: DJG	Revision No.: 1

p:\autocad drawings\geotechnical\pg57xx\pg5758\pg5758-1 thlp (rs).dwg



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

KANATA WOODS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY RESIDENTIAL BUILDING
KANATA AVENUE AND CANADIAN SHIELD AVENUE

OTTAWA, ONTARIO

Title:

ROCK SAFETY SETBACKS

Scale: 1:600	Date: 11/2021
Drawn by: NFRV	Report No.: PG5758-MEMO.01
Checked by: SD	Dwg. No.: PG5758-2
Approved by: DJG	Revision No.:

p:\autocad drawings\geotechnical\pg57xx\pg5758\pg5758-1 thlp (rss).dwg