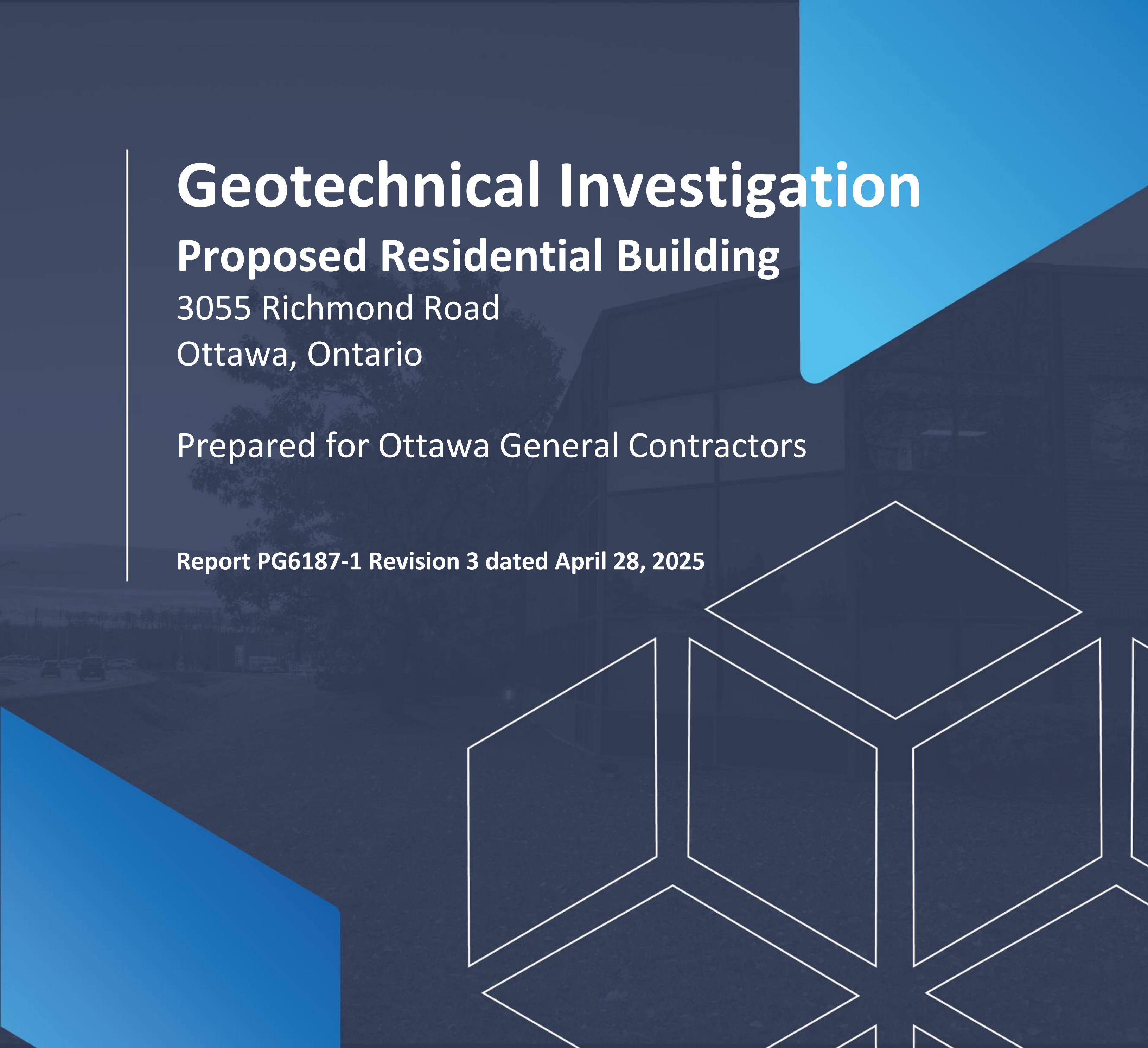

Geotechnical Investigation

Proposed Residential Building

3055 Richmond Road
Ottawa, Ontario

Prepared for Ottawa General Contractors

Report PG6187-1 Revision 3 dated April 28, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ottawa General Contractors to conduct a geotechnical investigation for the proposed residential building to be located at 3055 Richmond Road, 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 test holes.

☐ 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 the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

It is understood that the proposed development will consist of a mid-rise apartment building with one level of underground parking. Access lanes, walkways, and landscaped areas are also planned as part of the development. It is further anticipated that the proposed building(s) 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 on May 6, 2022, and consisted of advancing a total of 3 boreholes to a maximum depth of 5.9 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 test hole locations are shown on Drawing PG6187-1 – Test Hole Location Plan included in Appendix 2.

A supplemental investigation was carried out on July 13, 2023. At that time, a total of 2 test pits and one hand auger were advanced, to a maximum depth of 2.21 m below the existing ground surface, for the purpose of site-specific, in-situ permeameter testing.

The boreholes were completed using a low-clearance, track-mounted drill rig operated by a two-person crew. The test pits were excavated using a rubber-tired backhoe excavator. The hand augers were advanced using a stainless-steel hand auger. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The testing 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 collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler or from the drill auger flights. Test pit samples were collected at selected intervals from the test pit sidewalls. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the drill auger, split-spoon and grab samples were recovered from the boreholes and test pits are shown as AU, SS and G, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A 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.

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

The thickness of the overburden was evaluated by a dynamic cone penetration test (DCPT) at borehole BH 3-22. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Flexible polyethylene standpipes were installed in all boreholes to permit monitoring of the groundwater levels following the completion of the sampling program.

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 PG6187-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

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

3.5 Permeameter Testing

In-situ permeameter testing was conducted at two (2) locations using a Pask (Constant Head Well) Permeameter to confirm infiltration rates of the surficial soils at the subject site on July 13, 2023. Two test pits were excavated to a maximum depth of 2.2 m below existing ground surface (begs). Each test pit was excavated in approximately 0.5 m increments to allow for safe entry into the pits as well as permeameter testing to be conducted at different elevations.

Permeameter tests were conducted at approximately 0.4 to 0.7 m begs and 1.9 to 2.2 m begs within each test pit. At approximately 0.3 m above the desired testing elevation, an 83 mm diameter hole was excavated using a Riverside/Bucket auger to the desired testing depth. All soil from the auger flights was visually inspected and initially classified on site. An aggregated soil sample was gathered at each test location. The test was conducted by filling the permeameter reservoir water and inverting it into the hole, ensuring it was relatively vertical and rests on the bottom of the hole.

The water level of the reservoir was monitored at 0.5 to 5 minute intervals until the rate of fall out of the permeameter reached equilibrium, known as quasi “steady state” flow rate. Quasi steady state flow can be considered to have been obtained after measuring 3 to 5 consecutive rate of fall readings with identical values. The values for the steady state rate of fall were recorded for each location. The steady state rate of fall was converted to a field saturated hydraulic conductivity value (K_{fs}) using the Engineering Technology Canada Ltd. conversion tables. Unfactored infiltration rates were estimated based on the methodology outlined in Appendix C of the Credit Valley Conservation’s Low Impact Development Stormwater Management Planning and Design Guide.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a single-family residential dwelling with landscaped areas, fences and an asphalt paved driveway. Mature trees are present within the southern portion and along the western boundary of the subject site.

The site is bordered by residential dwellings to the north and west, and by Richmond Road to the east and south. The ground surface in front of the existing dwelling is relatively flat and at grade with the Richmond Road with an approximate geodetic elevation of 75.5 m. The ground surface slopes downward toward the rear and southwest side yard of the property to an approximate geodetic elevation of 74 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil underlain by fill consisting of brown silty sand to sandy silt with trace to some clay and gravel and trace topsoil and brick. Compact, brown silty sand was encountered underlying the fill within borehole BH 1-22 and test pit TP 1-23. A deposit of very stiff, brown silty clay was encountered underlying the fill or compact, brown silty sand to sandy silt within boreholes BH 1-22 and BH 3-33.

Glacial till was encountered underlying the above noted layers at all test holes and consisted of very dense, grey silty sand, sandy silt or silty clay with gravel, cobbles and boulders. Practical refusal to DCPT was encountered in borehole BH 3-21 at a depth of 9.1 m below the existing ground surface.

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

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic shale of the Rockcliffe formation, with an estimated overburden drift thickness of 5 to 10 m depth.

4.3 Groundwater

Groundwater levels were measured within the polytube piezometers installed in the boreholes on May 6, 2022. Furthermore, observations of groundwater infiltration into the open holes were recorded during the supplemental investigation on July 13, 2023. The measured groundwater levels and infiltration rates are presented in Table 1 below.

Table 1 – Measured Groundwater Levels

Test Hole Number	**Method**	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Date**
			Depth (m)	**Elevation (m)**	
BH 1-22	Piezometer	74.49	2.15	72.34	May 19, 2022
BH 2-22	Piezometer	73.90	5.84	68.06	May 19, 2022
BH 3-22	Piezometer	75.58	2.44	73.14	May 19, 2022
TP 1-23	Sidewall Infiltration	73.24	Dry	-	July 13 2023
TP 2-23	Sidewall Infiltration	73.54	Dry	-	July 13, 2023

NOTE: The ground surface elevations at the test hole location of the current investigation were surveyed by Paterson using a high precision GPS unit and was referenced to a geodetic datum.

It is important to note that surface water can be perched within the confines of the boreholes where impermeable soils are present. Further, the 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 an approximate depth of **3.5 to 4.5 m** below existing grade. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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

4.3 Permeameter Testing Results

In-situ permeameter testing was conducted at two (2) locations using a Pask (Constant Head Well) Permeameter to confirm infiltration rates of the surficial soils at the subject site on July 13, 2023. Preparation and testing of this investigation are in accordance with the Canadian Standards Association (CSA) B65-12-Annex E. Field saturated hydraulic conductivity (K_{fs}) values and estimated infiltration rates are presented in Table 2 on the following page. Field saturated hydraulic conductivity values were determined using the Engineering Technologies Canada (ETC) Ltd. conversion tables.

Estimated unfactored infiltration rates were estimated based on the methodology outlined in Appendix C of the Credit Valley Conservation's Low Impact Development Stormwater Management Planning and Design Guide.

Table 2 – Summary of field saturated hydraulic conductivity values and estimate infiltration rates.

Test Hole ID	Ground Surface Elevation (m)	Permeameter Testing Depth (m begs)	Permeameter Testing Elevation (m)	*K_{fs} (m/sec)	**Estimated Infiltration Rate (mm/hr)	***Soil Type
TP 1-23	73.24	0.4	72.84	$1.6x10^{-6}$	52	Fill
		1.9	71.34	$1.1x10^{-6}$	47	Glacial Till
TP 2-23	73.54	0.7	72.84	$7.5x10^{-6}$	79	Fill
		2.2	71.34	$\leq 8.1x10^{-9}$	≤13	Glacial Till

*Field Saturated Hydraulic Conductivity.
**The infiltrations rates do not include a safety correction factor. A safety correction should be applied to the estimated infiltration rates for design purposes.
***Soil descriptions are shown on the appended Soil Profile and Test Data sheets.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed development will be founded on conventional shallow footings placed on an undisturbed, compact silty sand to sandy silt, very stiff to stiff, brown silty clay and/or a dense, glacial till bearing medium.

Where glacial till is excavated, it is anticipated that cobbles and boulders will be encountered frequently. All contractors should be prepared for boulder removal throughout the subject site.

Due to the presence of a silty clay deposit, a permissible grade raise restriction is required for the subject site for all footings founded on a silty clay bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

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

It is anticipated that existing fill within the proposed building footprint, free of deleterious material and significant amounts of organics, and approved by the geotechnical consultant at the time of construction can be left in place below the proposed building footprints outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled by a vibratory roller making several passes under dry conditions and above freezing temperatures and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved engineered fill.

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

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, connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, stiff to very stiff, brown silty clay bearing medium can be designed using a bearing resistance value and serviceability limit states (SLS) of **150 kPa** and a bearing resistance value at ultimate limit states (USL) of **225 kPa**, incorporating a geotechnical factor of 0.5.

Footings placed on an undisturbed, compact silty sand can be designed using a bearing resistance factor at SLS of **120 kPa** and a bearing resistance value at ULS of **180 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings placed on an undisturbed, very dense glacial till can be designed using a bearing resistance factor at SLS of **200 kPa** and a bearing resistance value at ULS of **300 kPa**, incorporating a geotechnical resistance factor of 0.5.

Where the subgrade is found to be in a loose state of compactness below the proposed footings, the subgrade should be proof rolled using heavy vibratory compaction equipment making several passes and approved by Paterson prior to placing the footings. Any soft or poor performing areas should be removed and backfilled with OPSS Granular A crushed stone.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen, or disturbed soil, have been removed, in the dry, prior to the placement of concrete footings.

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 silty clay to sandy silt, silty sand and glacial till when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Permissible Grade Raise

Due to the presence of a silty clay deposit within the subject site, it is recommended that a permissible grade raise of **2.5 m** be provided for the areas where a silty clay deposit is present. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations. 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.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2024 for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native, undisturbed, compact silty sand to sandy silt, stiff to very stiff silty clay, or dense glacial till will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction.

Provisions should be made to proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any engineered fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings).

For underground parking structures, reference should be made to Table 5 in Subsection 5.7 for underground parking rigid pavement and subgrade construction.

For basement slabs which will not be used for underground parking (i.e. mechanical or storage rooms, etc.), it is recommended that the upper 200 mm of sub-slab fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose lifts and compacted to a minimum 99% of its SPMDD.

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/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.

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

Static Conditions

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

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

An additional pressure with a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above 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 Conditions

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

The seismic earth force (ΔP_{AE}) could be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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 2024. The vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions could be calculated using:

$P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions presented above.

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

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

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

5.7 Pavement Design

Car only parking areas, heavy traffic access areas, and underground parking areas are expected at this site. The proposed pavement structures are presented in Tables 3, 4 and 5 below and on the following page.

Table 3 – Recommended Flexible Pavement Structure – Car Only Parking Areas

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

Table 4 – Recommended Flexible Pavement Structure – Access Lanes and Heavy Truck Parking/Loading 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.	

Table 5 – Recommended Rigid Pavement Structure – Underground Parking Level

Thickness (mm)	Material Description
150	**Rigid Concrete Pavement** – 32 MPa concrete with air entrainment
300	**BASE –** OPSS Granular A Crushed Stone
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil.	

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 compacted to a minimum of 100% of the material’s SPMDD using suitable compaction equipment.

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.

5.8 Retaining Wall Design

It is understood that a retaining wall, greater than 1 m in height, is proposed along the northeast corner of the building, between a proposed at-grade parking area and the property line. The soil parameters presented in Tables 5 and 6 on the following page should be used for the design of the retaining walls.

Global Stability Analysis

The global stability analysis was modeled in SLIDE, a computer program which permits a two-dimensional slope stability analysis calculating several Limit Equilibrium methods including the Bishop's method, which is a widely accepted slope analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to forces 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 subsurface soil and groundwater conditions, a factor of safety greater than 1.0 is generally required for the failure risk to be considered acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise 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 analyses including seismic loading. The retaining wall cross-section was studied as the worst-case scenario. The following parameters were used for the slope stability analysis under static and seismic conditions:

Table 6 - Effective Soil Parameters for Static Analysis

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Unspecified Fill (Backfill)	16	33	1
Brown Silty Clay	17	33	5
Silty Sand	17	33	0
Glacial Till	20	33	1
Bedrock	22	-	-

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the open test holes completed at the time of our geotechnical investigation and based on our general knowledge of the geology in the area. The strength parameters used for seismic analysis under undrained conditions at the slope cross-section are presented in Table 7 on the following page.

Table 7 - Effective Soil Parameters for Seismic Analysis

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Unspecified Fill (Backfill)	16	33	1
Brown Silty Clay	17	-	100
Silty Sand	17	33	0
Glacial Till	20	33	1
Bedrock	22	-	-

Analysis Results

The factor of safety for each retaining wall section was greater than 1.5 for static conditions. Similarly, the results under seismic loading yielded a factor of safety for this section greater than 1.1. The results from the the global stability analysis are presented in Figures 2 to 4 presented in Appendix 2.

Based on these results, the global stability of the proposed retaining wall system is considered to be stable under static and seismic loading.

6.0 Design and Construction Precautions

6.1 Foundation Drainage, Waterproofing and Backfill

Foundation Drainage

Perimeter Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 100 to 150 mm diameter perforated, corrugated plastic pipe which is surrounded on all sides by 150 mm of 19 mm clear crushed stone, wrapped in a non-woven geotextile, and placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit.

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration for the lower basement area. For preliminary design purposes, it is recommended that 100 to 150 mm diameter perforated PVC pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Waterproofing System

Foundation Wall Waterproofing

A foundation waterproofing system is recommended for the proposed building. Based on review of the design drawings, it is anticipated that the majority of the building construction will be undertaken using open excavation methods, allowing for easy installation of the foundation drainage and waterproofing system. If temporary shoring is required, Paterson should provide supplemental foundation drainage and waterproofing recommendations.

The following waterproofing system is recommended for the proposed building below the groundwater table:

- ☐ A 150 MIL thick HDPE membrane, or an approved equivalent, should be installed in horizontal lifts to the manufacturer's specifications in a shingle fashion. The waterproofing membrane can be terminated 1 m above the long-term ground water table. It should be noted that termination elevation should be confirmed by Paterson once the excavation is completed.

- ☐ It is recommended that a composite foundation drainage membrane, such as Delta Terraxx, G100N by MiraDrain, or equivalent and approved other, be placed to the outside of the waterproofing membrane to divert water captured by the building foundation drainage system to the appropriate sump pump system. The composite foundation drainage membrane should extend from finished grade to the underside of footing level with the geotextile layer facing away from the foundation wall in a shingle fashion.

- ☐ It is highly recommended that the drainage boards be installed with a minimum horizontal and vertical overlap of 150 mm between the sheets (not the filter cloth) to minimize the joints between the sheets. The top 150 mm flap of each lower horizontal lift of the drainage board should be secured behind the bottom end lap of the overlying horizontal course of the drainage board to allow for proper shingling between lifts.

- ☐ 100 to 150 mm diameter sleeves should be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior drainage pipe. Further, the drainage sleeves should be mechanically connected to the exterior or interior perimeter drainage pipe and the underfloor drainage system. The perimeter and underslab drainage pipes should direct water to the storm sump pit(s) within the lower basement area by gravity. It is recommended that an ‘X’ shaped incision through the composite foundation drainage board be cut to connect the sleeve. The sleeves should be fastened and secured into place at the footing/wall interface prior to casting concrete for the foundation wall. The incision in the drainage board should be sealed with 3M tape around the sleeve. Further, the waterproofing membrane should be sealed around the sleeve.

- ☐ All joints between drainage board sheets (i.e., overlaps) should be sealed using 3M Tape, or equivalent other product approved by Paterson. Further, all protrusions through the waterproofing and drainage board layers liner, such as fasteners or damage by materials such as concrete, should be sealed using 3M Tape or equivalent other products approved by Paterson to mitigate the potential for water to drain behind the HDPE face of the drainage board.

Elevator (and Sump) Pit Waterproofing

All elevator shaft exterior foundation walls and floor slabs should be waterproofed to avoid any infiltration into the elevator pit.

The underside of the elevator pit slab should be waterproofed using a membrane such as Colphene BSW H for horizontal applications (or approved equivalent).

It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved equivalent), is applied to the exterior of the elevator shaft foundation wall. The membrane should extend to the top of the footing in accordance with the manufacturer's specifications.

A continuous PVC waterstop, such as Southern Waterstop 14RCP (or approved equivalent), should be installed within the interface between the concrete base slab below the elevator pit sidewalls. An outlet for any trapped water should be installed through the elevator pit wall or floor slab with a gravity connection to the underfloor drainage system, or directly to the sump pit.

A protection board should be placed over the waterproofing membrane to protect the membrane from damage during the backfilling operations.

Consideration should also be given to waterproofing the sump pit(s). If chosen, the above-noted waterproofing will also be applicable to sump pit waterproofing.

Foundation Backfill

Foundation Walls

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.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas, or other settlement sensitive structures which are not adjacent to the buildings, should consist of free-draining, non-frost susceptible material.

This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the materials' standard Proctor maximum dry density (SPMDD) under dry and above freezing conditions.

Finalized Drainage and Waterproofing Design

Paterson can provide a more detailed drainage and waterproofing design once detailed design drawings, as well as an excavation plan are available. Paterson should be provided with the finalized architectural, civil, mechanical and structural drawings for the proposed buildings to provide a building-specific waterproofing and drainage design which includes the above-noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double-sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

Perimeter footings 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, retaining walls, and underground parking ramp footings (or footings adjacent to garage bay doors which may be open to exterior conditions for extended periods of time), are more prone to deleterious movement associated with frost action.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for all exterior unheated footings.

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 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.

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, such as against Richmond Road or the existing dwelling to the north. 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 event 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 7 – 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 is 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 98% 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 sand to sandy silt and silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions.

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 low to 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 of 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. 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.

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 moderately aggressive environment.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

☐ Review preliminary and detailed grading, servicing and landscaping plans from a geotechnical perspective.

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

☐ Review the proposed foundation drainage design system.

☐ Complete field reviews of the excavation operations.

☐ Complete field reviews of the foundation drainage system installation.

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

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

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

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

☐ Observation of all subgrades prior to placing backfill material.

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

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

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

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Ottawa General Contractors 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.

O. Canton

Owen R. Canton, B. Eng.

LICENSED PROFESSIONAL ENGINEER
Apr. 28, 2025
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

❑ Ottawa General Contractors (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 Residential Building - 3055 Richmond Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

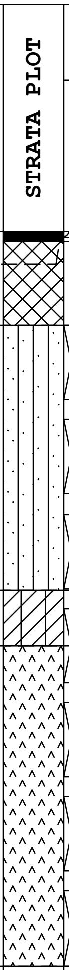
DATE 2022 May 6

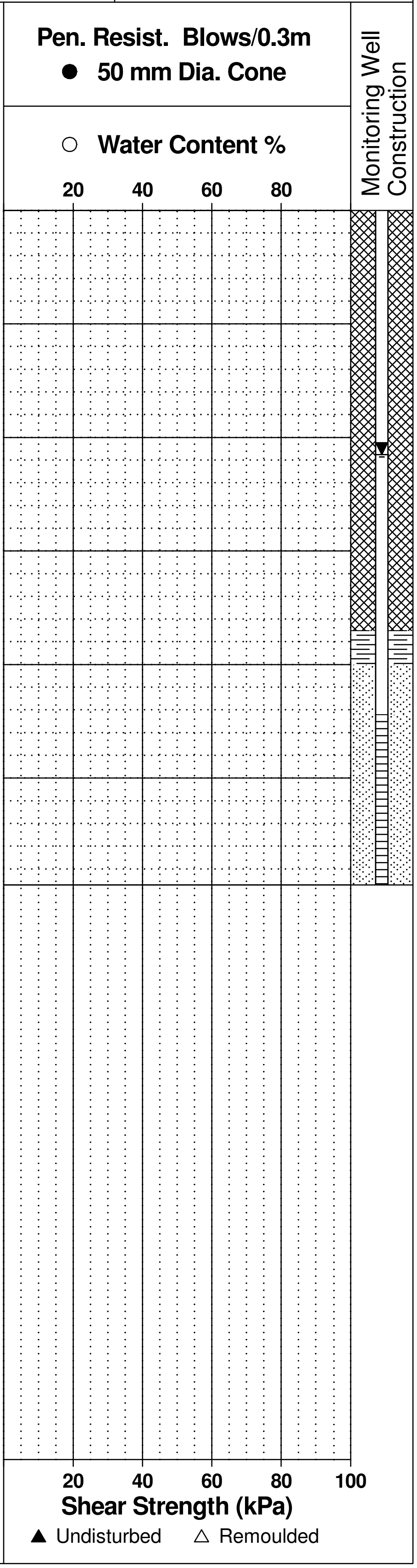
FILE NO. PG6187

HOLE NO. BH 1-22

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.08	AU	1		
FILL: Brown silty sand, some clay, trace gravel	0.76				
Comapct, brown **SILTY SAND to SANDY SILT,** some clay, trace gravel	2.90	SS SS SS	2 3 4	42 100 17	15 14 19
Very stiff, brown **SILTY CLAY**	3.35	SS	5	100	14
GLACIAL TILL: Hard to very stiff, grey silty clay with sand, gravel, cobbles and boulders	5.94	SS SS SS	6 7 8	75 58 25	13 24 12
End of Borehole					
(GWL at 2.15 m depth - May 19, 2022)					

DEPTH (m)	ELEV. (m)
0	74.49
1	73.49
2	72.49
3	71.49
4	70.49
5	69.49





patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Building - 3055 Richmond Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2022 May 6

FILE NO. PG6187

HOLE NO. BH 2-22

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	73.90
TOPSOIL 0.10						
FILL: Brown silty sand, trace clay 0.60	AU	1				
FILL: Brown silty sand to sandy silt, trace clay 1.70	SS	2	75	11	1	72.90
FILL: Brown silty sand, trace gravel	SS	3	17	49	2	71.90
2.97	SS	4	4	71		
GLACIAL TILL: Hard, grey silty clay with sand, gravel, cobbles and boulders	SS	5	42	25	3	70.90
4.42	SS	6	50	14	4	69.90
GLACIAL TILL: Very dense, grey silty sand to sandy silt with gravel, cobbles and boulders	SS	7	58	75	5	68.90
5.89	SS	8		60		

End of Borehole

Practical refusal to augering at 5.89m depth.

(GWL at 5.84 m depth - May 19, 2022)

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

○ **Water Content %**

20 40 60 80

Monitoring Well 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 Residential Building - 3055 Richmond Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2022 May 6

FILE NO. **PG6187**

HOLE NO. **BH 3-22**

GROUND SURFACE

SOIL DESCRIPTION	Depth (m)
TOPSOIL	0.08
FILL: Brown silty sand, trace clay, organics, topsoil and brick	1.83
Very stiff to stiff, brown **SILTY CLAY** - grey by 3.7m depth	4.50
GLACIAL TILL: Hard, grey silty clay with sand, gravel, cobbles and boulders	5.94
Dynamic Cone Penetration Test commenced at 5.94m depth.	9.09

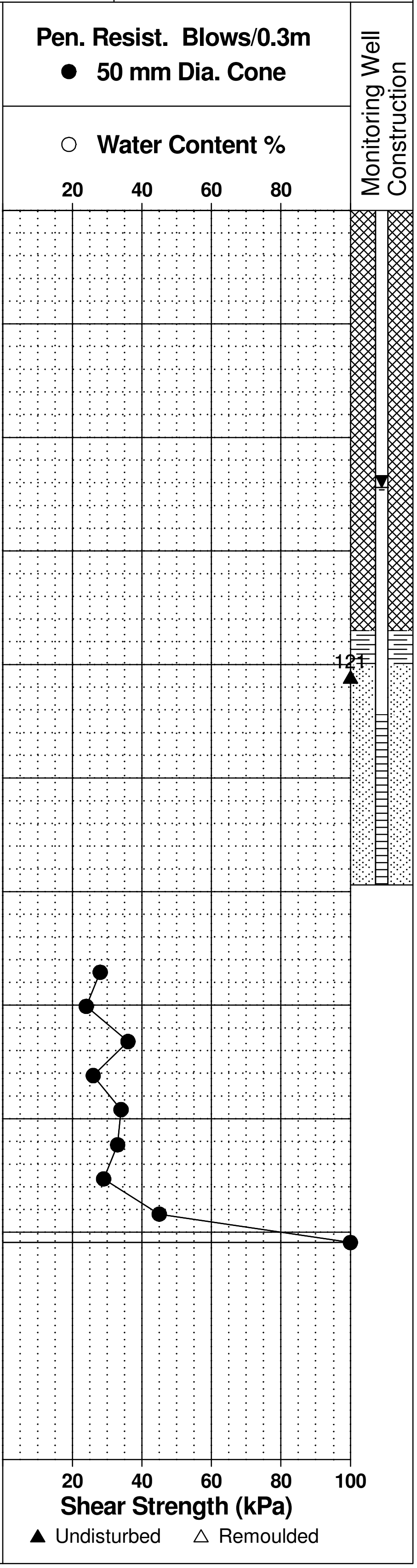
End of Borehole

Practical DCPT refusal at 9.09m depth.

(GWL at 2.44 m depth - May 19, 2022)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	8	24
SS	3	75	7
SS	4	92	4
SS	5	92	6
SS	6	100	2
SS	7	50	16
SS	8	42	23

DEPTH (m)	ELEV. (m)
0	75.58
1	74.58
2	73.58
3	72.58
4	71.58
5	70.58
6	69.58
7	68.58
8	67.58
9	66.58



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Building - 3055 Richmond Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE July 13, 2023

FILE NO. PG6187

HOLE NO. TP 1-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
Ground Surface						0	73.24		
TOPSOIL 0.10		G	1						
FILL: Brown silty sand, trace to some crushed stone, trace asphalt and clay 0.84		G	2						
Compact to dense, brown **SILTY SAND to SANDY SILT,** trace to some gravel 1.25		G	3			1	72.24		
GLACIAL TILL: Dense, brown silty sand to sandy silt, some gravel, trace to some clay 1.92		G	4						
End of Test Pit (TP dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Building - 3055 Richmond Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE July 13, 2023

FILE NO. PG6187

HOLE NO. TP 2-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
Ground Surface						0	73.54		
TOPSOIL 0.15		G	1						
FILL: Topsoil with crushed stone, some gravel 0.25		G	2						
FILL: Brown silty sand with crushed stone and gravel, trace glass, occasional cobbles and boulders		G	3						
						1	72.54		
1.20									
GLACIAL TILL: Brown silty sand to sandy silt with clay and gravel, occasional cobbles and boulders		G	4						
						2	71.54		
2.21									
End of Test Pit (TP dry upon completion)									

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 relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

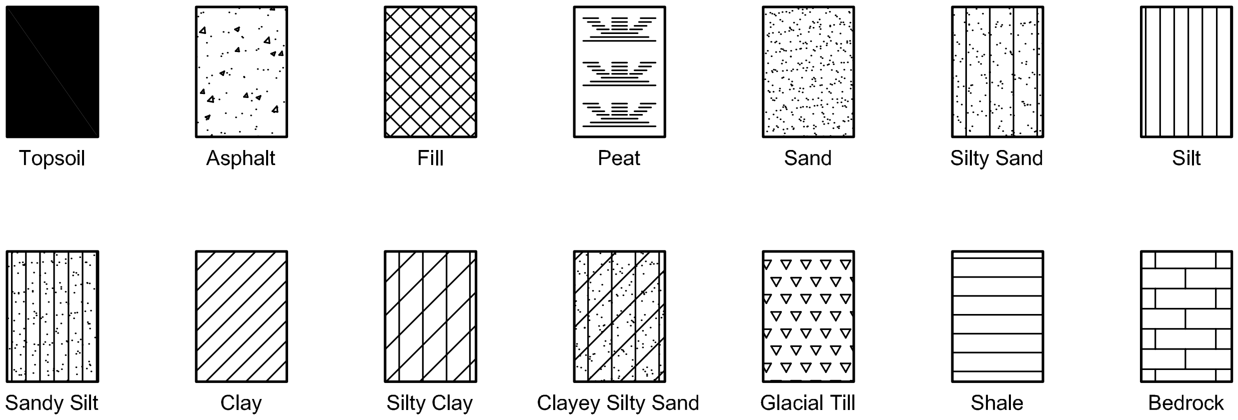
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

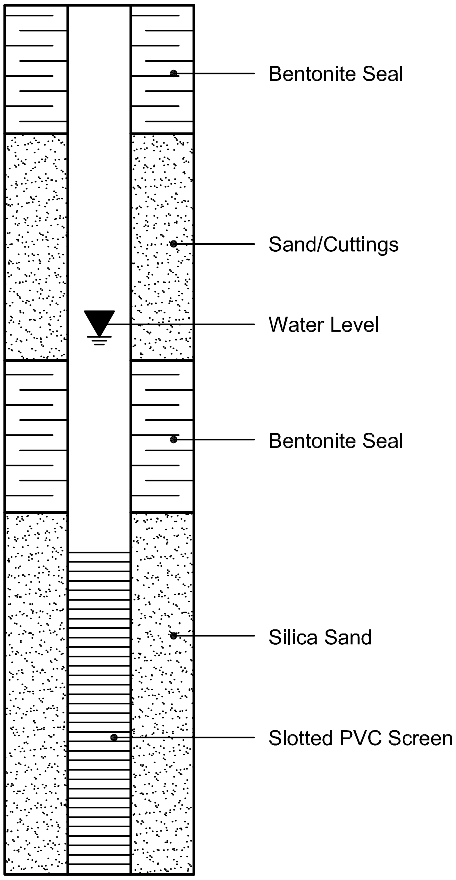
SYMBOLS AND TERMS (continued)

STRATA PLOT

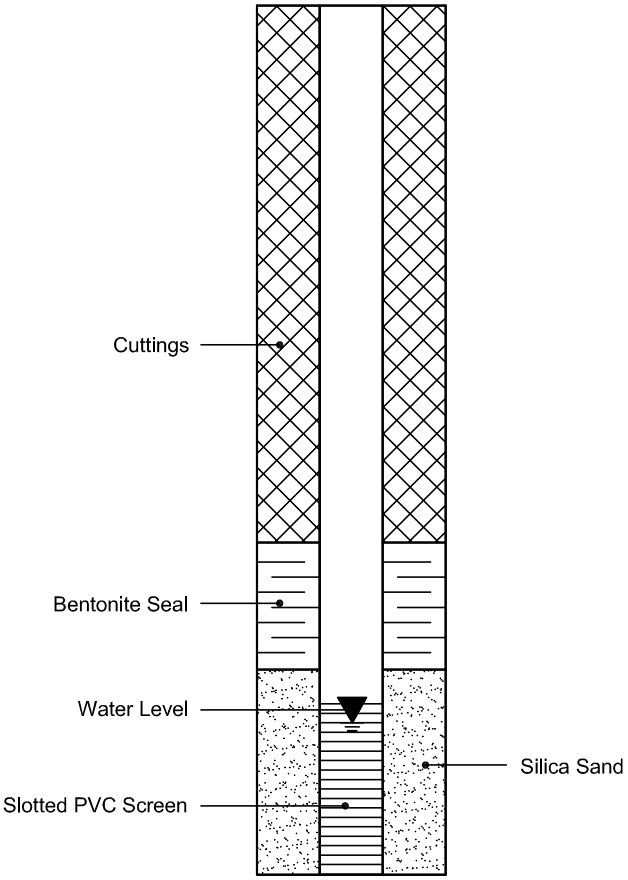


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: **54540**

Report Date: 13-May-2022

Order Date: 9-May-2022

Project Description: PG6187

	Client ID:	BH1-22-SS3	-	-	-
	Sample Date:	06-May-22 09:00	-	-	-
	Sample ID:	2220173-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	84.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.50	-	-	-
Resistivity	0.10 Ohm.m	30.6	-	-	-
Anions					
Chloride	5 ug/g dry	96	-	-	-
Sulphate	5 ug/g dry	46	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – RETAINING WALL SECTION ‘A’ – STATIC ANALYSIS

FIGURE 3 – RETAINING WALL SECTION ‘A’ – SEISMIC ANALYSIS – 0.16G

FIGURE 4 – RETAINING WALL SECTION ‘B’ – STATIC ANALYSIS

FIGURE 5 – RETAINING WALL SECTION ‘B’ – SEISMIC ANALYSIS – 0.16G

DRAWING PG6187-1 – TEST HOLE LOCATION PLAN

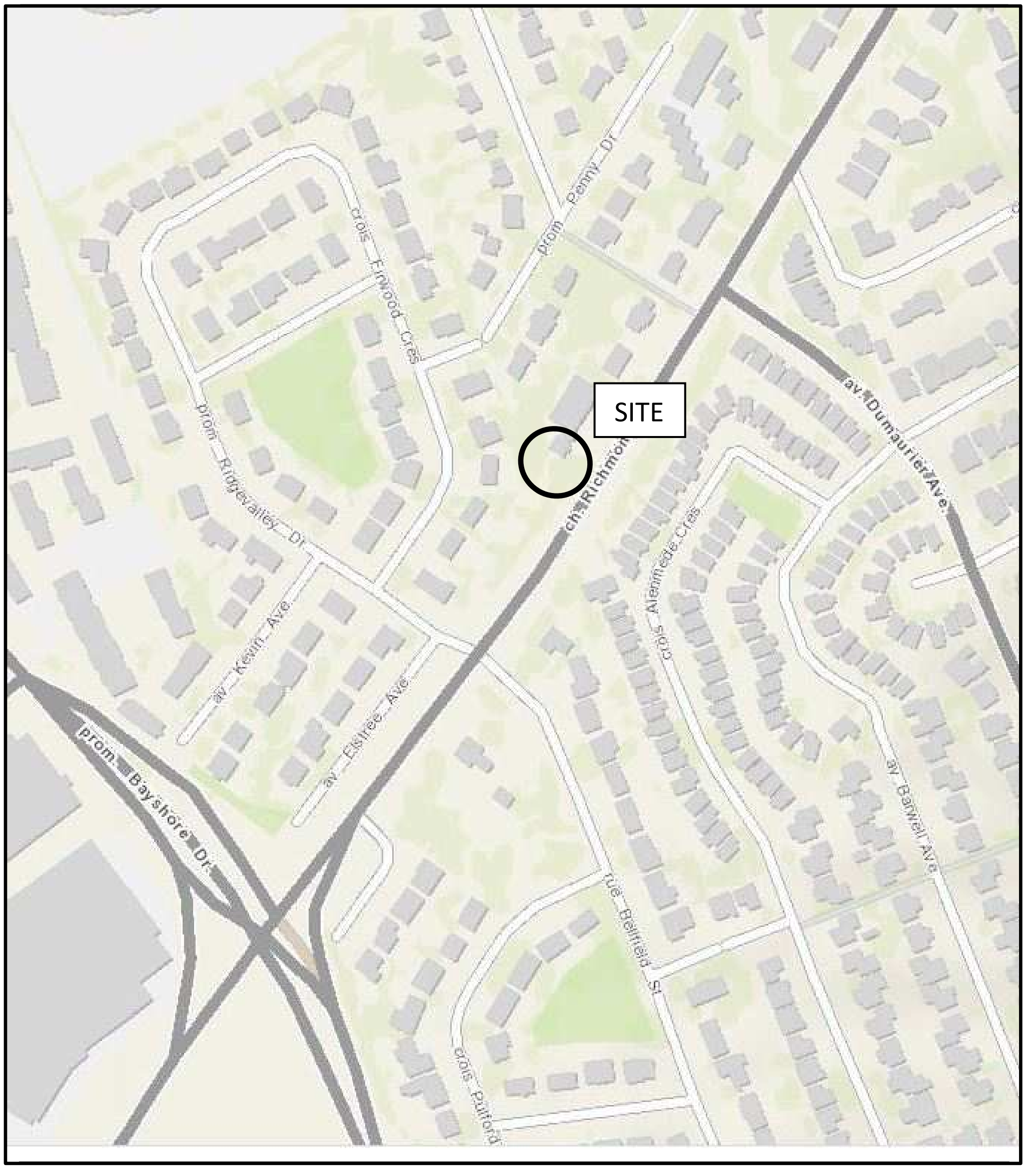


FIGURE 1 – KEY PLAN

PATERSON GROUP

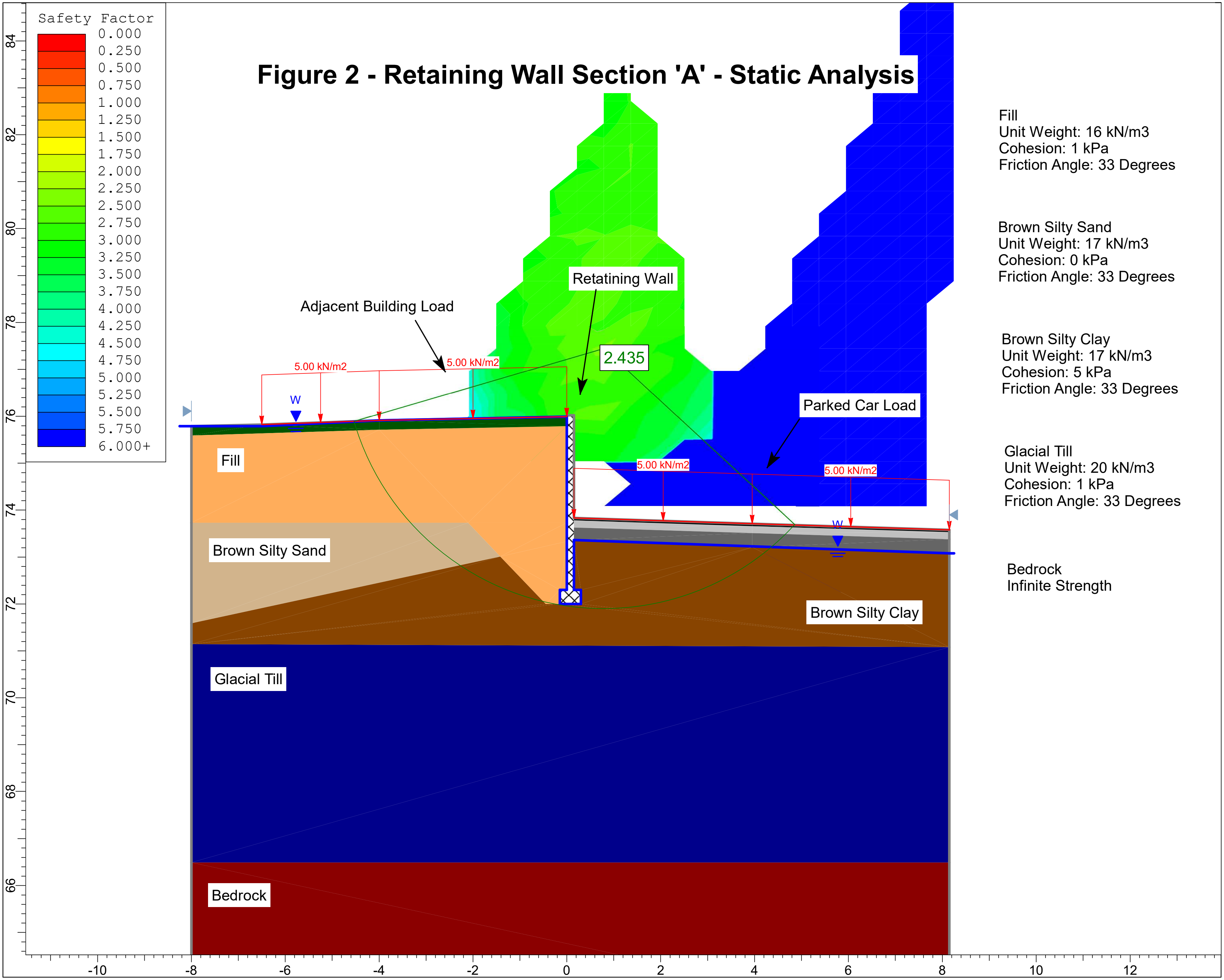


Figure 2 - Retaining Wall Section 'A' - Static Analysis

Fill
Unit Weight: 16 kN/m3
Cohesion: 1 kPa
Friction Angle: 33 Degrees

Brown Silty Sand
Unit Weight: 17 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 Degrees

Brown Silty Clay
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 Degrees

Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 33 Degrees

Bedrock
Infinite Strength

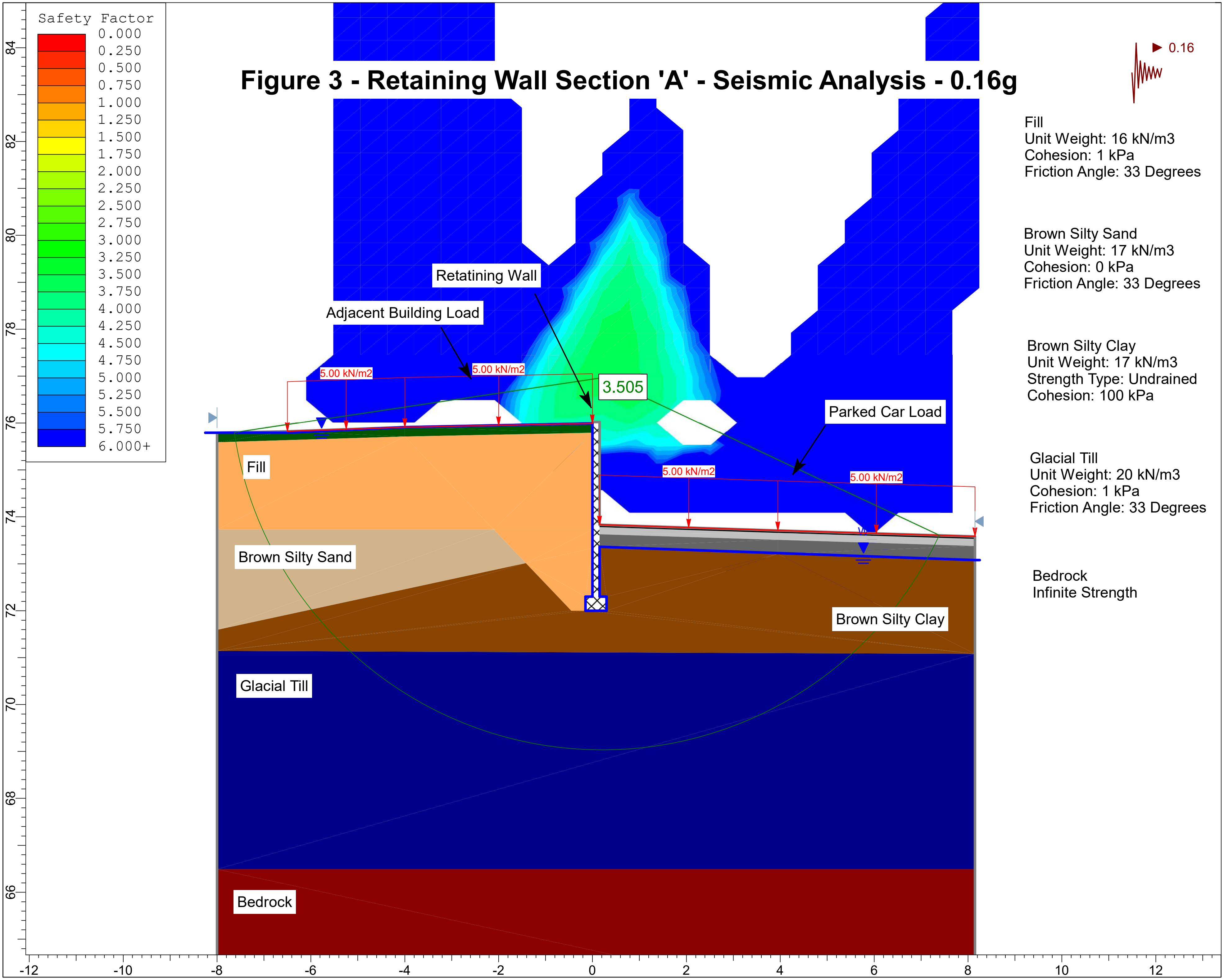


Figure 3 - Retaining Wall Section 'A' - Seismic Analysis - 0.16g

Fill
Unit Weight: 16 kN/m3
Cohesion: 1 kPa
Friction Angle: 33 Degrees

Brown Silty Sand
Unit Weight: 17 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 Degrees

Brown Silty Clay
Unit Weight: 17 kN/m3
Strength Type: Undrained
Cohesion: 100 kPa

Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 33 Degrees

Bedrock
Infinite Strength

Figure 4 - Retaining Wall Section 'B' - Static Analysis

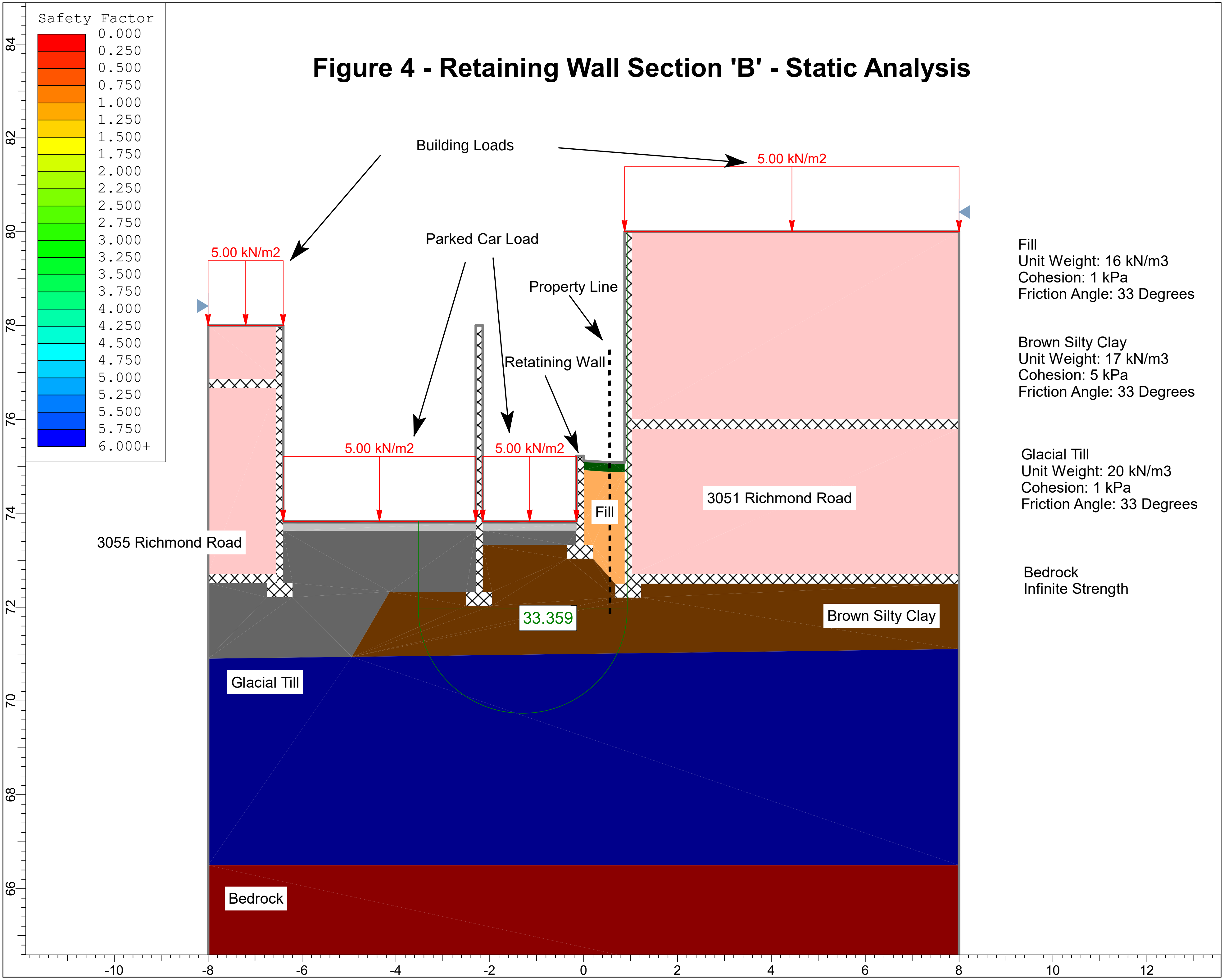
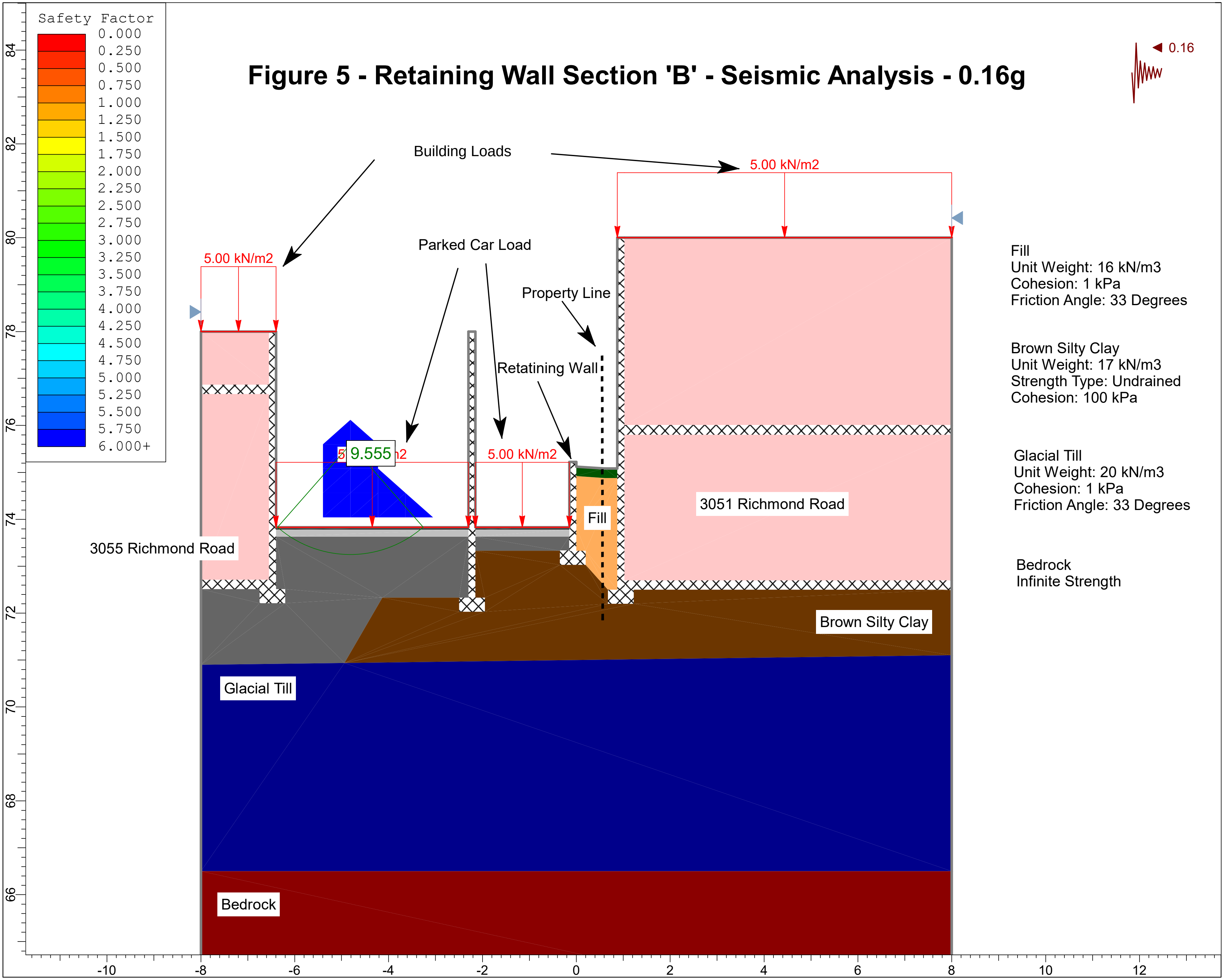
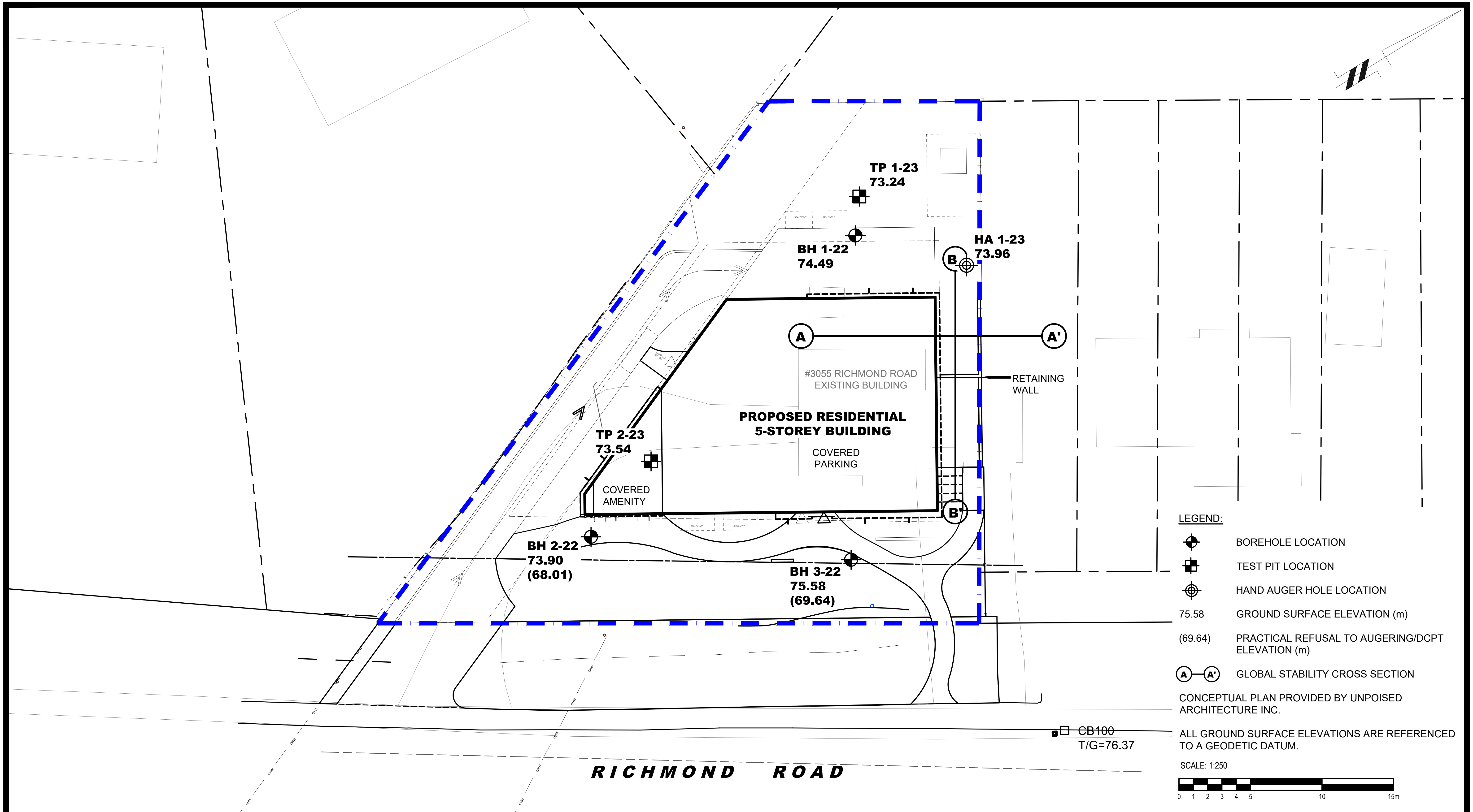


Figure 5 - Retaining Wall Section 'B' - Seismic Analysis - 0.16g





LEGEND:

⊕	BOREHOLE LOCATION
■	TEST PIT LOCATION
◎	HAND AUGER HOLE LOCATION
75.58	GROUND SURFACE ELEVATION (m)
(69.64)	PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)
Ⓐ—Ⓐ'	GLOBAL STABILITY CROSS SECTION

CONCEPTUAL PLAN PROVIDED BY UNPOISED ARCHITECTURE INC.

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:250

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
3	UPDATED TO NEW CONCEPTUAL PLAN	27/03/2025	OC
2	ADDED TEST PITS AND HAND AUGER HOLE LOCATION	20/07/2023	OC
1	UPDATED TO NEW CONCEPTUAL PLAN	07/03/2023	OC

OTTAWA GENERAL CONTRACTORS

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
3055 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 05/2022
Drawn by: NFRV
Report No.: PG6187-1
Checked by: OC
Dwg. No.: PG6187-1
Approved by: FA
Revision No.: 3

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