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Geotechnical Investigation

Proposed Mixed Use Development
2582-2584, 2600 and 2626 Bank Street
Ottawa, Ontario

Prepared For

Mr. Nabil Abdullah

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Report: PG5545-1
Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Mr. Nabil Abdullah to prepare a geotechnical investigation report based on existing information for the proposed mixed-use development to be located at 2582-2584, 2600 and 2626 Bank Street in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the current investigation were to:

❏ determine the subsoil and groundwater conditions at the site by means of nearby test holes taken from previous investigations conducted by Paterson in the same development.

❏ provide geotechnical recommendations for the design of the proposed development based on the results of the boreholes and other soil information available. These recommendations include final grade raises, long term settlements and other construction considerations which may affect its 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 proposed development as it is understood at the time of writing this report.

2.0 Proposed Development

It is understood that the proposed development will consist of three mixed-use buildings (Buildings A, B, and C). Each of proposed buildings will generally include 3 storeys each, with an average footprint area of around 1,100 to 1,630 m^2. Two common underground parking level will be constructed within the footprint of Buildings A and B and partially extending below Building C. The remaining portion of Building C is proposed to be constructed as slab-on-grade buildings. Associated access lanes, at-grade parking areas and landscaped areas are also anticipated. It is expected that the proposed buildings will be municipally serviced.

3.0 Existing Geotechnical Information

3.1 Field Investigation

Field Program

A previous field investigation conducted by others was completed in October 2018. At that time, a total of four boreholes were advanced to a maximum depth of 11.6 m below existing grade at the aforementioned site. The locations of the test holes are shown on Drawing PG5545-1- Test Hole Location Plan included in Appendix 2.

3.2 Surface Conditions

The majority of the subject site is currently covered with trees and vegetation with the exception of 2600 Bank Street, which is occupied by an existing one-storey slab on grade commercial building with the associated asphalt and gravel surfaced parking areas. 2582 Bank Street was observed to be heavily treed throughout the higher portions of the property with brush and occasional trees along its slope face. The property currently designated as 2626 Bank Street is generally tree-covered with a section of brush and a gravel path throughout the centre and eastern boundary of the property parcel. A private storm sewer alignment is present along the eastern boundary of 2626 Bank Street and outletting into the adjacent creek.

The area of 2600 Bank Street is generally at grade with Bank Street and surrounding properties. However, the ground surface slopes downward from Bank Street towards the west and south property boundaries throughout 2582 and 2626 Bank Street. The ground surface slopes downward from east to west between approximate geodetic elevations of 99.5 to 95.0 m across 2582 Bank Street. The ground surface slopes downward from north to south between approximate geodetic elevations of 99.5 to 90.5 m across 2626 Bank Street. The ground surface along the eastern 2626 Bank Street property is approximately 1.5 m below the adjacent grade at the north and south ends of the neighbouring property, respectively.

A slope is located along the east and southeastern boundary lines backing into Sawmill Creek. A site visit was completed by Paterson personnel to review slope conditions throughout the area adjacent to the aforementioned creek, and is further discussed in Section 6.8 of this report. It should be notes that various slopes and a small craters were observed and noted on the south portion of the site. Base on historical imaging, it appears that this portion of the site was occupied by a structure that was later demolished and removed. Signs of brick and construction debris can be observed throughout the area and along the adjacent slope.

The subject site is bound to the north by Bank Street, to the east by an existing one-storey commercial building followed by a cemetery, to the south by a creek and to the west by a residential subdivision. Existing topographic elevations are lines with the current conditions including the existing building are presented in Drawing PG5545-1 - Test Hole Location Plan in Appendix 2.

3.3 Subsurface Profile

Overburden

Based on the existing soils information within the location of the proposed development and our knowledge of the subsurface profile within the neighbouring properties, the subsurface profile consists of topsoil and/or asphaltic concrete followed by a layer of very loose to loose silty sand with occasional gravel and crushed stone. Glacial till consisting of silty sand with clay, gravel, cobbles and boulders was encountered below the above noted layer. It should be noted that a layer of peat was encountered at the location of BH 3 between 1.1 to 2.4m below ground surface.

The subsoil profile encountered at the locations of the test holes are detailed in the borehole logs prepared by others and presented in Appendix 1.

Bedrock

Based on available geological mapping, the bedrock in this area consists of shale of the Carlsbad formation, and is expected at depths ranging from 15 to 50 m.

3.4 Groundwater

The results of groundwater level (GWL) measurements were recorded from the monitoring wells installed by others. The recorded groundwater is summarized in Table 1 below. Based on the recorded groundwater reading, the recorded colour and consistency of the subsurface profile, the groundwater table is expected at a depth ranging from 9 to 10 m below existing grade at an approximate elevation of 89 to 90 m. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could be different at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Groundwater Depth (m)	Recording Date
BHMW18-2	10.10	October 15, 2018
BHMW18-3	8.37	October 15, 2018

4.0 Discussion

4.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development. It is expected that the proposed buildings be founded using conventional shallow footings placed over an undisturbed, compact silty sand to sandy silt or compact glacial till bearing surface.

Due to the presence of a peat layer within the northwest portion of the site (near building A and building B), additional site preparation recommendations will be required depending on the extent of the peat layer prior to placement of the proposed footings.

Where existing fill is located within the footprint of a previously demolished building, the existing fill should be entirely removed from within the proposed buildings and replaced with engineered fill. The engineered fill placement is further discussed in Subsection 4.2.

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

4.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt pavement and deleterious fill, such as those containing significant amounts of organic material or construction debris, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

A peat layer was noted in BH/MW18-3 between 1.1 to 2.4m below ground surface. In areas where peat is encountered below a settlement sensitive structures (i.e. building floor slabs and roads), the peat layer should be entirely sub-excavated within the proposed building footprint and replaced with engineering fill, approved by Paterson at the time of construction. The engineered fill should consist of OPSS Granular A or Granular B Type II compacted to a minimum 98% of the material's SPMDD. Additional fill placement recommendations are summarized below.

Fill Placement

Fill used 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, 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 building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

4.3 Foundation Design

Bearing Resistance Values

Footings placed on the undisturbed, compact silty sand to sandy silt bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **200 kPa** incorporating a geotechnical factor of 0.5. It is important to note that where the silty sand to sandy silt bearing medium is encountered at a loose state of compactness, proof rolling must be completed. The proof rolling operations should be completed using vibratory equipment making several passes and reviewed and approved by the Paterson at the time of construction to achieve the above noted bearing resistance values.

Footings placed on a glacial till or engineered fill placed on an undisturbed glacial till bearing surface, can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa.** A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

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 prior to the placement of concrete for footings.

Settlement

Footings placed on an undisturbed soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

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 sand above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

4.4 Design for Earthquakes

The proposed site can be taken as seismic site response **Class D** as defined in the Ontario Building Code 2012 (OBC 2012 (R2019); Table 4.1.8.4.A) for foundations considered at this site. The soils underlying the site are not susceptible to liquefaction.

4.5 Slab on Grade and Basement Slab

With the removal of all topsoil and fill containing organic matter within the footprints of the proposed buildings, the native soil surface or engineered fill will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Provision should be made for proof-rolling the soil subgrade using heavy vibratory compaction equipment making several passes and approved by Paterson prior to placing any fill.

It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone. It is also recommended that the basement slab be placed over a minimum 300 mm thick layer of clear 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 at least 98% of its SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

4.6 Basement Walls

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structures. 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.

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.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

The peak ground acceleration, (a_{max}), for the 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.

4.7 Pavement Structure

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

Table 2 - Recommended Pavement Structure - Car Parking Areas (Driveways)

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 or fill	

Table 3 - Recommended Pavement Structure - Access Lanes and Heavy Truck Loading Areas

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
375	**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	

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 I or II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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 vibratory equipment.

5.0 Design and Construction Precautions

5.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for proposed structures. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, 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.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

Underfloor Drainage

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

5.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard. Exterior (unheated) footings should be provided with a minimum 2.1 m thick soil cover (or equivalent).

The parking garage entrance may require protection against frost action depending on the founding depth of the adjacent footings and the drainage measures taken at the entrance. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

5.3 Excavation Side Slopes

The side slopes of excavations in the soil materials should either be 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). However, temporary shoring may be required for some sections around the perimeter of the site at the property boundaries.

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 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 at least 4 to 6 m away from the excavation face depending on the excavation depth and soil consistency.

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 (Buildings A and B)

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 4 - Soil Parameters

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

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

5.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for pipe bedding for sewer and water pipes for a soil subgrade. The bedding should be increased to 300 mm for areas where the subgrade consists of bedrock. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. 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.

5.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

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

Long-Term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the perimeter or sub-slab drainage system will be directed to the proposed buildings' sump pits. Provided that the selected groundwater infiltration control system is properly implemented and approved by Paterson at the time of construction, it is expected that groundwater flow will be low (i.e. less than 25,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

It is understood that one underground parking level is planned for the proposed Buildings A and B of the aforementioned development. Based on our review of the long-term groundwater table along with the subsurface soil profile, the long-term groundwater table is expected at depths ranging from 9 to 10 m depth below existing grade. Therefore, no issues are expected with respect to groundwater lowering that would cause long term damage to the adjacent structures surrounding the proposed development.

5.6 Winter Construction

The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur. Precautions should be taken if winter construction is considered for this project.

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.

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

5.7 Slope Stability Assessment

Background Information and Field Observations

A section of Sawmill Creek is located at the south property boundary of 2626 Bank Street. The slope condition was reviewed by Paterson field personnel as part of this geotechnical investigation. Two slope cross-sections were studied as the worst case scenarios. The slope stability analysis was carried out to determine the required construction setback from the top of the creek's bank based on a factor of safety of 1.5. Toe erosion and erosion access allowances were also considered in the determination of limits of hazard lands and are discussed on the following pages. The cross section locations are shown on Drawing PG5545-2 - Limit of Hazard Lands Plan in Appendix 2.

Based on our current field observations, no erosion activity was observed along the toe or face of the subject slope along the southern boundaries of the subject site. Sawmill Creek was observed to consist of a 4 to 5 m wide channel with relatively shallow banks. The section of the creek adjacent to the subject site was observed to be relatively straight with no meandering noted. Generally, the banks along Sawmill Creek are well vegetated and stable. The slope was noted to be 1 to 1.5 m high with a minimum inclination ranging between 1.5H:1V and 2H:1V along the creek. The upper portion of the slope was observed to be vegetated and stable with no signs of surficial erosion activity. Photos taken of the creek slopes during the site visit are presented in Appendix 3.

Due to the presence of construction debris within the southern portion of the site as a result of demolishing an old building, the slope adjacent to the former building is considered a "man-made" slope. Therefore, once the new development is constructed, it is expected that reinstating the non-natural slope will be completed as part of the subject development. Refer to subsection 4.2 for fill placement requirements.

The following sections will discuss the slope stability analysis of the existing slope along Sawmill Creek. The discussions will also include the recommended limit of hazard lands setbacks as per the City of Ottawa Guidelines for slope stability.

Limit of Hazard Lands

The slope condition was reviewed based on available topographic mapping along the side slopes of the creek within the south portion of the subject development. A total of 2 slope cross-sections were assessed as the worst case scenarios. The cross section locations are presented on Drawing PG5545-2 - Limit of Hazard Lands in Appendix 2.

A slope stability assessment was carried out to determine the required stable slope allowance setback from the top of slope based on a factor of safety of 1.5. A toe erosion and 6 m access allowances were also included in the determination of limits of hazard lands and are discussed below. The proposed limit of hazard lands (as shown on Drawing PG5545-2 - Limit of Hazard Lands Plan includes:

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

Slope Stability Assessment

The analysis of slope stability was carried out using SLIDE, a computer program that 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 favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain than the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

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

Due to the permeability of the subsurface soils, it is anticipated that the groundwater table will be located at an elevation of 89 to 90 m. Subsoil conditions at the cross-sections were inferred based on the findings of the borehole located along the top of slope and general knowledge of the area's geology.

Stable Slope Allowance

The results of the stability analysis for static conditions at Sections 1 and 2 are presented on Figures 2A and 3A, respectively attached to Appendix 2. The results indicate that the factor of safety for the sections is greater than 1.5 for both sections. Therefore, a stable slope setback is not required for the subject slope.

The results of the analyses including seismic loading for Section 1 and 2 are shown on Figures 2B and 3B, respectively and attached to Appendix 2. The results indicate that the factor of safety for both sections is greater than 1.1.

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

Toe Erosion and Access Allowance

The toe erosion allowance for the valley corridor wall slope was determined based on the cohesionless nature of the silty sand with the subject slope, the observed current erosion activities and the width and location of the current watercourse. Since no sign of erosion activity was observed during our site investigation, a toe erosion allowance of 2 m is recommended for the site.

An erosion access allowance of 6 m is required from the top of slope as per the City of Ottawa Slope Stability Guidelines. The limit of hazard lands, which includes these allowances, is indicated on Drawing PG5545-2 - Limit of Hazard Lands Plan in Appendix 2.

5.8 Landscaping Considerations

Tree Planting Restrictions

According to the City of Ottawa Guidelines for tree planting, where a sensitive silty clay deposit is present within the vicinity of the site, tree planting restrictions should be determined. However, for this site, since no silty clay is present within the subject site, tree planting restrictions are not required from a geotechnical perspective.

6.0 Recommendations

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

- ❑ Review detailed grading plan(s) from a geotechnical perspective.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to ensure that the specified level of compaction has been 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 request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

7.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of 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 contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractors construction methods, equipment capabilities and schedules.

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Aug. 23, 2021
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Joey R Villenueve, M.A.Sc., P.Eng Faisal I. Abou Seido, P.Eng.

Report Distribution:

- ❏ Mr. Nabil Abdullah
- ❏ Paterson Group Inc.

APPENDIX 1

EXISTING BOREHOLE LOGS - BY OTHERS

BH18-1

DST consulting engineers

DST Project No.	TS-SO-032782	Date	11 October 2018
Client	Power Market Real Estate Brokerage	Method	CME 75
Project	Phase II ESA	Diameter	50.8mm
Address	2582-2584, 2600, 2626 Bank Street, Ottawa, ON	Coordinates	5022391.9 m N, 449941.8 m E

Depth (m)	Elevation (m)	Water level (m)	Well construction	Depth (m) Elevation (m)	Symbol	Material Description	Sample ID	Sample Type	'N' Value / RQD	CCGD / PID Reading: CCGD	PID	Analysis (Submitted for laboratory analysis): PAHs	PHCs	Metals	VOCs	Others	Remarks
				0		SAND WITH GRAVEL - coarse grained, dry	SS1		25	0 ppm							
1							SS2		18	0 ppm							
							SS3		39	0 ppm			✓		✓		
2							SS4		33	0 ppm			✓				
							SS5		60	0 ppm							
3							SS6		37	0 ppm							
4							SS7		35	0 ppm							
							SS8		38	0 ppm							
5							SS9		35	0 ppm							
6							SS10		35	0 ppm							
							SS11		68	0 ppm							
7							SS12		77	0 ppm							
				7.2		Sand - coarse grained, some gravel, dry											

BH18-1

DST consulting engineers

DST Project No.	TS-SO-032782	Date	11 October 2018
Client	Power Market Real Estate Brokerage	Method	CME 75
Project	Phase II ESA	Diameter	50.8mm
Address	2582-2584, 2600, 2626 Bank Street, Ottawa, ON	Coordinates	5022391.9 m N, 449941.8 m E

Depth (m)	Elevation (m)	Water level (m)	Well construction	Depth (m) Elevation (m)	Symbol	Material Description	Sample ID	Sample Type	'N' Value / RQD	CCGD / PID Reading: CCGD	CCGD / PID Reading: PID	Analysis (Submitted for laboratory analysis): PAHs	PHCs	Metals	VOCs	Others	Remarks
						Sand - coarse grained, some gravel, dry	SS13		38	0 ppm							
8							SS14		49	0 ppm							
9							SS15		55	0 ppm							
						End of Borehole at 9.1 m.											
10																	
11																	
12																	
13																	
14																	

BH/MW18-2

DST consulting engineers

DST Project No.	TS-SO-032782	Date	11 October 2018
Client	Power Market Real Estate Brokerage	Method	CME 75
Project	Phase II ESA	Diameter	50.8mm
Address	2582-2584, 2600, 2626 Bank Street, Ottawa, ON	Coordinates	5022383.6 m N, 449923.5 m E

Depth (m) / Elevation (m)	Material Description	Sample ID	Sample Type	'N' Value / RQD	CCGD	PID	PAHs	PHCs	Metals	VOCs	Others	Remarks
0	SAND WITH GRAVEL - followed by stiff gravelly clay	SS1		12	0 ppm							
0.6	SILTY SAND - some gravel, brown, dry	SS2		11	0 ppm							
1.2	SAND WITH GRAVEL - brown, dry	SS3		6	0 ppm							
		SS4		34	0 ppm							
2.4	- coarse grained	SS5		53	0 ppm							
		SS6		24	0 ppm			✓		✓		
		SS7		45	0 ppm							
		SS8		26	0 ppm							
		SS9		37	0 ppm							
		SS10		31	0 ppm							
6.1	SAND - some gravel, dry	SS11		46	0 ppm							
		SS12		38	0 ppm							

BH/MW18-2

consulting engineers

DST Project No.	TS-SO-032782	Date	11 October 2018
Client	Power Market Real Estate Brokerage	Method	CME 75
Project	Phase II ESA	Diameter	50.8mm
Address	2582-2584, 2600, 2626 Bank Street, Ottawa, ON	Coordinates	5022383.6 m N, 449923.5 m E

Depth (m)	Elevation (m)	Water level (m)	Well construction	Depth (m) / Elevation (m)	Symbol	Material Description	Sample ID	Sample Type	'N' Value / RQD	CCGD	PID	PAHs	PHCs	Metals	VOCs	Others	Remarks
						SAND - some gravel, dry	SS13		19	0 ppm							
8							SS14		43	0 ppm							
				8.4		- coarse grained, brown											
9							SS15		35	0 ppm							
							SS16		31	0 ppm							
				9.7		- moist											
10							SS17		68	0 ppm							Groundwater level at 10.10 mbgs
				10.3		- wet											
							SS18		54	0 ppm							
11							SS19			0 ppm							
						End of Borehole at 11.6 m.											
12																	
13																	
14																	

Analysis: Submitted for laboratory analysis

BH/MW18-3

consulting engineers

DST Project No.	TS-SO-032782	Date	12 October 2018
Client	Power Market Real Estate Brokerage	Method	CME 75
Project	Phase II ESA	Diameter	50.8mm
Address	2582-2584, 2600, 2626 Bank Street, Ottawa, ON	Coordinates	5022444.1 m N, 449855 m E

Depth (m)	Elevation (m)	Water level (m)	Well construction	Depth (m) Elevation (m)	Symbol	Material Description	Sample ID	Sample Type	'N' Value / RQD	CCGD / PID Reading: CCGD	PID	Analysis (Submitted for laboratory analysis): PAHs	PHCs	Metals	VOCs	Others	Remarks
				0		SAND WITH GRAVEL - trace organics	SS1		14	0 ppm							
1							SS2		10	0 ppm							
				1.1		ORGANIC											
				1.2		PEAT - some silt, dark brown	SS3		5	0 ppm							
2							SS4		7	0 ppm							
				2.4		SANDY SILT - moist	SS5		24	0 ppm							
3				3.1		SAND - some gravel, medium grained, light brown	SS6		17	0 ppm							
				3.6		- interbedded with sandy clay layer	SS7		16	320 ppm							
4				4.3		- brown, dry	SS8		21	0 ppm							
5							SS9		28	80 ppm							
							SS10		18	0 ppm							
6							SS11		28	0 ppm							
7							SS12		15	0 ppm							

BH/MW18-3

DST consulting engineers

DST Project No.	TS-SO-032782	Date	12 October 2018
Client	Power Market Real Estate Brokerage	Method	CME 75
Project	Phase II ESA	Diameter	50.8mm
Address	2582-2584, 2600, 2626 Bank Street, Ottawa, ON	Coordinates	5022444.1 m N, 449855 m E

Depth (m)	Elevation (m)	Water level (m)	Well construction	*Depth (m)* Elevation (m)	Symbol	Material Description	Sample ID	Sample Type	'N' Value / *RQD*	CCGD / PID Reading: CCGD	CCGD / PID Reading: PID	Analysis (Submitted for laboratory analysis): PAHs	PHCs	Metals	VOCs	Others	Remarks
				7.3		SAND - some gravel, medium grained, light brown - moist	SS13		11	0 ppm							
8				*7.8*		- wet	SS14		13	260 ppm			✓		✓		Groundwater level at 8.37 mbgs
				8.4		- coarse grained											
9							SS15		18	0 ppm							
							SS16		32	5 ppm							
10							SS17			95 ppm							
						End of Borehole at 10.4 m.											
11																	
12																	
13																	
14																	

BH/MW18-4

consulting engineers

DST Project No.	TS-SO-032782	Date	03 October 2018
Client	Power Market Real Estate Brokerage	Method	CME 75
Project	Phase II ESA	Diameter	50.8mm
Address	2582-2584, 2600, 2626Bank Street, Ottawa, ON	Coordinates	5022309.4 m N, 449980.2 m E

Depth (m)	Elevation (m)	Water level (m)	Well construction	Depth (m) Elevation (m)	Symbol	Material Description	Sample ID	Sample Type	'N' Value / RQD	CCGD / PID Reading: CCGD	PID	Analysis (Submitted for laboratory analysis): PAHs	PHCs	Metals	VOCs	Others	Remarks
				0		TOPSOIL - sand and silt, brown	SS1		8	0 ppm							
				0.6		SILTY SAND - trace organics, brown	SS2		14	0 ppm							
1				1.2		SAND - some silt, trace organics, medium grained, brown	SS3		8	0 ppm							
2							SS4		15	0 ppm							
							SS5		13	0 ppm							
3				3		- moist											
				3.1		SILTY SAND - trace clay, wet	SS6		8	0 ppm							
				3.7		SANDY SILTY CLAY - trace coarse sand, trace clay, wet	SS7		11	0 ppm							
4							SS8		15	0 ppm						✓	
5						End of Borehole at 4.9 m.											
6																	
7																	

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 3 - SECTIONS FOR SLOPE STABILITY ANALYSIS

DRAWING PG5545-1 - TEST HOLE LOCATION PLAN

DRAWING PG5545-2 - LIMIT OF HAZARD LANDS

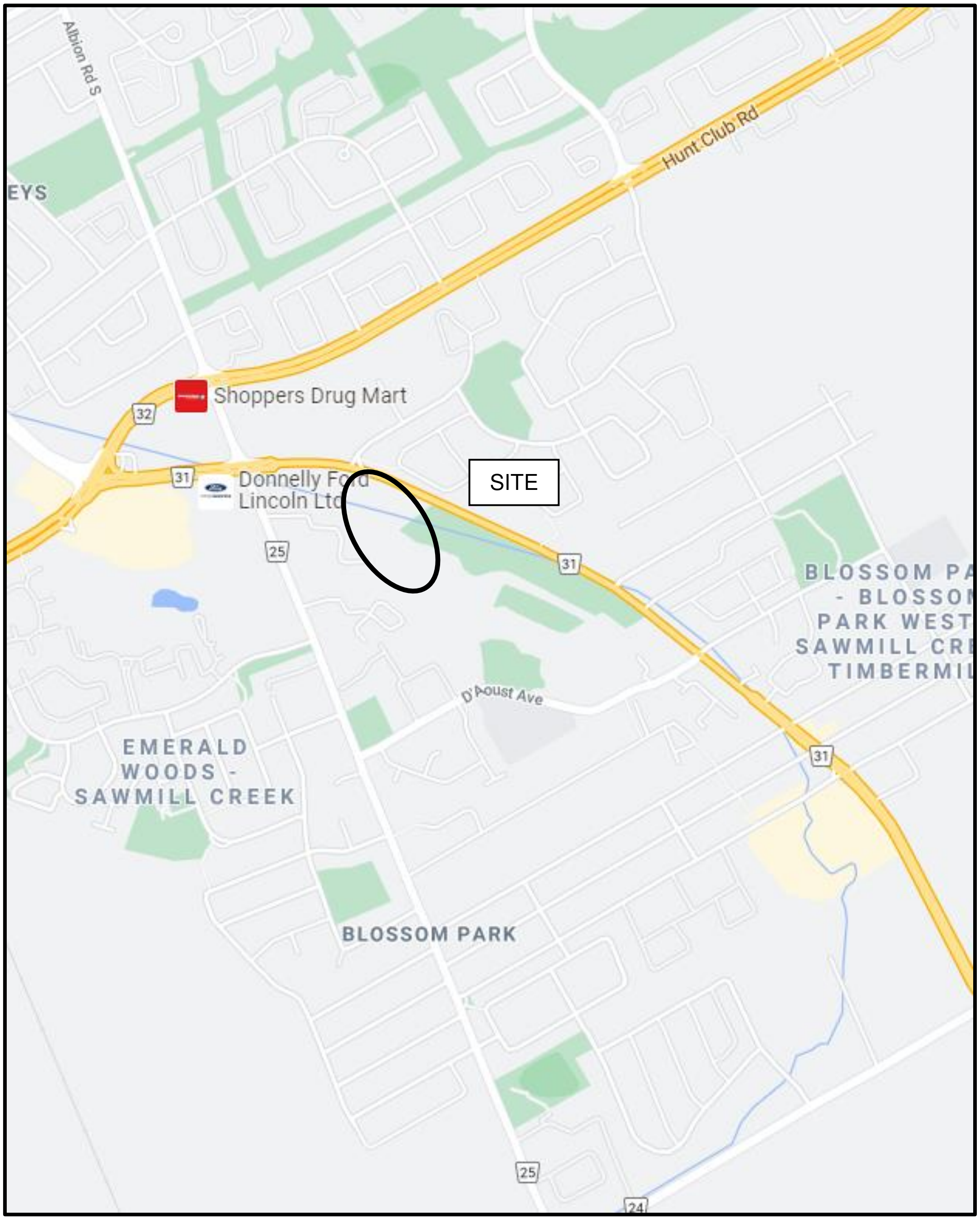

FIGURE 1

KEY PLAN

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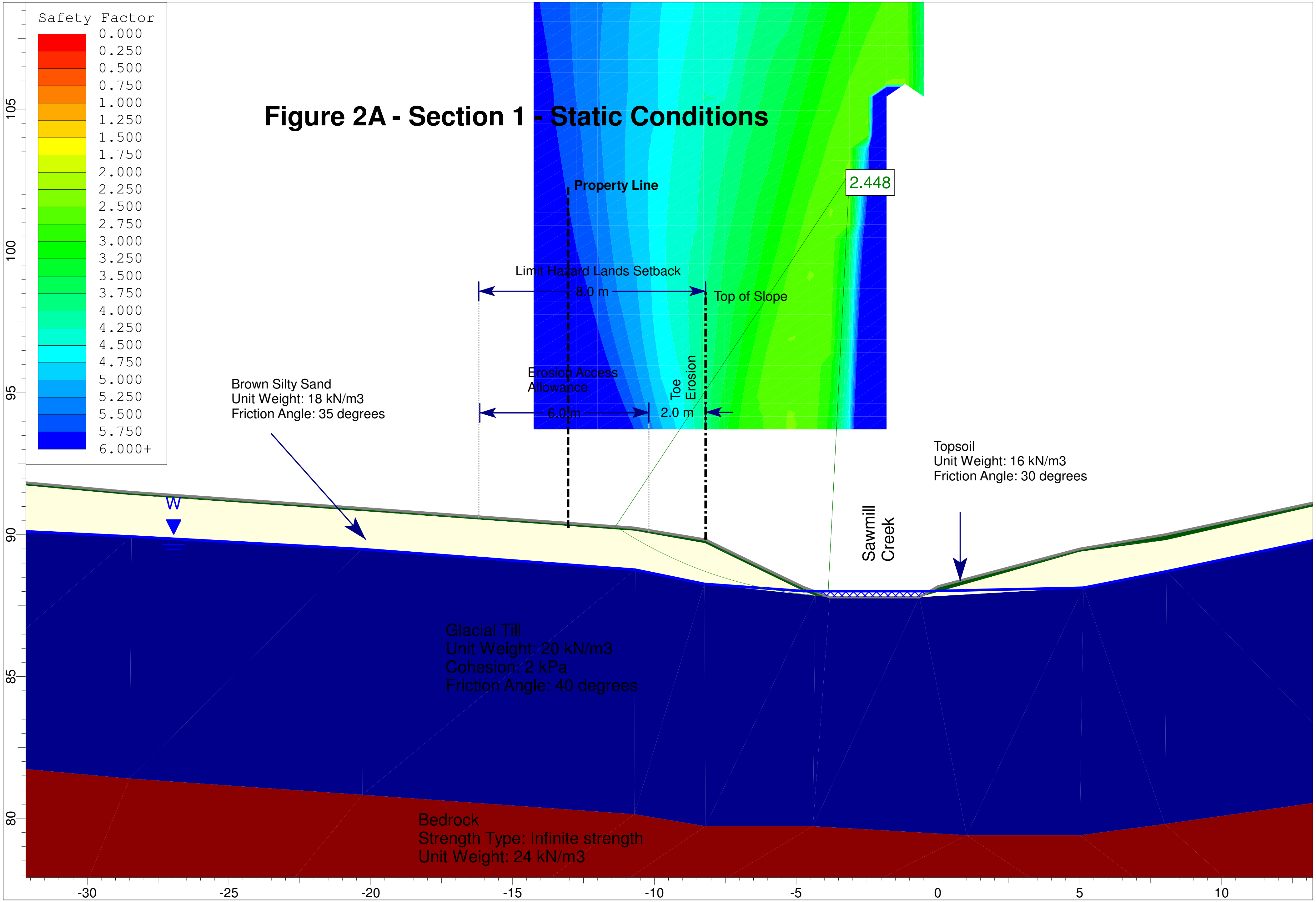
Figure 2A - Section 1 - Static Conditions



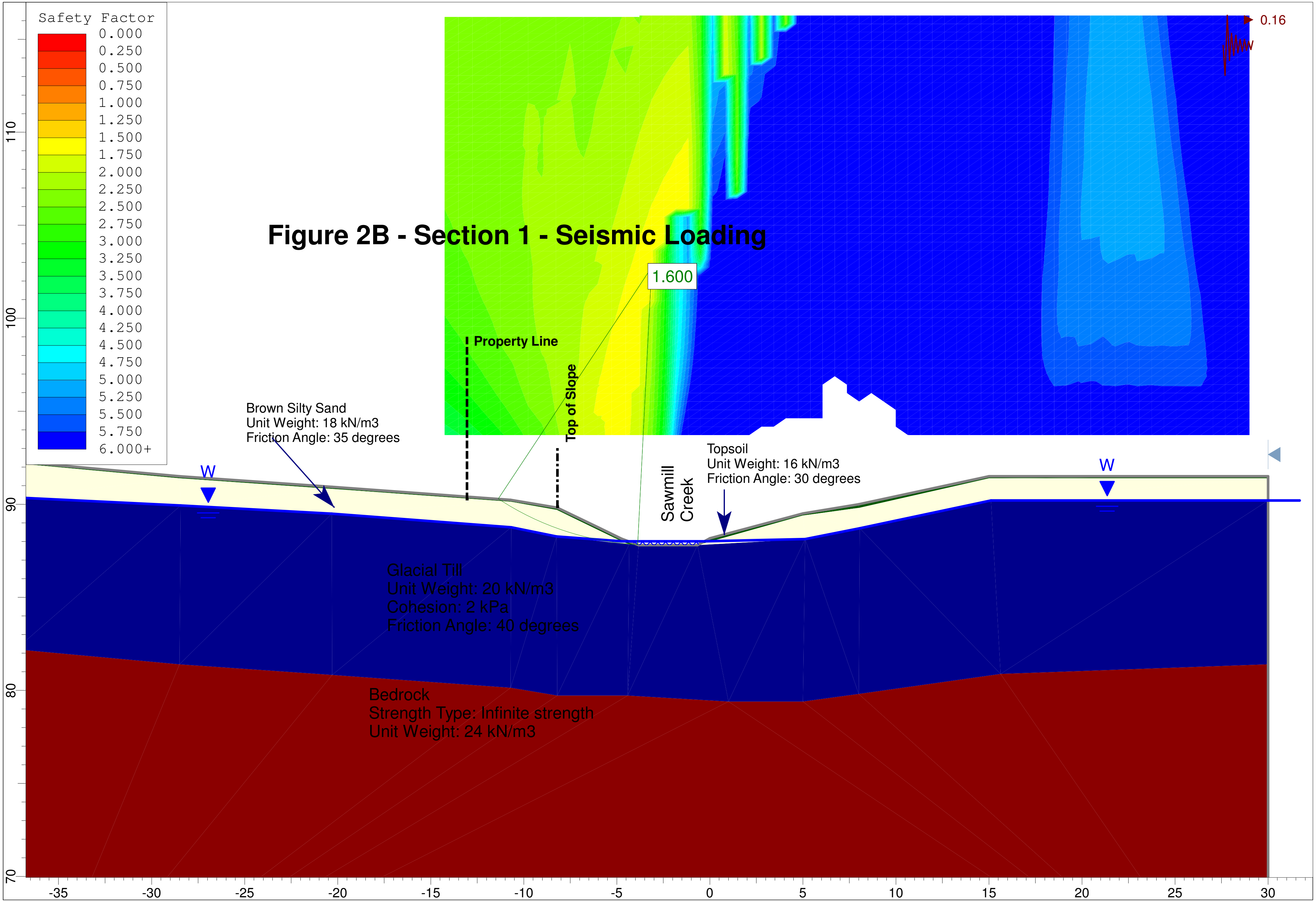
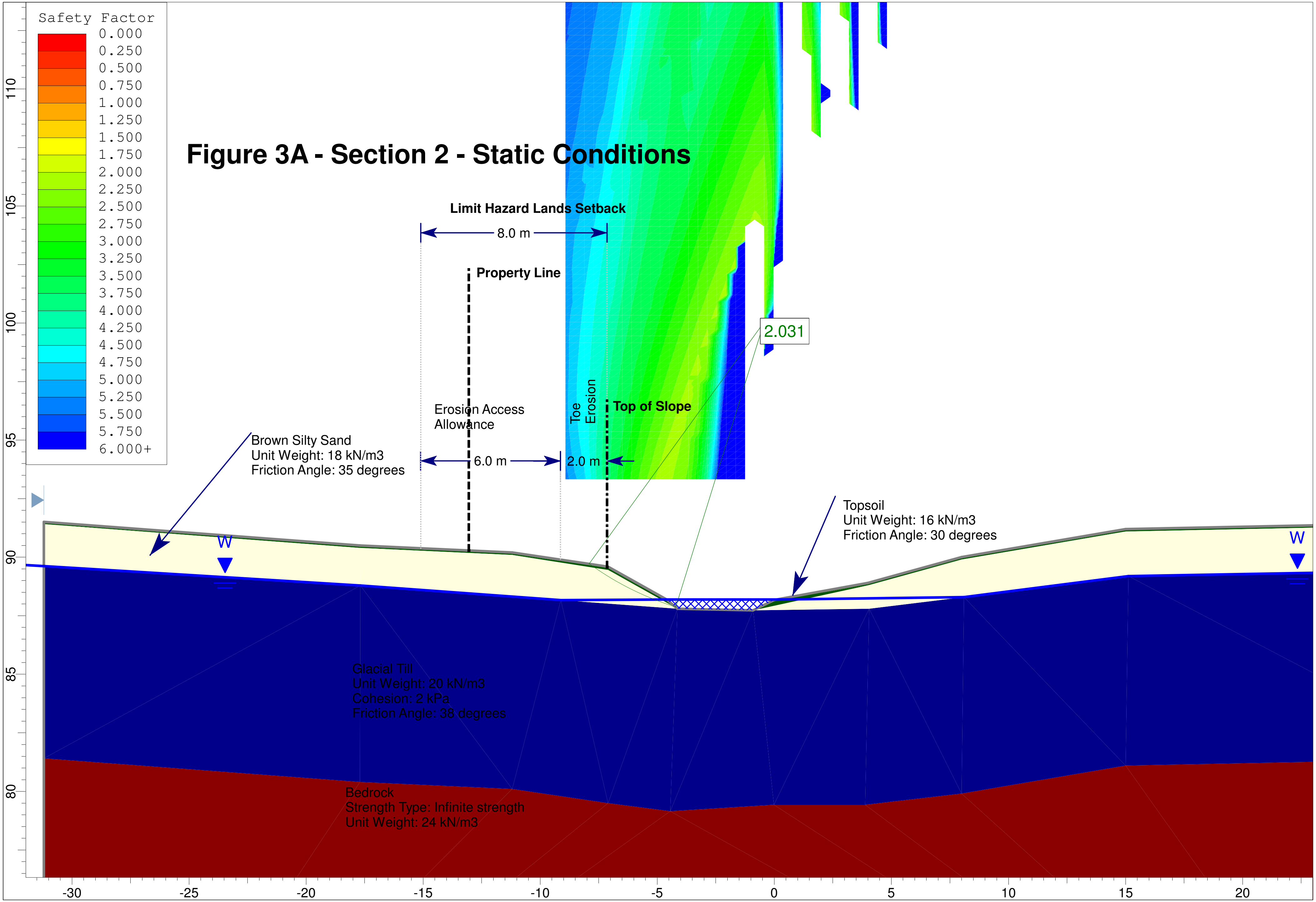


Figure 2B - Section 1 - Seismic Loading
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
1.600
Property Line
Top of Slope
Brown Silty Sand
Unit Weight: 18 kN/m3
Friction Angle: 35 degrees
Topsoil
Unit Weight: 16 kN/m3
Friction Angle: 30 degrees
Sawmill Creek
W
W
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 40 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 24 kN/m3
110
100
90
80
70
-35
-30
-25
-20
-15
-10
-5
0
5
10
15
20
25
30

Figure 3A - Section 2 - Static Conditions



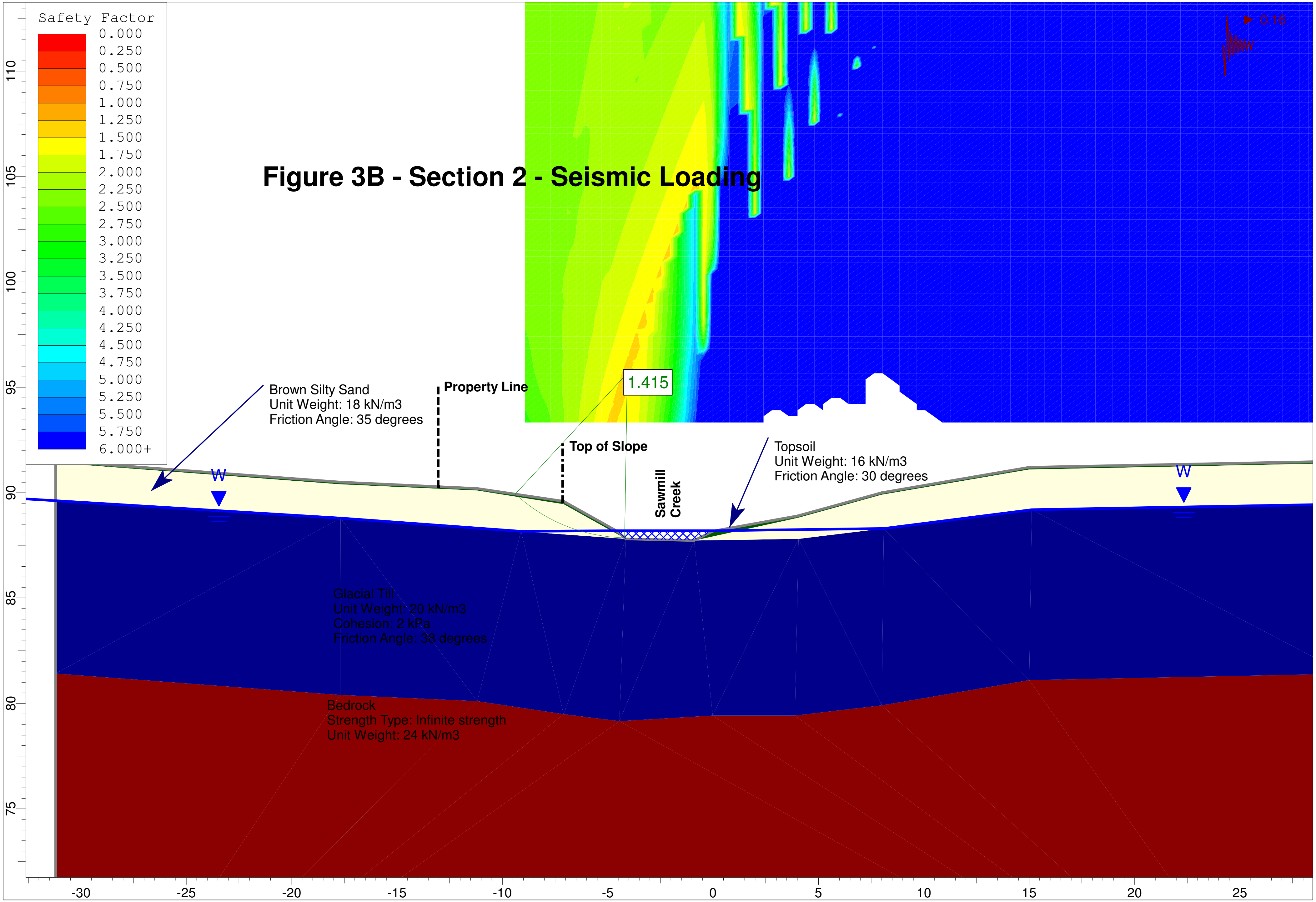
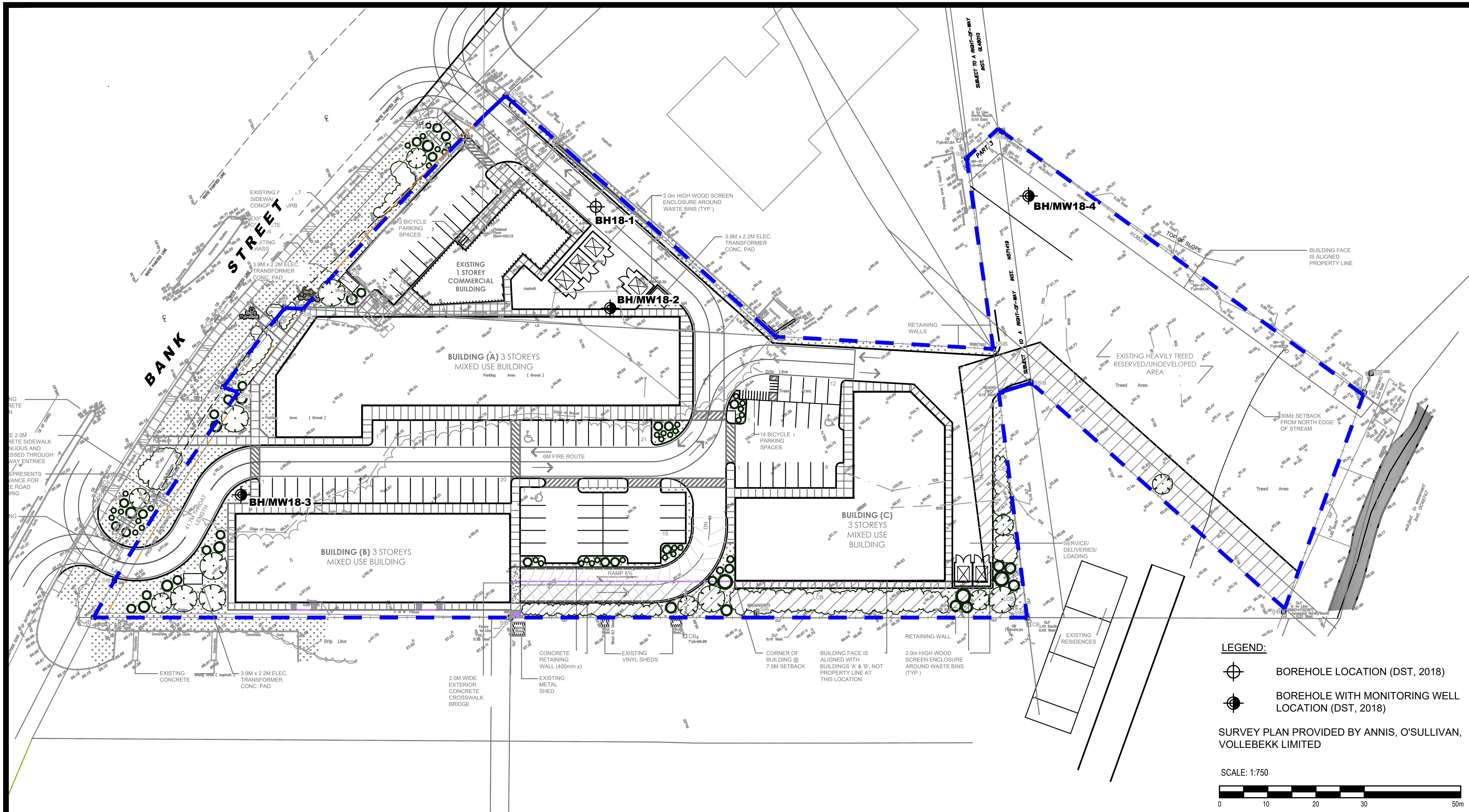


Figure 3B - Section 2 - Seismic Loading
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
1.415
Brown Silty Sand
Unit Weight: 18 kN/m3
Friction Angle: 35 degrees
Property Line
Top of Slope
Sawmill Creek
Topsoil
Unit Weight: 16 kN/m3
Friction Angle: 30 degrees
W
W
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 38 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 24 kN/m3
110
105
100
95
90
85
80
75
-30
-25
-20
-15
-10
-5
0
5
10
15
20
25



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NO.	REVISIONS	DATE	INITIAL
1	CONCEPTUAL SITE PLAN UPDATED	23/08/2021	MS

MR. NABIL ABDULLA
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT - 2582 TO 2626 BANK STREET
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 10/2020
Drawn by: YA	Report No.: PG5545-1
Checked by: MS	Dwg. No.: PG5545-1
Approved by: FA	Revision No.: 1

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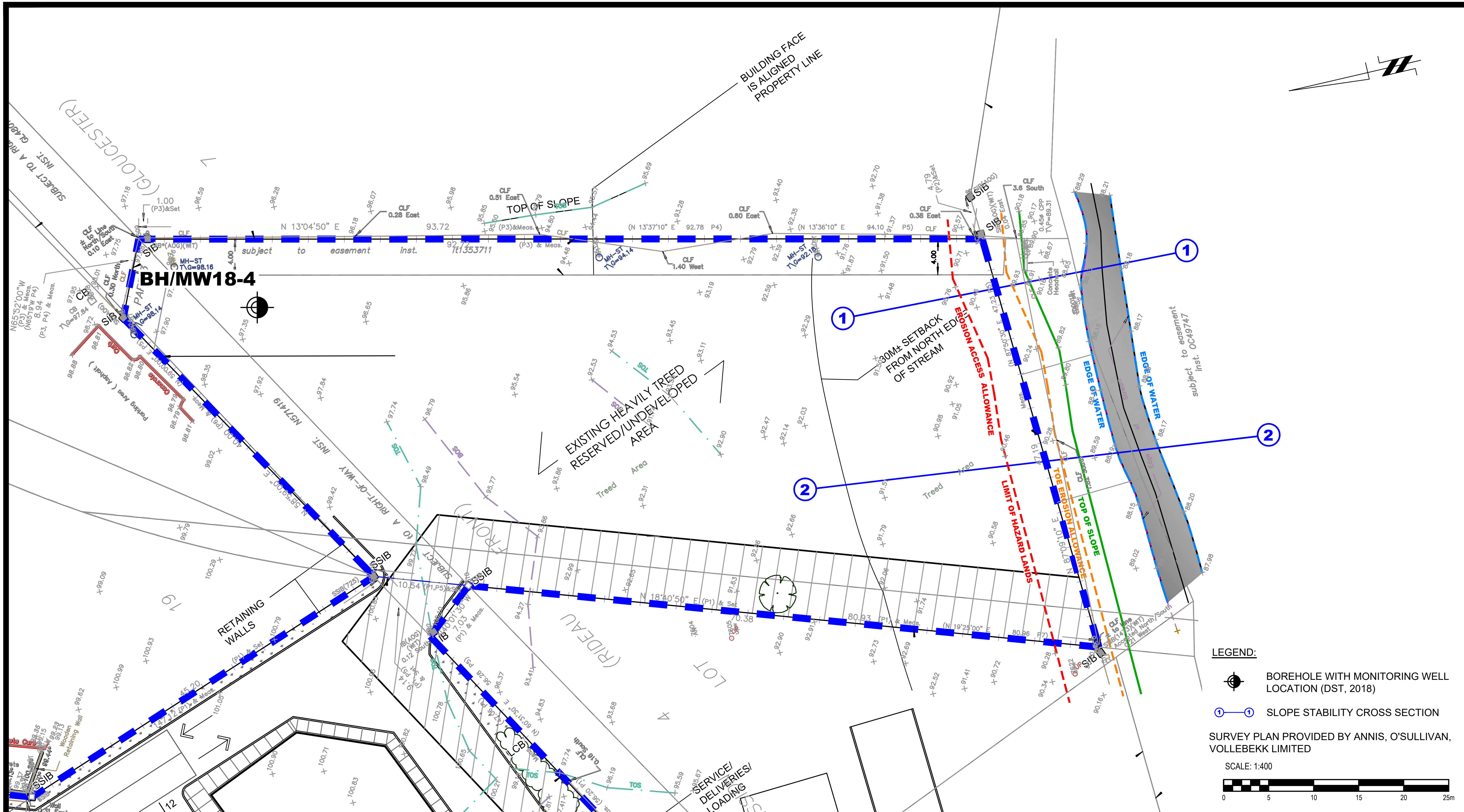

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154 Colonnade Road South
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MR. NABIL ABDULLA
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT - 2582 TO 2626 BANK STREET
OTTAWA, ONTARIO

Title:
LIMIT OF HAZARD LANDS

Scale: 1:400	Date: 11/2020		
Drawn by: NFRV	Report No.: PG5545-1		
Checked by: JV	Dwg. No.: PG5545-2		
Approved by: FA	Revision No.: 1		

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APPENDIX 3

SITE VISIT PHOTOS

HISTORICAL ARIAL PHOTOGRAPHS

Photographs from Site Visit – October 22, 2020

Photo 1: Photo of slope face and adjacent creek facing east and upstream of the creek. No signs of erosion or distress noted along the face of the slope.

Photo 2: Photo of slope face and adjacent creek facing west and downstream of the creek. No signs of erosion or distress notes along the face of the slope.

Photo 3: Photo of cursory observation of vegetation (peat moss and topsoil) and brush located along the face and top of the slope.

Photo 4: Photo of cursory observation of vegetation and brush located along the face and top of the slope.

Photo 5: Aerial photo taken in 1965 of the south portion of the site. A structure can be observed located in the area slopes and a small crater were noted during our site visit.

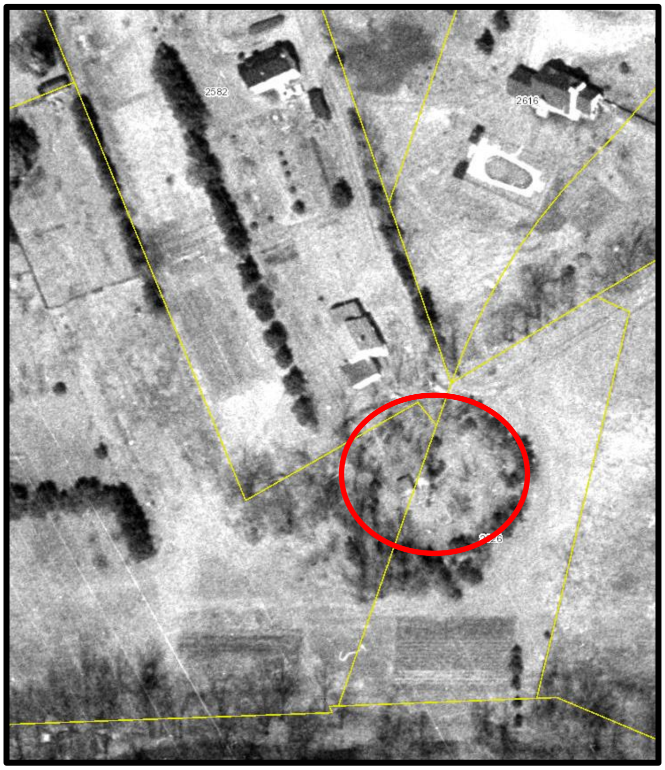

Photo 6: Aerial photo taken in 1976 of the south portion of the site. A building was removed and new residential dwelling can be observed to the north.