Geotechnical Investigation

Proposed Development

320 Bren-Maur Road
Ottawa, Ontario

Prepared for Uniform Developments

Report PG7253 – 1 dated September 16, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Uniform Developments to conduct a geotechnical investigation for the proposed development to be located at 320 Bren-Maur Road in the City of Ottawa Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

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 work of this present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Details for the proposed development were not available during the preparation of this report. However, it is anticipated that development will consist of a multi-storey building with 1 below-grade level

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on September 3, 2024, and consisted of a total of 2 boreholes sampled to a maximum depth of 10.5 m below the ground surface throughout the subject site.

The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access. The approximate locations of the boreholes are shown on Drawing PG7253-1 - Test Hole Location Plan included in Appendix 2.

All boreholes were advanced using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes either by sampling directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 2-24. The DCPT testing consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the 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 standpipe piezometers were installed as part of the current investigation in boreholes to permit monitoring of the groundwater levels after the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The borehole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS referenced to a geodetic datum. The locations of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG7253-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

Soil samples were submitted for analytical testing from previous geotechnical investigations 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 sample. The results are presented in Appendix 1 and are discussed further in Section 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by 2 existing residential dwellings with associated laneways and landscaped areas. The subject site is bordered to the north by Bren-Maur Road, to the east by a stormwater management pond, to the west by Longfields Drive, and to the south by the Jock River.

The ground surface across the majority of the site is relatively level and at grade with the surrounding roadways at approximate geodetic elevation 90 m. However, the eastern portion of the site slope downward to geodetic elevation 86 m, along the shore of the Jock River.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile was observed to consist of topsoil and fill underlain by a deposit of silty clay and/or glacial till.

The fill material consists of brown silty sand with gravel, clay, organics, cobbles and boulders. The thickness of the fill layer was observed to range between approximately 0.7 m and 2.9 m.

A deposit of very stiff to stiff, brown silty clay was encountered underlying the fill at borehole BH 2-24, extending to approximately 3.2 m below ground surface.

A glacial till deposit was encountered below the fill or silty clay layer. The glacial till was generally observed to consist of compact to very dense, brown to grey silty sand to sandy silt with gravel, trace clay, cobbles and boulders.

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

Bedrock

Based on geological mapping, the bedrock at the subject site consists of interbedded sandstone and dolomite of the March formation with an overburden drift thickness ranging between 10 and 15 m.

4.3 Groundwater

Groundwater level readings for the current investigation were measured on September 9, 2024, and are presented in Table 1 below and on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-24	90.78	3.29	87.49	Sept 09, 2024
BH 2-24	90.77	4.22	86.55	Sept 09, 2024

Note:
- The ground surface elevations are referenced to a geodetic datum.
- * Borehole with groundwater monitoring well

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is anticipated to be 3 to 4 m below the ground surface.

However, groundwater levels are subject to seasonal fluctuations and could vary during the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for proposed development. It is recommended that a proposed building be founded on conventional spread footings bearing on the undisturbed, hard to very stiff silty clay or compact to very dense glacial till.

Due to the presence of a silty clay deposit throughout portions of the site, permissible grade raise restrictions have been provided.

A slope stability assessment has been completed to evaluate the stability of the existing slope at the site, and to provide a Limit of Hazard Lands setback. This is discussed further in Section 6.9.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by Paterson personnel at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low-frequency vibrations, the maximum allowable peak particle velocity is less than that for high-frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill placed for grading beneath the 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 a maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building areas 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 can 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.

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 98% of their respective SPMDD.

5.3 Foundation Design

Conventional Spread Footings

Conventional spread footings placed on an undisturbed, very stiff silty clay or undisturbed, compact to dense glacial till, can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

An undisturbed soil-bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

The bearing resistance value at SLS 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. Adequate lateral support is provided to a soil-bearing medium 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.

Permissible Grade Raise

Due to the presence of the silty clay deposit throughout the western portion of the subject site, a permissible grade raise restriction of **1.5 m** is recommended for grading at the subject site where silty clay is present. **However, it should be noted that grade raises are not permitted within the Limit of Hazard Lands.**

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** for the foundations considered. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2020 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 from within the footprints of the proposed buildings, the undisturbed existing fill or native soil will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction.

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone.

For structures with slab-on-grade construction, the upper 300 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprints of the proposed buildings should be placed in a maximum of 300 mm thick loose lifts and compacted to a minimum of 98% of its SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the proposed structure's basement walls. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3.

The foundation wall is anticipated to be provided with a perimeter drainage system; therefore, the retained soils should be considered drained.

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 $0.375 \cdot a \cdot H^2/g$ 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 $P_o = 0.5\ K_o \cdot \gamma \cdot H^2$, where K = 0.5 for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

For design purposes, the following pavement structures, presented below, are recommended for the design of the car parking areas and local roadways.

Table 2 - Recommended Pavement Structure – 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 – Local Residential Roadways

Thickness mm	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in a maximum of 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for any proposed buildings with below-grade space. The system, where considered, should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is 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.

However, if a proposed building at this site contains several below-grade levels which extend below the water level of the adjacent Jock River, then waterproofing of the foundation walls would also be required.

Foundation Backfill

For proposed buildings with below-grade space, the 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 Delta Drain 6000 or equivalent) is installed over the exterior of the perimeter foundation walls and connected to the perimeter drainage system.

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. Generally, a minimum of 1.5 m thick soil cover (or an equivalent combination of soil cover and foundation insulation) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. It is anticipated that sufficient space will be available for the great part of the excavations to be undertaken by open-cut methods (i.e., 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. A flatter slope is required for excavation below the groundwater level. The subsoil at this site appeared to be mainly a Type 2 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 over 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.

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 crushed stone 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 a maximum of 225 mm thick lifts compacted to 99% of the material’s standard Proctor maximum dry density.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The trench backfill should be placed in a maximum of 225 mm thick loose lifts and compacted to a minimum of 98% of the material’s SPMDD. All cobbles larger than 200 mm in the longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

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.

Groundwater Control for Building Construction

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application 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 using 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 the founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or 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 the subject site, whereas the resistivity is indicative of a non-aggressive to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Considerations

Due to the presence of the silty clay deposit in the western portion of the site, the location of street trees will be governed by the potential for soil volume change where trees and houses are located above a silty clay deposit.

Based on the current guidelines, large trees (mature height over 14 m) can be planted within the western portion of the subject site provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g., in a park or other green space). Tree planting setback limits may be reduced to **4.5 m** for small (mature tree height up to 7.5 m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the conditions noted below are met:

- ❑ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the center of the tree trunk and verified by means of the Grading Plan.

- ❑ A small tree must be provided with a minimum of 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

- ❑ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

- ❑ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

6.9 Slope Stability Analysis

Paterson completed a field review of the slope along the eastern portion of the site, alongside the Jock River as part of the current investigation.

The field review generally consisted of observing surface conditions along the length of the slope face and watercourse identifying the presence of vegetation, erosion and other features associated with slope stability.

Water levels and flow within the watercourses were reviewed generally, if present, including identifying signs of recent high-water marks or other signs of previous rises in the water levels. The top of slope alignment was determined in the field by Paterson personnel based on our field observations and recorded using a high-precision handheld GPS unit.

Topographic surface elevations were measured at select cross-sections to analyze slope stability using SLIDE, a computer program for two-dimensional slope stability analysis. Overall, a total of one (1) slope cross-section throughout the above-noted locations was analyzed as part of the slope stability analysis.

Based on the results of our field observations and slope stability analysis, a Limit of Hazard Lands was assigned from the top of the slope for the above-noted cross-section A-A. The cross-section locations and associated Limit of Hazard Lands setbacks are presented on Drawing PG7253-1 – Test Hole Location Plan in Appendix 2.

Field Observations

The slope observed at the eastern portion of the site was observed to have an approximate incline ranging between 2H:1 to 3H:1V and surfaced with mature vegetation and small trees. The slope was noted to range between 4 m and 6 m in height. Rip Rap was observed along the toe of the slope and extending into the water course bordering the Jock River. No signs of erosion, distress or sloughing were observed throughout the slope surface at the time of our field review.

Slope Stability Analysis

The slope stability analysis was modelled in SLIDE, a computer program which permits a two-dimensional slope stability analysis calculating several 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 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 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.

One (1) slope cross-section (Section A) were studied as the worst case scenario. The cross-section location is presented in Drawing PG7253-1 – Test Hole Location Plan in Appendix 2. It should be noted that details of the slope height and slope angle at the cross-section locations are presented in Figures 2 through 3 in Appendix 2, based on the topographic data obtained during the field investigation, as well as the available topographic data.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the site investigation. The effective strength soil parameters used for static analysis are presented in Table 4.

Table 4 – Effective Stress Soil Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill	19	34	-
Brown Silty Clay	17	33	7
Glacial Till	20	36	0
Bedrock	22	-	-

The total strength parameters for seismic analysis were chosen based on the subsurface conditions observed in the test holes, and our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 5 on the next page.

Table 5 – Total Stress Soil Parameters (Seismic Analysis)			
Soil Layer	**Unit Weight (kN/m^3)**	**Friction Angle (degrees)**	**Cohesion (kPa)**
Fill	19	34	-
Brown Silty Clay	17	-	150
Glacial Till	20	36	0
Bedrock	22	-	-

Stable Slope Allowance

The static analysis results for slope cross-section A-A are presented in Figure 2 provided in Appendix 2. The factor of safety of less than 1.5 was noted, therefore, a slope stability setback would be required if the existing slope was not re-graded as part of the proposed development. A stable slope setback of 1.5 m, for slope cross-section A-A, would be required if the existing slope is not modified.

The results of the analyses with seismic loading are shown in Figure 3, presented in Appendix 2. The factor of safety for the slopes was noted to be less than 1.1 therefore, a slope stability setback would be required if the existing slope was not re-graded as part of the proposed development. A stable slope setback of 2.6 m, for slope cross-section A-A, would be required if the existing slope is not modified.

Toe Erosion and Erosion Access Allowance

The slope was generally observed to be covered by heavy vegetation and mature trees along the slope, and rip rap stone along the slope toe. No erosion or distress associated with erosion was observed at the time of our review. Based on current guidelines, since the edge of the watercourse contains rip rap systems for toe erosion protection, a toe erosion allowance setback was not considered applicable. As per the City of Ottawa slope stability guidelines an erosion access allowance of 6 m was applied.

Limit of Hazard Lands

The results of the slope stability assessment indicate that the Limit of Hazard Lands setback of **8.6 m** measured from the top of the slope, should be provided for any proposed structures at the subject site. The Limit of Hazard Lands setback line is shown on the attached Drawing PG7253-1 – Test Hole Location Plan.

Additional Considerations

Grade raises are not recommended throughout the sloped portions of the subject site.

It is recommended that the existing vegetation and mature trees not be removed from the slope faces as the presence of the vegetation reduces surficial erosion activities.

7.0 Recommendations

For the foundation design data provided herein to be applicable a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

- ❑ Review detailed Grading and Servicing Plans from a geotechnical perspective.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Observation of the placement of the foundation insulation, if applicable.
- ❑ 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 and follow-up field density tests to determine the level of compaction achieved.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson.

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

8.0 Statement of Limitations

The recommendations provided herein 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 Uniform Developments, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Sept. 16, 2024
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Otillia McLaughlin, B.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❏ Uniform Developments. (e-mail copy)
- ❏ Paterson Group Inc (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

320 Bren-Maur Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 364953.50 **NORTHING:** 5013898.67 **ELEVATION:** 90.78

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 03, 2024

FILE NO. : PG7253

HOLE NO. : BH 1-24

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
TOPSOIL and organics (0.20m [90.58m])		AU 1			
FILL: Brown silty sand, with gravel, trace organics	1	SS 2	50	6-10-8-8 18	
	2	SS 3	25	6-7-4-5 11	
(2.97m [87.81m])		SS 4	17	6-10-8-6 18	
GLACIAL TILL: Compact to dense, brown silty fine sand to sandy silt, with gravel, trace cobbles and boulders	3	SS 5	67	4-9-6-5 15	
	4	SS 6	17	4-50-/-/ 50/0.1	
	5	SS 7	58	7-6-4-13 10	
- Grey by 5.33 m depth		SS 8	42	13-7-5-4 12	
	6	SS 9	17	2-4-6-8 10	
	7	SS 10	17	5-10-11-10 21	
(8.23m [82.55m])	8	SS 11	83	3-18-9-11 27	
End of Borehole					
(GWL at 3.29 m depth - September 10, 2024)	9				

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

PIEZOMETER CONSTRUCTION: 3.3 m ▼ 2024-09-10

ELEVATION (m): 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80, 79, 78

DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13

P:/AutoCAD Drawings/Test Hole Data Files/PG72xx/PG7253/data.sqlite 2024-09-16, 15:54 Paterson_Template AE

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

320 Bren-Maur Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 364945.77 **NORTHING:** 5013946.08 **ELEVATION:** 90.77

PROJECT: Proposed Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: September 03, 2024

FILE NO. : PG7253

HOLE NO. : BH 2-24

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD
GROUND SURFACE	0			
TOPSOIL and organics — 0.18m [90.59m]		AU 1		
FILL: Brown silty sand, with gravel, some clay, trace topsoil and organics — 0.69m [90.08m]	1	SS 2	58	4-6-7-8 13
Very stiff, brown **SILTY CLAY** / **CLAYEY SILT**	2	SS 3	100	4-6-7-7 13
		SS 4	100	5-5-5-5 10
3.20m [87.57m]	3			
GLACIAL TILL: Dense, brown silty sand to sandy silt, with gravel, trace clay cobbles and boulders		SS 5	83	5-8-5-26 13
	4	SS 6	83	7-18-23-17 41
- Grey by 4.88 m depth	5	SS 7	67	8-14-14-16 28
		SS 8	8	12-50-/-/ 50/0.03
	6	SS 9	58	3-3-5-5 8
	7	SS 10	0	5-9-25-18 34
	8	SS 11	33	5-11-12-8 23
- Sand content increasing by 8.38 m depth		SS 12	58	4-7-8-11 15
	9	SS 13		31-43-38-26 81
	10	SS 14		28-31-36-46 67
10.52m [80.25m]				
Dynamic Cone Penetration Test commenced at 10.52 m depth — 10.59m [80.18m]	11			
End of Borehole	12			
Practical refusal to DCPT at 10.59 m depth (GWL at 4.22 m depth - September 10, 2024)	13			

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = $D60 / D10$

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

CONSOLIDATION TEST

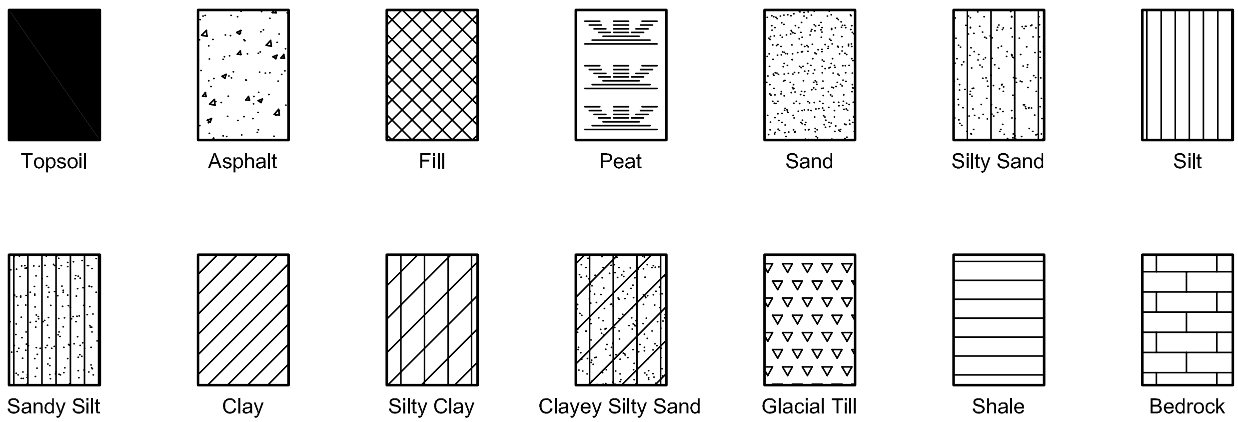
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

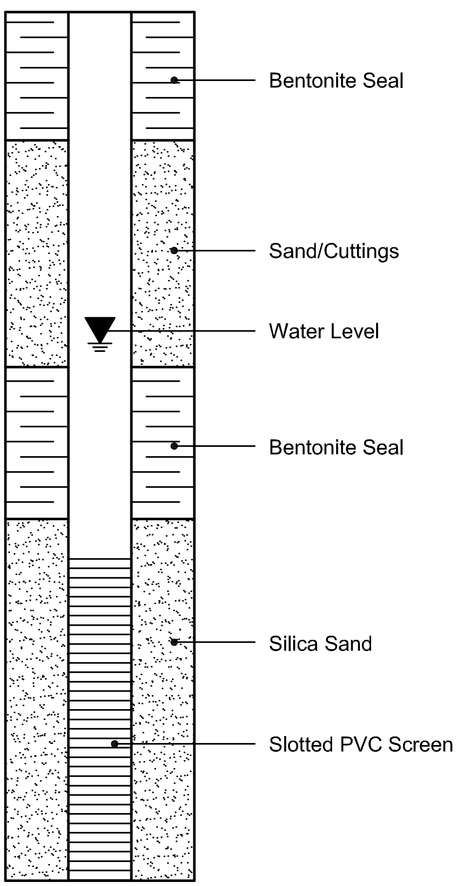
SYMBOLS AND TERMS (continued)

STRATA PLOT

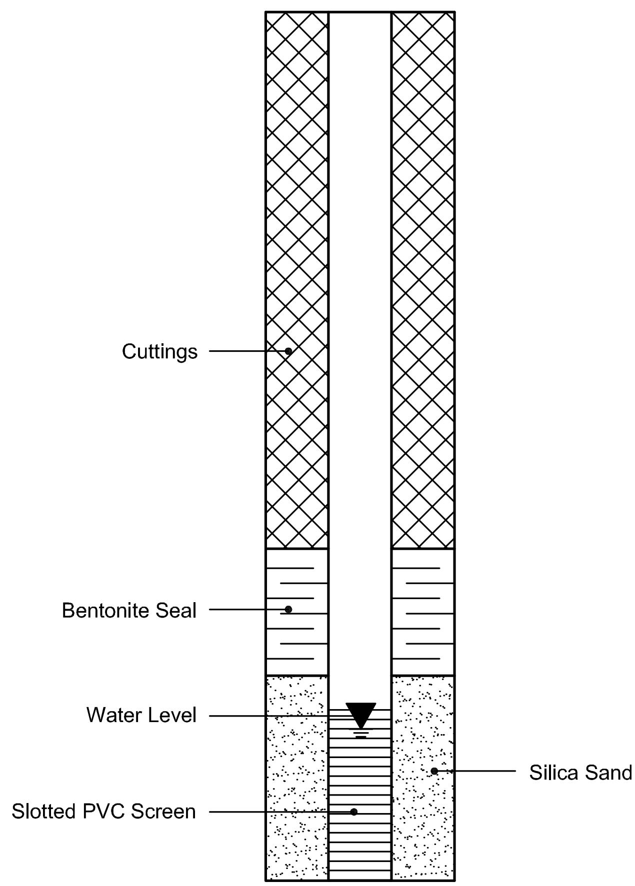


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 2437246

Certificate of Analysis

Report Date: 13-Sep-2024

Client: Paterson Group Consulting Engineers (Ottawa)

Order Date: 10-Sep-2024

Client PO:

Project Description: PG7253

Client ID:		BH2-24-SS3-5' - 7' / 320 Bren-Maur	-	-	-		
Sample Date:		03-Sep-24 09:00	-	-	-	-	-
Sample ID:		2437246-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	74.3	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.06	-	-	-	-	-
Resistivity	0.1 Ohm.m	86.1	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	26	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 TO 3 – SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG7253-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP

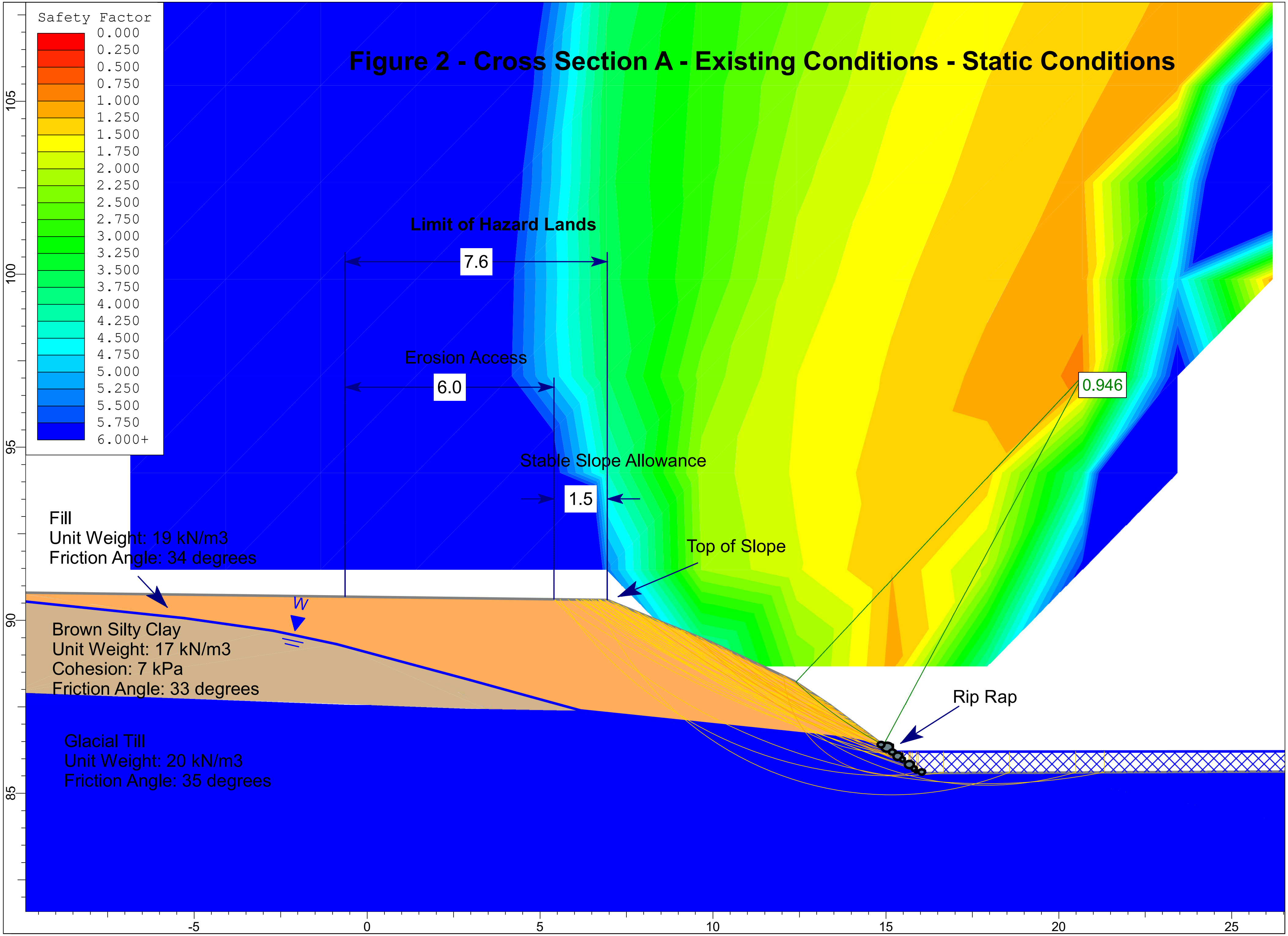


Figure 2 - Cross Section A - Existing Conditions - Static Conditions
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
7.6
Erosion Access
6.0
Stable Slope Allowance
1.5
Top of Slope
0.946
Rip Rap
Fill
Unit Weight: 19 kN/m3
Friction Angle: 34 degrees
W
Brown Silty Clay
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
Glacial Till
Unit Weight: 20 kN/m3
Friction Angle: 35 degrees
105
100
95
90
85
-5
0
5
10
15
20
25

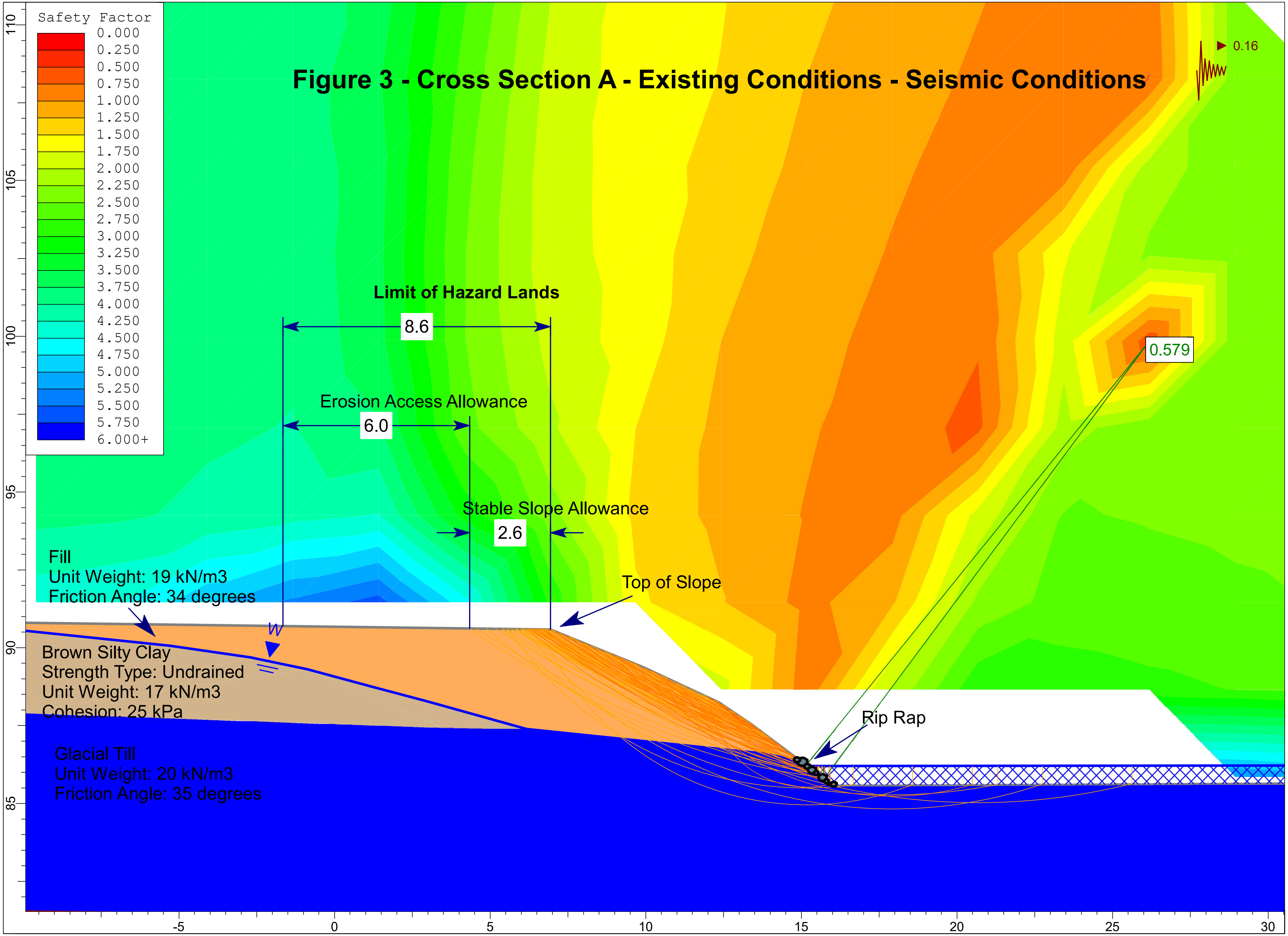
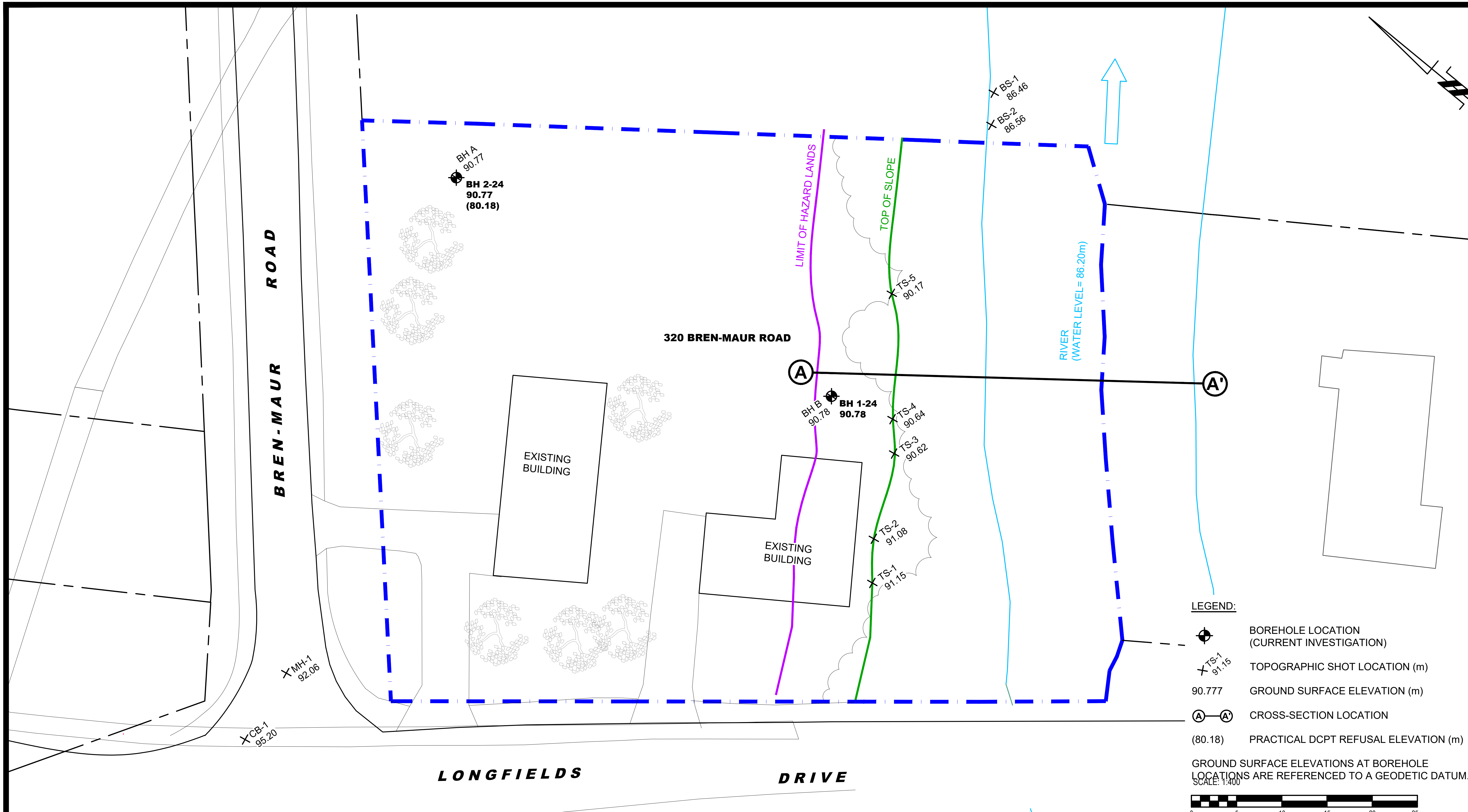


Figure 3 - Cross Section A - Existing Conditions - Seismic Conditions
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
0.579
Limit of Hazard Lands
8.6
Erosion Access Allowance
6.0
Stable Slope Allowance
2.6
Top of Slope
Rip Rap
Fill
Unit Weight: 19 kN/m3
Friction Angle: 34 degrees
Brown Silty Clay
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion: 25 kPa
Glacial Till
Unit Weight: 20 kN/m3
Friction Angle: 35 degrees
W
110
105
100
95
90
85
-5
0
5
10
15
20
25
30



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9 AURIGA DRIVE
OTTAWA, ON
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NO.	REVISIONS	DATE	INITIAL

UNIFORM DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
320 BREN-MAUR ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 09/2024
Drawn by: YA	Report No.: PG7253-1
Checked by: OM	Dwg. No.: PG7253-1
Approved by: SD	Revision No.:

11x17

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