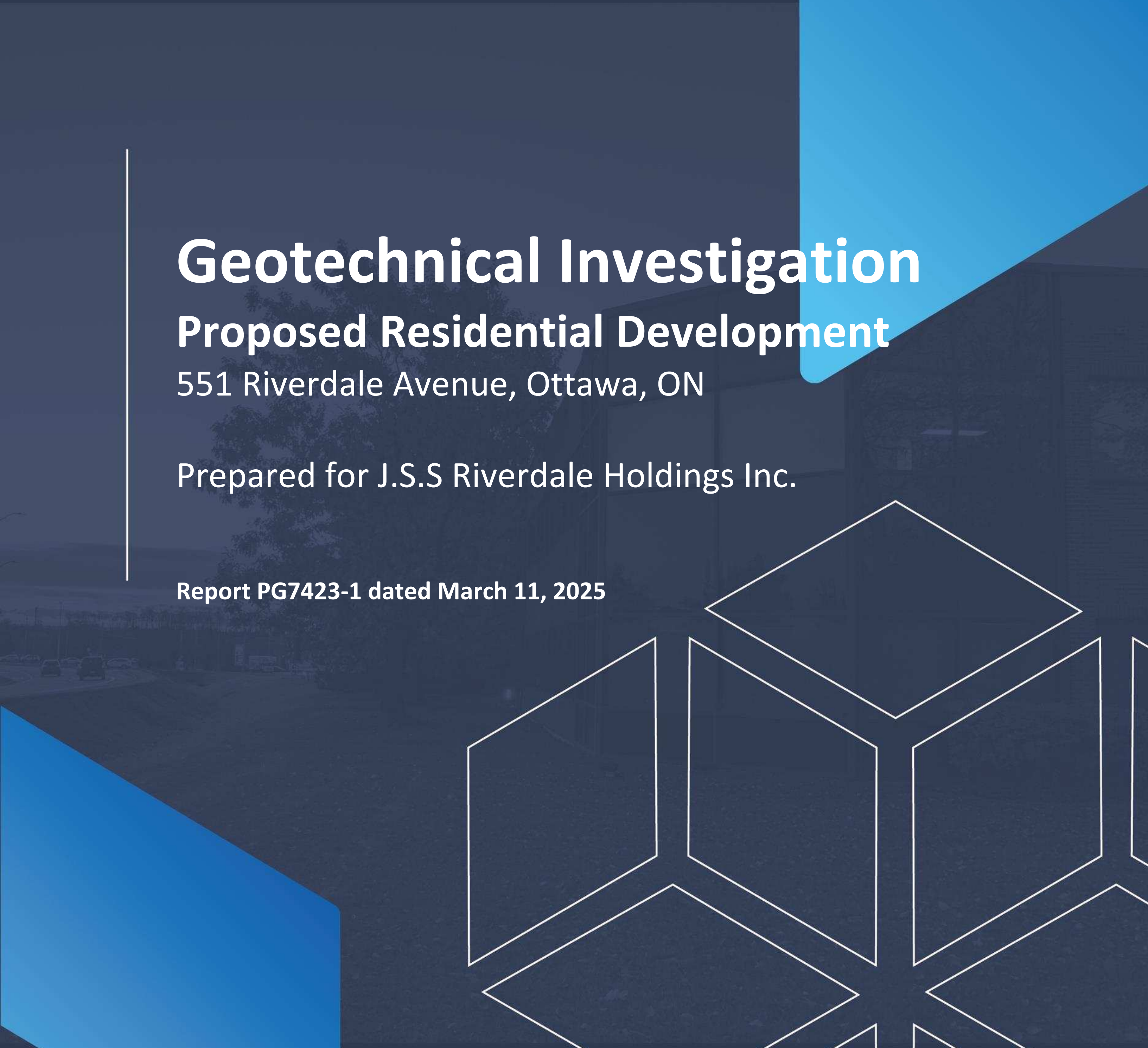
Geotechnical Investigation

Proposed Residential Development

551 Riverdale Avenue, Ottawa, ON

Prepared for J.S.S Riverdale Holdings Inc.

Report PG7423-1 dated March 11, 2025



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1.0 Introduction

Paterson Group (Paterson) was commissioned by J.S.S Riverdale Holdings Inc. to conduct a geotechnical investigation for the proposed residential development to be located at 551 Riverdale Avenue in the City of Ottawa (reference should be made to Figure 1 – Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test holes.

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

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

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

2.0 Proposed Development

The details of the proposed development are unknown at the time of preparing this report. However, based on preliminary information provided by the client, the proposed development is expected to consist of a mid-rise residential building with one underground level.

Associated access roads, landscaped areas, loading zones, and asphalt-surfaced parking areas will also be included in the development. It is anticipated that the subject site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on January 28, 2025, and February 10, and 11, 2025, and consisted of advancing a total of three (3) boreholes to a maximum depth of 5.9 m below the existing ground surface and a total of three (3) hand auger test holes advanced to a maximum depth of 0.9 m below the existing ground surface.

The test hole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration existing site features and underground utilities. The test hole locations are presented on Drawing PG7423-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were advanced with a low-clearance track-mounted drill rig operated by a two-person crew while the hand auger test holes were excavated using a stainless-steel hand auger. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The drilling procedure consisted of augering to the required depth at the selected location, sampling, and testing the overburden.

Sampling and In Situ Testing

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

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

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

Groundwater

Monitoring wells were installed in three (3) boreholes to allow groundwater level monitoring. The groundwater level readings were obtained after a suitable stabilization period subsequent to the completion of the field investigation.

The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ Up to 1.5 m of slotted 51 mm diameter PVC screen at base the base of the boreholes.
- ☐ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No.3 silica sand backfill within annular space around screen.
- ☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high-precision handheld GPS and referenced to a geodetic datum. In addition, a topographic survey was conducted on January 28, 2025, for the purpose of assessing slope stability along the Rideau River located on the east side of the subject site.

The locations of the test holes, the ground surface elevation at each test hole location, and the topographic survey are presented on Drawing PG7423-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

Sample Storage

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by two-storey residential dwellings with asphalt-surfaced parking areas, landscaped areas, access roadways, walkways, and mature trees.

The site is bordered by a residential property to the north, by sloped, tree-covered lands followed by Rideau River to the east, by tree-covered lands to the south, and by residential properties and Riverdale Avenue to the west.

The ground surface elevation across the subject site is relatively flat with an approximate geodetic elevation between 58.5 and 59.2 m and approximately at grade with surrounding properties and streets.

4.2 Subsurface Profile

Generally, the subsurface soil profile encountered at the test hole locations consists of a thin layer of asphalt followed by fill extending to a maximum depth of 2.1 m below the existing grade, underlain by glacial till deposit. The glacial till deposit was generally observed to consist of undisturbed, very dense to dense, brown silty sand or silty clay with variable amounts of gravel, cobbles, and boulders.

Further, an undisturbed, compact, brown clayey silt layer was encountered only in BH 3-25 at a depth ranging from 1.8 to 2.1 m below the existing ground surface. The clayey silt layer is underlain by an undisturbed, compact, brown silty sand to a depth of 3.0 m below the existing ground surface at BH 3-25.

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

Bedrock

Based on available geological mapping, the bedrock in the subject area is part of the Billings formation, which consists of interbedded shale with an overburden drift thickness ranging between 5 to 10 m depth.

4.3 Groundwater

Groundwater levels were recorded at each test hole location and are presented in Table 1 below. The groundwater level readings are also presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-25	59.24	3.56	55.68	February 19, 2025
BH 2A-25	58.89	3.11	55.78	
BH 3-25	58.45	N/A	N/A	

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

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. The long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at a depth of approximately **3.0 to 4.0 m** below the existing ground surface.

Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is anticipated that the future residential building will be founded over conventional shallow footings placed on undisturbed, compact, brown clayey silt, compact, brown silty sand, very dense to dense, glacial till, or an approved engineered fill placed on an undisturbed in-situ soil bearing surface.

It is anticipated that the removal of large boulders will be required for building construction and servicing installation. Therefore, the contractor should be prepared for bedrock removal and the presence of large boulders within the subject site.

Due to the absence of the sensitive silty clay deposit, the proposed development will not be subjected to grade raise restrictions.

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

5.2 Grading and Preparation

Stripping Depth

Asphalt, topsoil, construction debris, and deleterious fill, such as those containing organic materials, should be removed from within the perimeter of the proposed buildings and from under paved areas, pipe bedding or other settlement sensitive structures.

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

Where loose or disturbed native silt or silty sand is encountered at subgrade level, a proof-rolling program should be implemented, consisting of compacting the loose material with several passes of a vibratory drum roller under dry conditions and above freezing temperatures, and under the observation of Paterson. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill. Provisions should be made for a proof rolling program at the subject site.

Fill Placement

Fill placed for grading beneath the proposed building's footprint, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II.

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and compacted by the tracks of the spreading equipment to minimize voids.

The placement of subgrade material should be reviewed at the time of placement by Paterson personnel. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Terraxx.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II, or select subgrade material. This material should be tested and approved by Paterson 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 paved areas should be compacted to at least 100% of its SPMDD.

Protection of Subgrade and Bearing Surfaces

It is expected that site grading and preparation will consist of stripping of the soils containing significant amounts of organic materials and/or existing fill within the design underside of footing elevation. The contractor should take appropriate precautions to avoid disturbing the subgrade and bearing surfaces from construction and worker traffic. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional engineered fill.

5.3 Foundation Design

Bearing Resistance Value (Conventional Shallow Foundation)

Using Continuously applied loads, strip footings, footings placed over an undisturbed brown silty sand, clyey silt, glacial till, or an approved engineered fill placed on an undisturbed in-situ soil bearing surface can be designed using the bearing resistance values presented in Table 2.

Table 2 – Bearing Resistance Values

Bearing Surface	Bearing Resistance Values at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Engineered Fill over an Undisturbed In-Situ Soil Bearing Surface	150	225
Stiff Silty Clay to Clayey Silt	120	180
Compact Silty Sand	120	180
Very Dense to Dense Glacial Till	200	300

Note: A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance values at ULS.

Where the silty sand subgrade is observed to be in a loose state of compaction, proof-rolling under dry conditions and above freezing temperatures should be completed by an adequately sized roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by the geotechnical consultant. Soft or poor performing areas should be subexcavated and replaced with an approved engineered fill such as OPSS Granular A or Granular B Type II compacted to a minimum of 98% of the material's SPMDD.

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

Settlement

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

Existing Fill Below Footings

Where existing fill is encountered directly below the underside of footing (USF), the footings may be required to be lowered to an undisturbed, native bearing surface. Alternatively, a zero-entry, vertical trench can be excavated below the USF down to a native material and in-filled with engineered fill, compacted to a minimum 98% of the material's SPMDD or lean concrete mix (Minimum 15 MPa, 28 day strength). The in-filled trenches should be extended a minimum 150 mm beyond the footing face on all directions.

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 the in-situ bearing medium soils 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.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for foundations constructed at the subject site, according to the Ontario Building Code (OBC 2024). A higher seismic site class, such as Class A or B, may be applicable for the proposed buildings if a site-specific seismic shear wave velocity test is completed at the subject site.

The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2024 Ontario Building Code 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 materials within the footprint of the proposed building, the native soil surface, free of organics, or approved engineered fill surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Where existing fill or native soil containing significant organic content, is encountered below the floor slab, provisions should be made to remove the material from within the building footprint and replace the fill with OPSS Granular A or Granular B Type II compacted to a minimum 98% of the material's SPMDD.

It is recommended that the upper 200 mm for basement slabs and 300 mm for slab-on grade of sub-floor fill consists of 19 mm clear crushed stone.

It is also acceptable to use workable, site excavated soil material, free of deleterious materials and organics, below the floor slab and outside the lateral support zone of the proposed footings provided the material is placed under dry conditions and above freezing temperatures, reviewed and approved by Paterson prior to placement.

The backfill should be compacted using a vibratory roller making several passes under the full supervision of Paterson field personnel. A minimum 300 mm thick cap layer of OPSS Granular A or Granular B Type II should be placed over the backfill material and compacted to a minimum 98% of the material's SPMDD.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Pavement Design

For design purposes, the following pavement structures, presented below, are recommended for the design of car only parking areas, heavy truck parking areas, loading zones, and access at the subject site.

Table 2 - Recommended Pavement Structure - Car-Only Parking Areas

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

Table 3 - Recommended Pavement Structure - Access Lanes, Loading Zones and Heavy Truck Parking 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
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill, 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 II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, 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 compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It should be noted that the following recommendations for foundation drainage and backfill are considered preliminary. Paterson should provide comprehensive, building-specific foundation drainage and backfill design once detailed architectural, grading, mechanical, and structural plans are available, to supplement the following recommendations.

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for each of the proposed buildings. The system should consist of a 150 mm diameter perforated and corrugated plastic pipe, wrapped in a geosock, 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 clear crushed stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pump pit.

Foundation Backfill

Foundation Walls

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

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free draining, non-frost susceptible material. This material should be placed in a maximum of 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions..

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 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 isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure, and require additional protection, such as soil cover of 2.1 m, or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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 subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods.

The shoring requirements designed by Paterson, or a structural engineer specializing in those works and approved by Paterson, 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 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 designer prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, neighboring buildings, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced.

The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

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

Table 4 – 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 should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent material specifications and standard detail drawings from the department of public works and services, infrastructure services branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 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 layer should be increased to a minimum thickness of 300 mm where the subgrade consists of grey silty clay. The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

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. All cobbles larger than 200 mm in their 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 low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

6.6 Winter Construction

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

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

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

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

6.7 Corrosion Potential and Sulphate

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

6.8 Slope Stability Assessment

Slope Conditions Field Review

The existing slope conditions along the east side of the subject site were reviewed by Paterson field personnel as part of the geotechnical investigation on January 28, 2025. Four (4) slope cross-sections were studied as the worst-case scenarios. The cross-section locations are presented on Drawing PG7423-1 – Test Hole Location Plan in Appendix 2.

Paterson completed a topographic survey of the subject slope to better assess the existing conditions using a high-precision handheld GPS and referenced to a geodetic datum. Reference should be made to Drawing PG7423-1 – Test Hole Location Plan for the topographic points of surveys and Photographs captured during the above-noted site visit, attached to the current report.

The existing slope conditions along the eastern boundary of the site are detailed below. The ground surface across the existing slope was found to vary in elevation and existing conditions. Reference may also be given to photographs taken as part of our site review in Appendix 2.

In addition, during our site visit on January 28, 2024, a total of three (3) hand auger test holes were advanced to a maximum depth of 0.9 m below the existing ground surface using a stainless-steel hand auger. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

Slope Face Observations

A walk along the top of the slope and a general assessment of the slope conditions were completed during the site visit. Mature and small trees, as well as some fallen trees, were observed along the slope face. Bedrock outcrops were noted at multiple locations at the bottom of slope, particularly toward the north directions at the location of Sections A to C. It was also observed that the edge of the watercourse is approximately at the bottom of the slope. It is also observed that an existing asphalt-covered pathway runs along the top of the slope.

The slope was noted to be in relatively stable condition with a vegetative cover. However, minor to moderate erosions were observed at the edge of the watercourse, particularly at the location of cross-section A during the site visit.

Subsurface Conditions at Slope Location

Based on hand auger holes conducted along the slope by Paterson on January 28, 2025, the subsurface conditions at the slope location generally consist of a thin topsoil layer underlain by a thin layer of stiff to very stiff, brown to grey silty clay deposit to a depth between 0.2 to 0.4 m. In addition, a grey silty sand layer was observed at the location of HA 1-25, where no silty clay deposit was observed. Practical refusal to hand augering was encountered at HA 2-25 at a depth of 0.3 m below the existing ground surface.

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

Stable Slope Allowance

The analysis of the stability of the slope was carried out using SLIDE2, a computer program which permits a two-dimensional slope stability analysis using several limit equilibrium methods that are widely used and accepted analysis methods. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

Subsoil conditions at the cross-sections were inferred based on the completed boreholes at the subject site, completed hand auger holes on the slope face, nearby historical test holes, and our knowledge of the area. For a conservative review of the groundwater conditions, the subsoil was noted to be fully saturated for our analysis along the existing slope.

Static Loading Analysis

The results for the existing static conditions are shown in Figures 2A to 5A in Appendix 2. The results indicate that the factor of safety is 1.03, 1.24, 1.95, and 1.96 for Sections A, B, C, and D, respectively, under static conditions along the slopes.

Therefore, due to the factor of safety for sections A and B being less than 1.5, a stable slope setback from the top of the slope is required to achieve a factor of safety of 1.5 for the limit of the hazard lands in the area of Sections A, and B, under static loading.

Seismic Loading Analysis

The results for the existing seismic conditions are shown in Figures 2B to 5B in Appendix 2. The results indicate that the factor of safety is 0.82, 0.94, 1.08, and 1.34 for Sections A, B, C, and D, respectively, under seismic conditions along the slopes.

Therefore, due to the factor of safety for sections A, B, and C being less than 1.1, a stable slope setback from the top of the slope is required to achieve a factor of safety of 1.1 for the limit of the hazard lands in the area of Sections A, C, and B, under seismic loading.

Stable Slope Allowance

Given that the factor of safety for the existing slope under the existing conditions at Sections A, B, and C is below 1.5 for static loading and/or 1.1 for seismic loading, a stable slope setback ranging from 2.4 to 8.7 m from the top of the slope is required. This setback ensures a factor of safety of at least 1.5 for static loading and 1.1 for seismic loading, establishing the limit of the hazard lands in the area of Sections A, B, and C.

Erosion and Access Allowances

Based on the soil profiles encountered at the test hole locations and our knowledge of the area, and the presence of bedrock outcrops and glacial till consisting of cobbles and boulders at the bottom of the slope at various locations, a toe erosion allowance of 2 m and an access allowance of 6 m are required from the top of the slope or geotechnical setback (where applicable). It should be noted that the brown to grey silty clay encountered at the bottom of the slope at the location of HA 2-25 and HA 3-25 is present as a thin layer; therefore, it does not affect the toe erosion allowance setback from a geotechnical perspective.

Limit of Hazard Lands Setbacks

Typically, the limit of hazard lands setback is comprised of a stable slope allowance, a toe erosion allowance, and a 6 m erosion access allowance. Based on our analysis and the above-noted results, the limit of hazard lands designation line for the subject site is indicated on Drawing PG7423-1 – Test Hole Location Plan attached to the end of this report.

7.0 Recommendations

It is also recommended that the following be carried out by Paterson once preliminary and/or detailed designs of the proposed development have been prepared:

- ☐ Review detailed grading, servicing, landscaping, and structural plan(s) from a geotechnical perspective.
- ☐ Review of detailed design drawings, once available, to determine if supplemental foundation drainage and/or waterproofing recommendations are required.

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

- ☐ Review and inspection of the installation of the foundation waterproofing and drainage systems (if applicable).
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

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

8.0 Statement of Limitations

The recommendations provided in this report 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 J.S.S Riverdale Holdings Inc., or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
March 11, 2025
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Yashar Ziaeimehr, M.Sc., EIT

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

☐ J.S.S Riverdale Holdings Inc. (Email Copy)
☐ Paterson Group (1 Copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

551 Riverdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369145.90 **NORTHING:** 5028252.20 **ELEVATION:** 59.24

PROJECT: Proposed Residential Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 10, 2025

FILE NO. : PG7423

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION		
GROUND SURFACE		
ASPHALT	0.10m [59.14m]	
FILL: Crushed stone, some sand, trace gravel	0.23m [59.01m]	
FILL: Brown silty sand, some gravel, trace crushed stone, brick and asphalt	2.13m [57.11m]	
GLACIAL TILL: Dense, brown silty sand, with gravel, cobbles and boulders	2.97m [56.27m]	
GLACIAL TILL: Compact, brown silty clay, with gravel, sand, cobbles and boulders	5.26m [53.98m]	
GLACIAL TILL: Very dense, grey silty sand, with gravel, cobbles and boulders	5.79m [53.45m]	

End of Borehole

Practical refusal to augering at 5.79 m depth

(GWL at 3.56 m depth - February 19, 2025)

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			
SS 2	50	12-16-16-16 32	
SS 3	31	15-32-50-/ 82/0.18	
SS 4	33	12-25-24-31 49	
SS 5	67	1-8-13-14 21	
SS 6	54	14-33-32-18 65	
SS 7	54	26-27-50-/ 77/0.18	

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

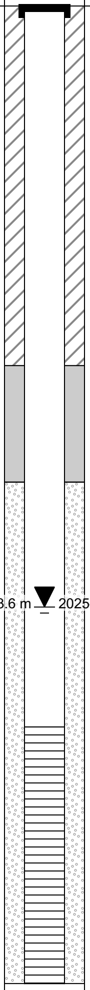
△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

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

STRATA PLOT

DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7, 8

MONITORING WELL CONSTRUCTION



ELEVATION (m): 59, 58, 57, 56, 55, 54, 53, 52

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

551 Riverdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369146.12 **NORTHING:** 5028249.84 **ELEVATION:** 59.23

PROJECT: Proposed Residential Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 10, 2025

FILE NO. : PG7423

HOLE NO. : BH 1A-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	■ 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	ELEVATION (m)
GROUND SURFACE									
ASPHALT 0.10m [59.13m]		0							59
FILL: Crushed stone, some sand, trace gravel 0.23m [59.00m]									
FILL: Brown silty sand, some gravel, trace crushed stone, brick and asphalt 2.13m [57.10m]		1 2							58 57
GLACIAL TILL: Dense, brown silty sand, with gravel, cobbles and boulders 2.62m [56.61m]									
End of Borehole Practical refusal to augering at 2.62 m depth		3 4 5 6 7 8							56 55 54 53 52

DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

551 Riverdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369125.02 **NORTHING:** 5028233.99 **ELEVATION:** 58.89

PROJECT: Proposed Residential Development

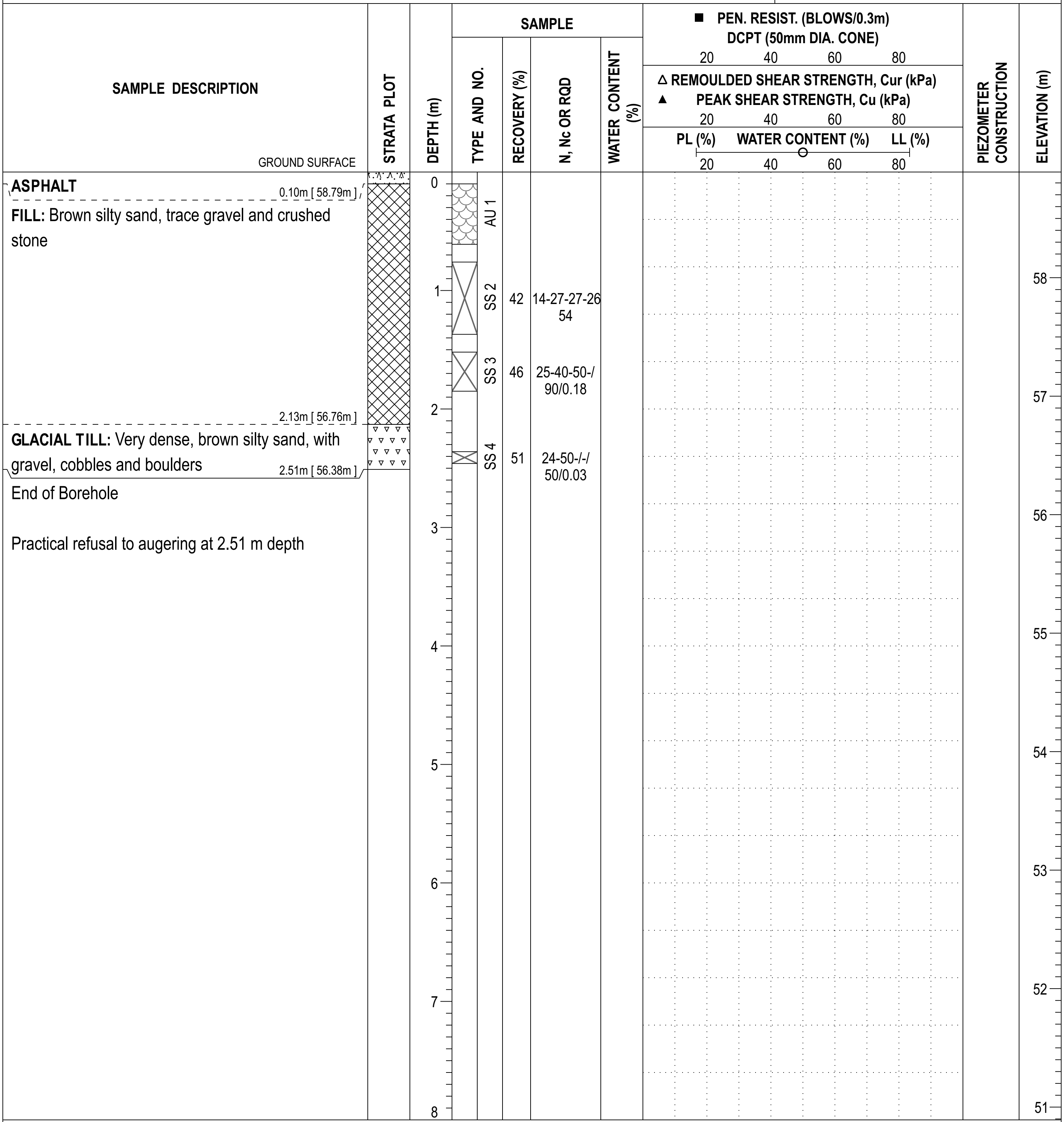
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 10, 2025

FILE NO. : PG7423

HOLE NO. : BH 2-25



SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE				
ASPHALT 0.10m [58.79m]				
FILL: Brown silty sand, trace gravel and crushed stone	AU 1			
	SS 2	42	14-27-27-26 54	
	SS 3	46	25-40-50-/ 90/0.18	
2.13m [56.76m]				
GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders	SS 4	51	24-50-/-/ 50/0.03	
2.51m [56.38m]				

End of Borehole

Practical refusal to augering at 2.51 m depth

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

551 Riverdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369125.02 **NORTHING:** 5028233.99 **ELEVATION:** 58.89

PROJECT: Proposed Residential Development

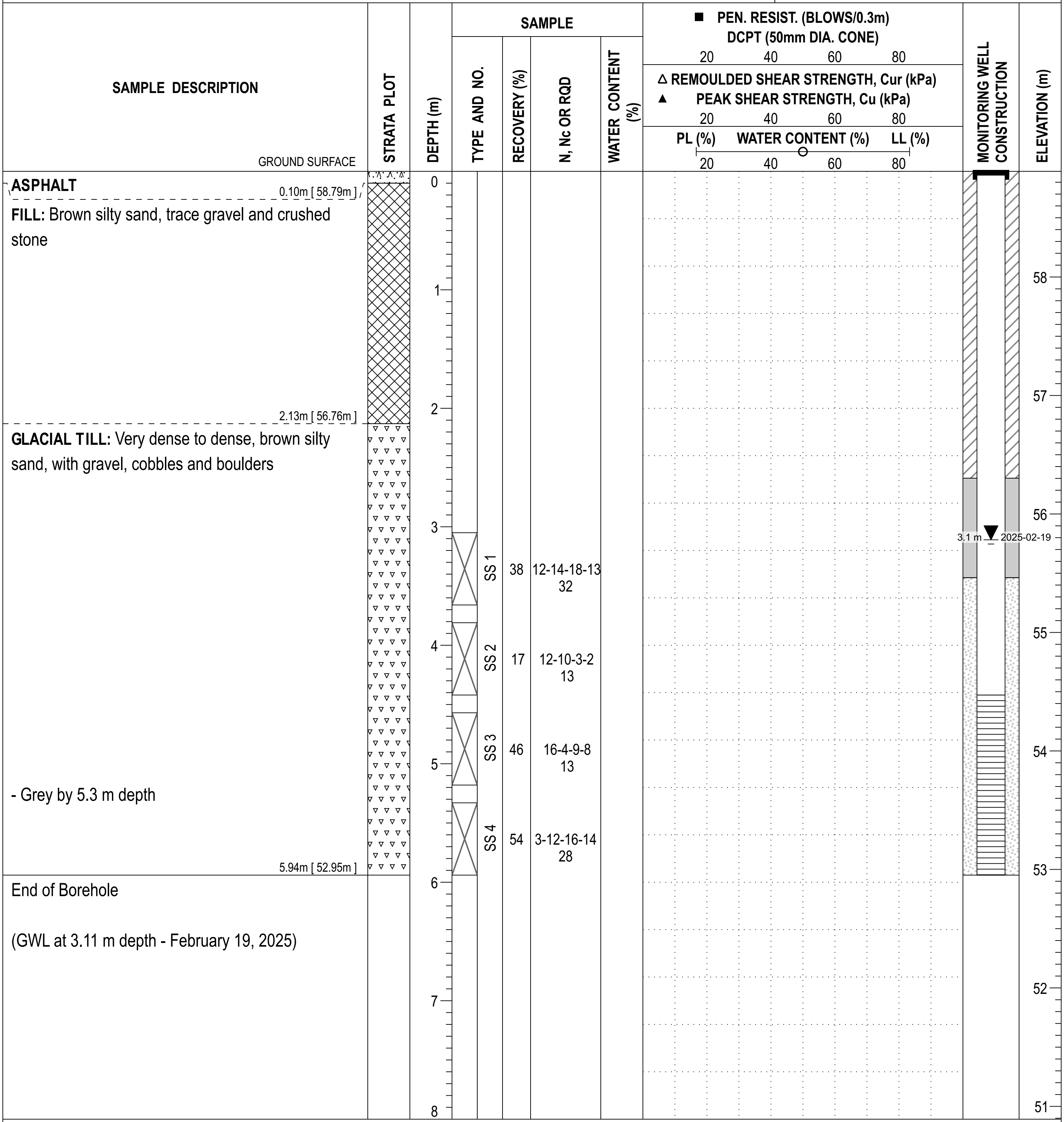
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 11, 2025

FILE NO. : PG7423

HOLE NO. : BH 2A-25



SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE				
ASPHALT 0.10m [58.79m]				
FILL: Brown silty sand, trace gravel and crushed stone 2.13m [56.76m]				
GLACIAL TILL: Very dense to dense, brown silty sand, with gravel, cobbles and boulders	SS 1	38	12-14-18-13 32	
	SS 2	17	12-10-3-2 13	
	SS 3	46	16-4-9-8 13	
- Grey by 5.3 m depth 5.94m [52.95m]	SS 4	54	3-12-16-14 28	

End of Borehole

(GWL at 3.11 m depth - February 19, 2025)

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

551 Riverdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369133.80 **NORTHING:** 5028170.36 **ELEVATION:** 58.45

PROJECT: Proposed Residential Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 10, 2025

FILE NO. : PG7423

HOLE NO. : BH 3-25

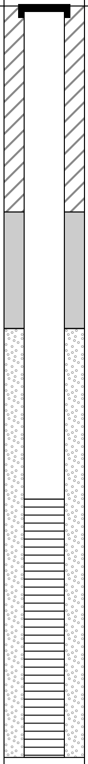
■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

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

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

SAMPLE DESCRIPTION	Depth to base	Elevation of base
GROUND SURFACE	0	58.45
ASPHALT	0.10m	[58.35m]
FILL: Brown silty sand, with gravel	0.15m	[58.30m]
FILL: Brown silty sand, some gravel, trace crushed stone and wood	1.83m	[56.62m]
Compact, brown **CLAYEY SILT**	2.21m	[56.24m]
Comapct, brown **SILTY SAND**	2.97m	[55.48m]
GLACIAL TILL: Very dense to dense, brown silty sand, with gravel, cobbles and boulders	4.45m	[54.00m]

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			
SS 2	17	11-5-7-8 12	
SS 3	33	7-7-7-6 14	
SS 4	42	10-10-8-12 18	
SS 5	25	15-26-26-14 52	
SS 6	38	6-17-23-22 40	



DEPTH (m): 0 1 2 3 4 5 6 7 8

ELEVATION (m): 58 57 56 55 54 53 52 51

End of Borehole

Practical refusal to augering at 4.45 m depth

The Groundwater level reading could not be taken on February 19, 2025, due to a snow pile obstructing the borehole location.

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

551 Riverdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369171.12 **NORTHING:** 5028178.62 **ELEVATION:** 55.40

PROJECT: Proposed Residential Development

BORINGS BY: Hand Auger

REMARKS:

DATE: January 28, 2025

FILE NO. : PG7423

HOLE NO. : HA 1-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	■ 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	ELEVATION (m)
GROUND SURFACE		0							
Grey **SILTY SAND**									55
0.84m [54.56m]									
GLACIAL TILL: Grey silty sand, with gravel									
0.89m [54.51m]									
End of Hand Auger Hole		1							
									54
		2							
									53
		3							
									52
		4							

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GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

551 Riverdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369197.43 **NORTHING:** 5028247.73 **ELEVATION:** 55.18

PROJECT: Proposed Residential Development

BORINGS BY: Hand Auger

REMARKS:

DATE: January 28, 2025

FILE NO. : **PG7423**

HOLE NO. : **HA 2-25**

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N, Nc OR RQD	SAMPLE WATER CONTENT (%)	■ 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	ELEVATION (m)
GROUND SURFACE									
Grey **SILTY CLAY** 0.33m [54.85m]	▨	0							55
End of Hand Auger Hole Practical refusal to augering at 0.33 m depth		1							54
		2							53
		3							52
		4							

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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

551 Riverdale Avenue, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369190.43 **NORTHING:** 5028248.56 **ELEVATION:** 57.73

PROJECT: Proposed Residential Development

BORINGS BY: Hand Auger

REMARKS:

DATE: January 28, 2025

FILE NO. : **PG7423**

HOLE NO. : **HA 3-25**

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	■ 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	ELEVATION (m)
GROUND SURFACE		0							
TOPSOIL and organics 0.05m [57.68m]	■								
Brown to grey **SILTY CLAY** 0.23m [57.50m]	▨								
End of Hand Auger Hole									
									57
		1							
									56
		2							
									55
		3							
									54
		4							

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

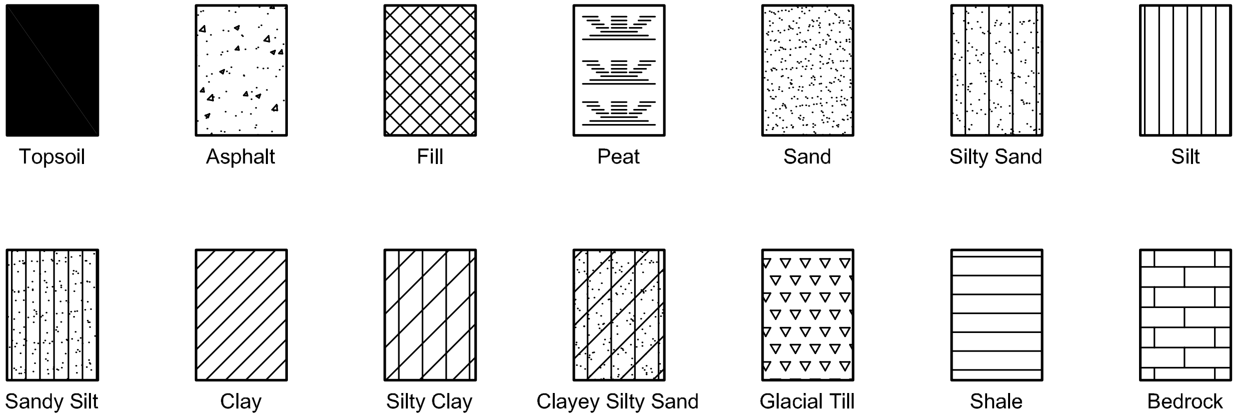
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

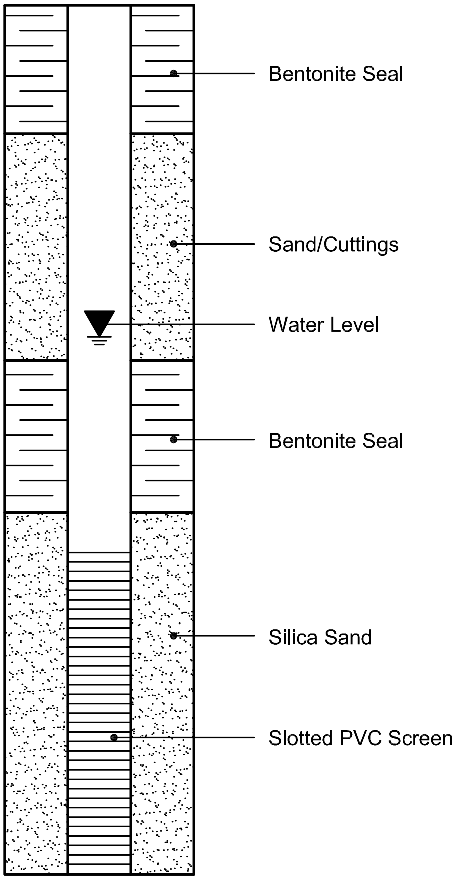
SYMBOLS AND TERMS (continued)

STRATA PLOT

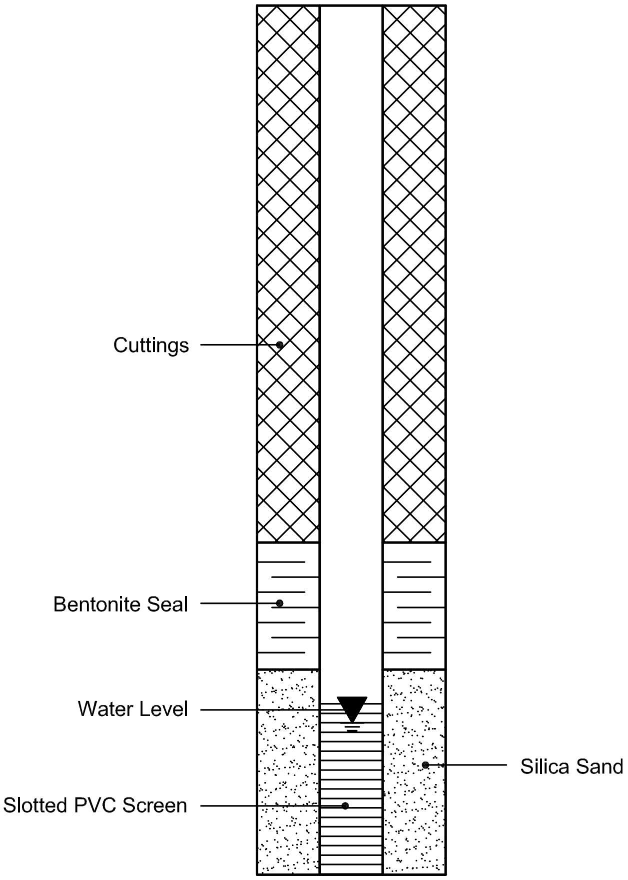


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 19-Feb-2025

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 12-Feb-2025

Client PO: 61913

Project Description: PG7423

Client ID:		BH1-25-SS4	-	-	-		
Sample Date:		10-Feb-25 09:00	-	-	-	**-**	**-**
Sample ID:		2507315-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	95.7	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.64	-	-	-	-	-
Resistivity	0.1 Ohm.m	39.7	-	-	-	-	-
Anions							
Chloride	10 ug/g	33	-	-	-	-	-
Sulphate	10 ug/g	83	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2A TO 5B – SECTIONS FOR SLOPE STABILITY ANALYSIS

PHOTOGRAPHS CAPTURED DURING SITE VISIT ON JANUARY 28, 2025

DRAWING PG7423-1 – TEST HOLE LOCATION PLAN

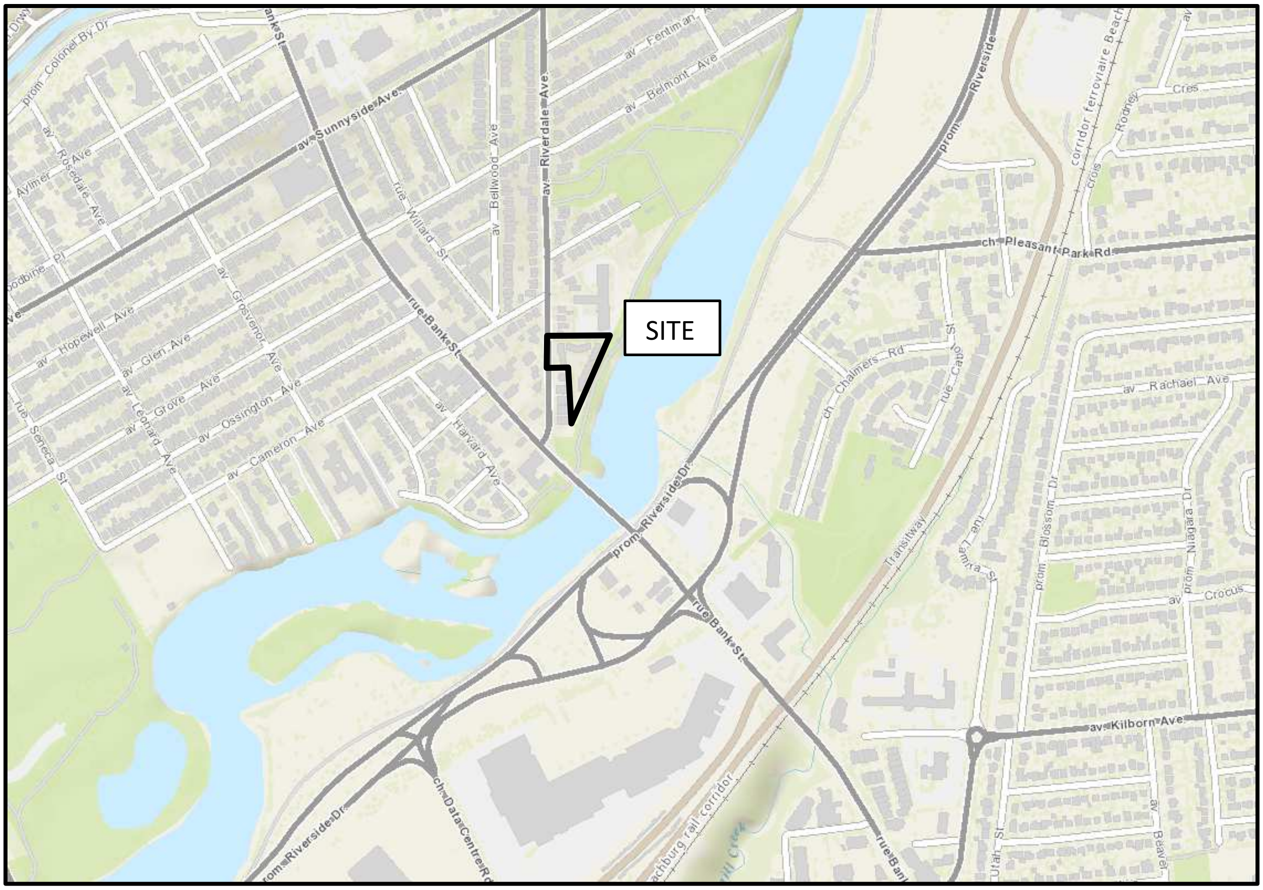
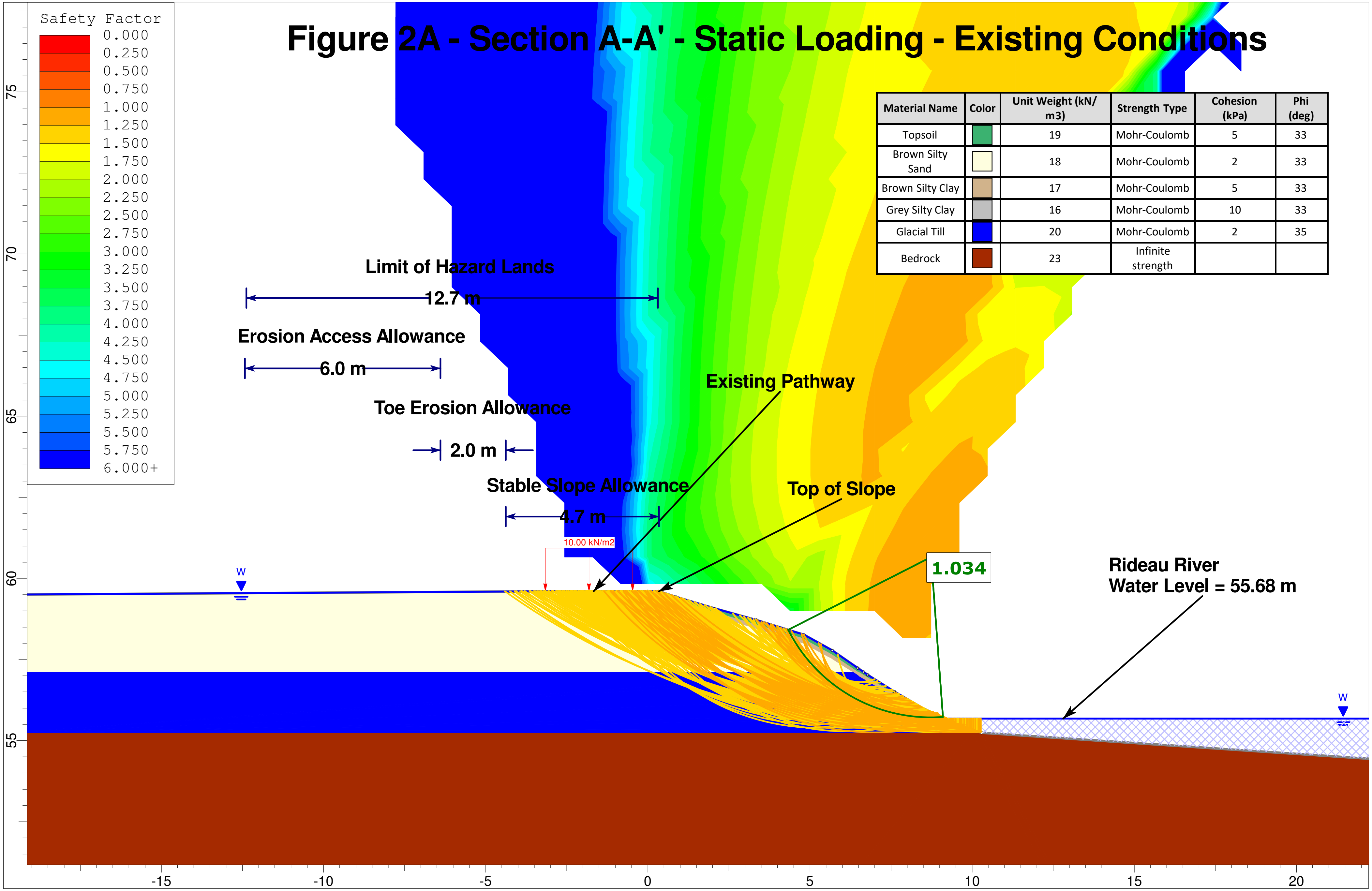


FIGURE 1

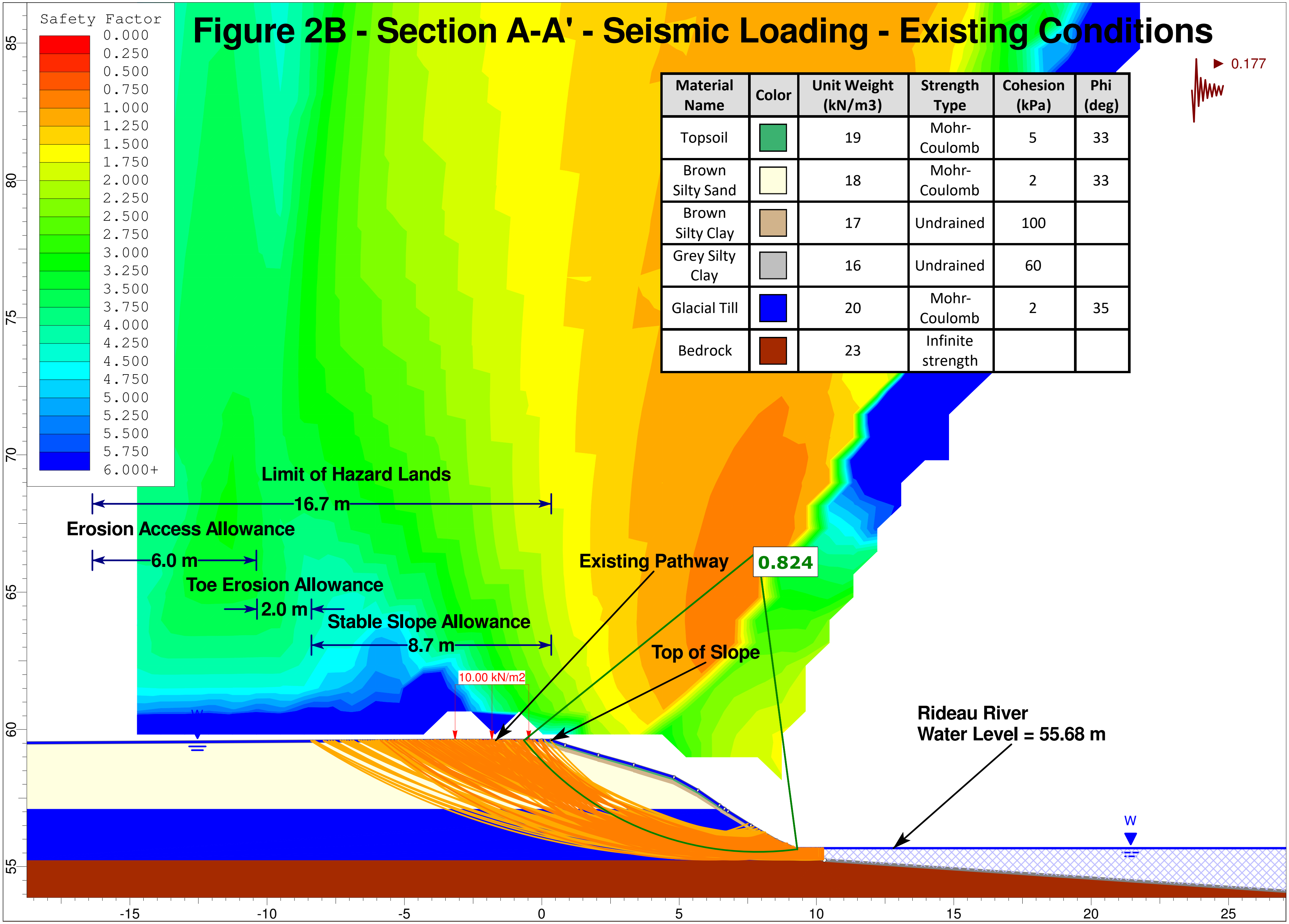
KEY PLAN

Figure 2A - Section A-A' - Static Loading - Existing Conditions



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	19	Mohr-Coulomb	5	33
Brown Silty Sand	🟨	18	Mohr-Coulomb	2	33
Brown Silty Clay	🟫	17	Mohr-Coulomb	5	33
Grey Silty Clay	⬜	16	Mohr-Coulomb	10	33
Glacial Till	🟦	20	Mohr-Coulomb	2	35
Bedrock	🟥	23	Infinite strength		

Figure 2B - Section A-A' - Seismic Loading - Existing Conditions



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	19	Mohr-Coulomb	5	33
Brown Silty Sand	🟨	18	Mohr-Coulomb	2	33
Brown Silty Clay	🟫	17	Undrained	100	
Grey Silty Clay	⬜	16	Undrained	60	
Glacial Till	🟦	20	Mohr-Coulomb	2	35
Bedrock	🟥	23	Infinite strength		

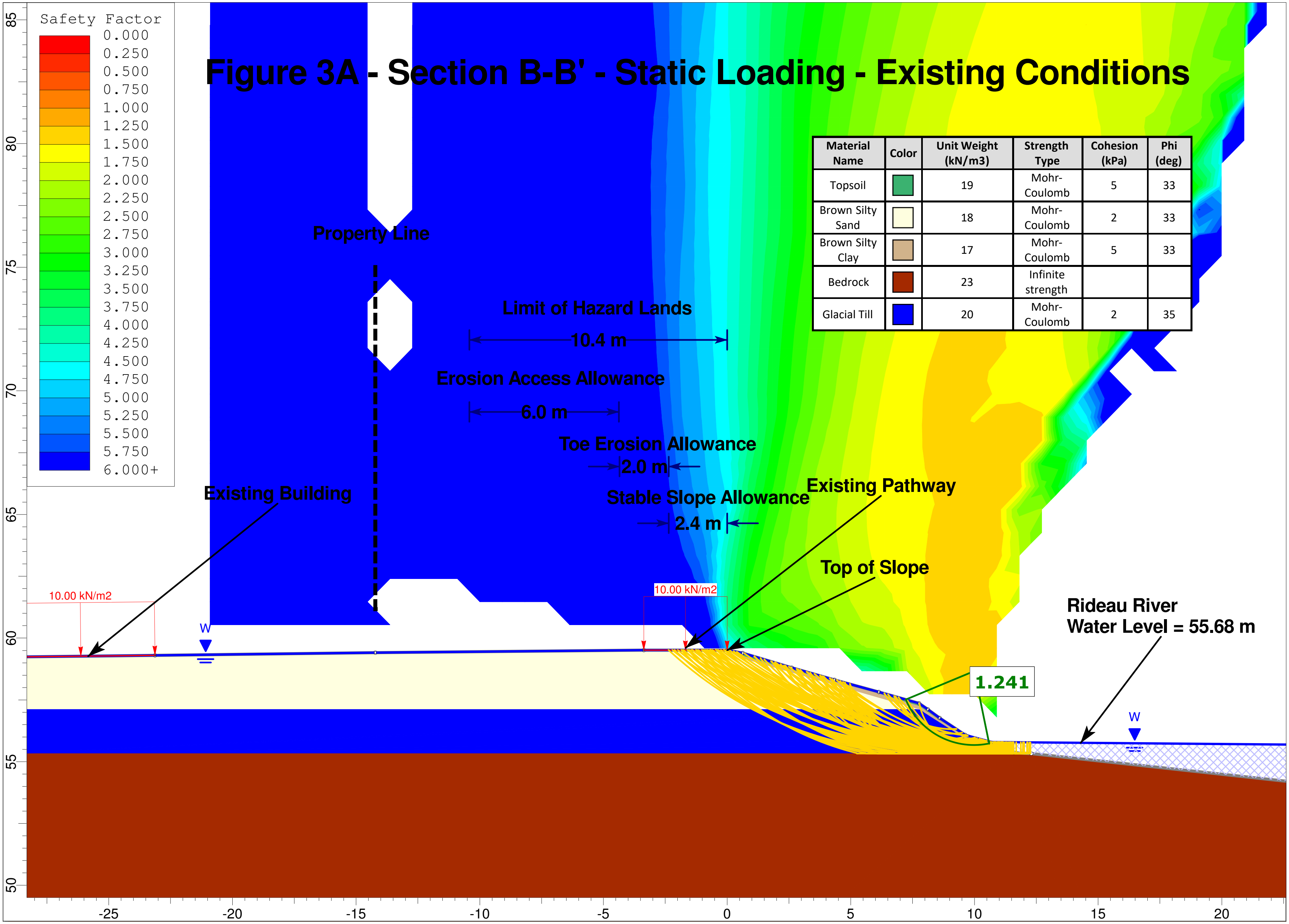


Figure 3A - Section B-B' - Static Loading - Existing Conditions

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	19	Mohr-Coulomb	5	33
Brown Silty Sand	🟨	18	Mohr-Coulomb	2	33
Brown Silty Clay	🟫	17	Mohr-Coulomb	5	33
Bedrock	🟥	23	Infinite strength		
Glacial Till	🟦	20	Mohr-Coulomb	2	35

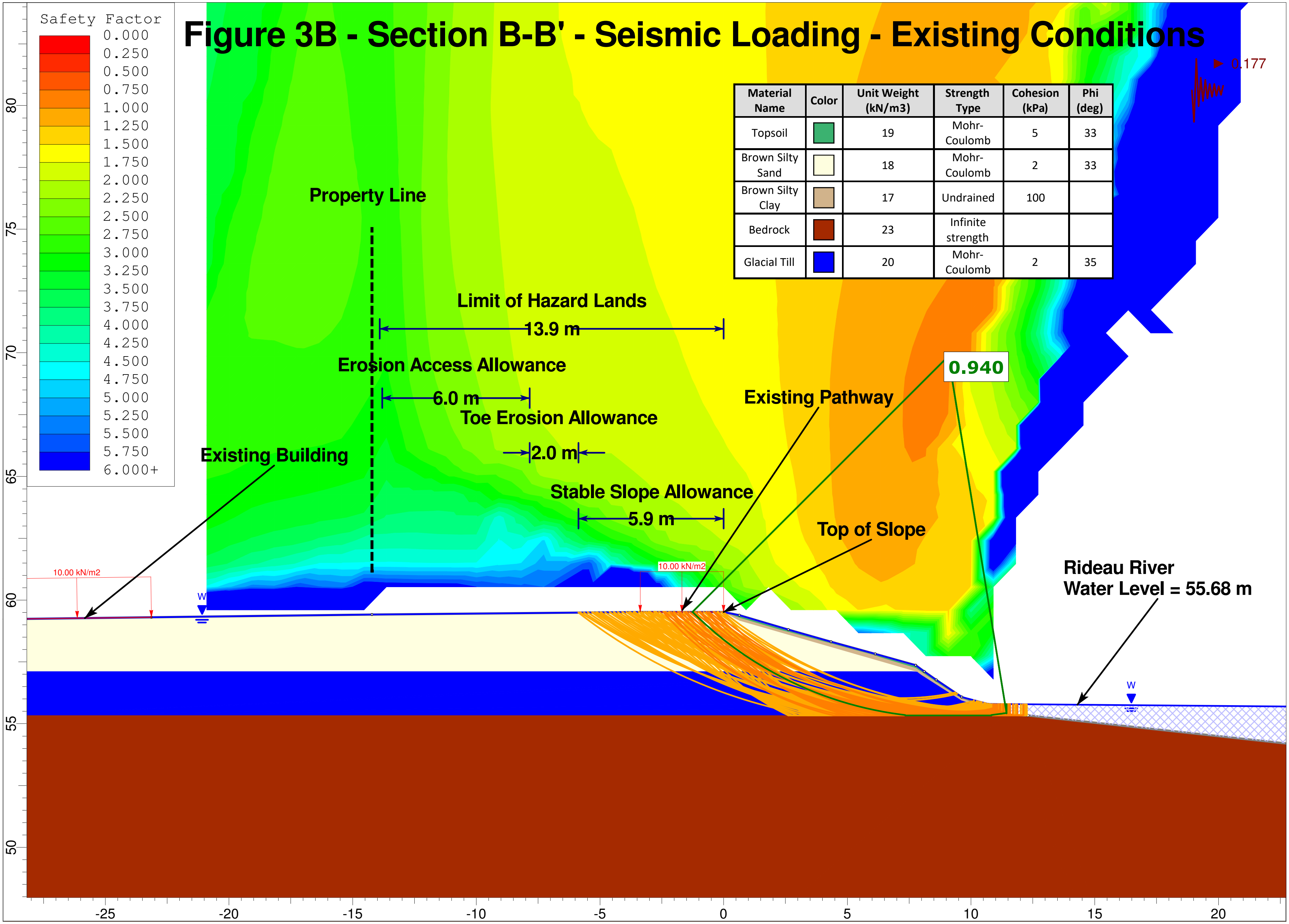


Figure 3B - Section B-B' - Seismic Loading - Existing Conditions

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	19	Mohr-Coulomb	5	33
Brown Silty Sand	⬜	18	Mohr-Coulomb	2	33
Brown Silty Clay	🟫	17	Undrained	100	
Bedrock	🟫	23	Infinite strength		
Glacial Till	🟦	20	Mohr-Coulomb	2	35

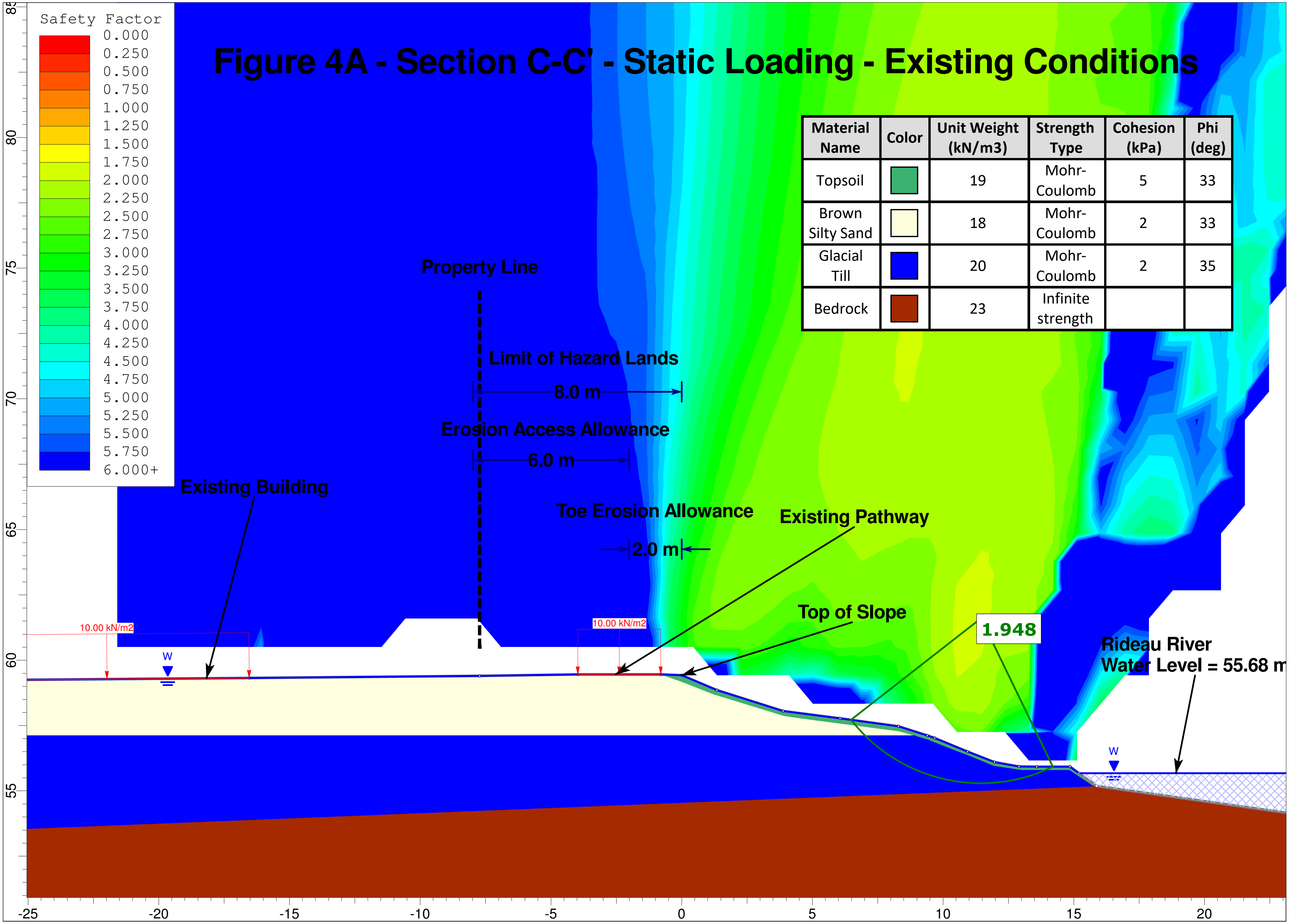
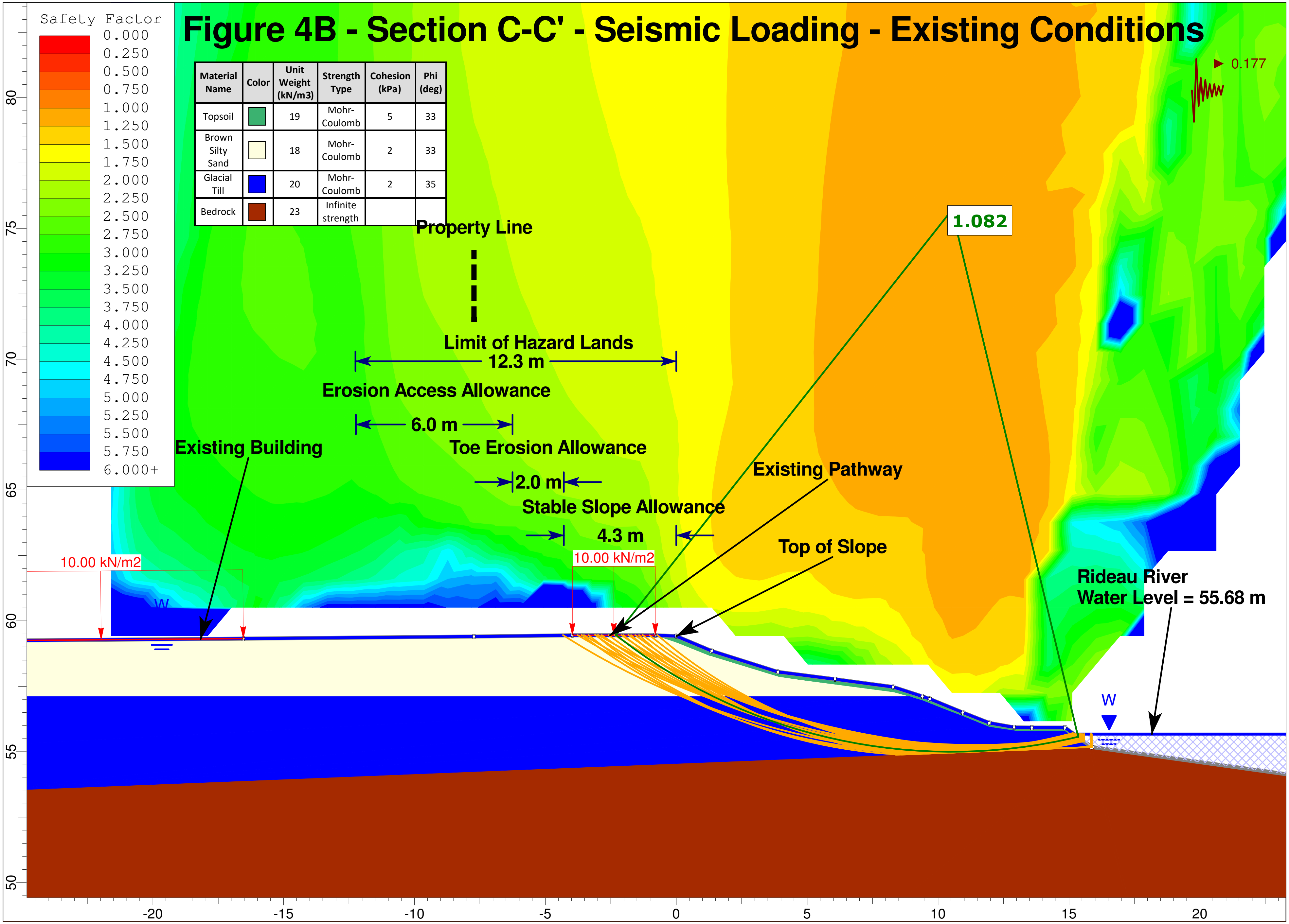


Figure 4A - Section C-C' - Static Loading - Existing Conditions

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	19	Mohr-Coulomb	5	33
Brown Silty Sand	⬜	18	Mohr-Coulomb	2	33
Glacial Till	🟦	20	Mohr-Coulomb	2	35
Bedrock	🟫	23	Infinite strength		

Figure 4B - Section C-C' - Seismic Loading - Existing Conditions



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	19	Mohr-Coulomb	5	33
Brown Silty Sand	🟨	18	Mohr-Coulomb	2	33
Glacial Till	🟦	20	Mohr-Coulomb	2	35
Bedrock	🟫	23	Infinite strength		

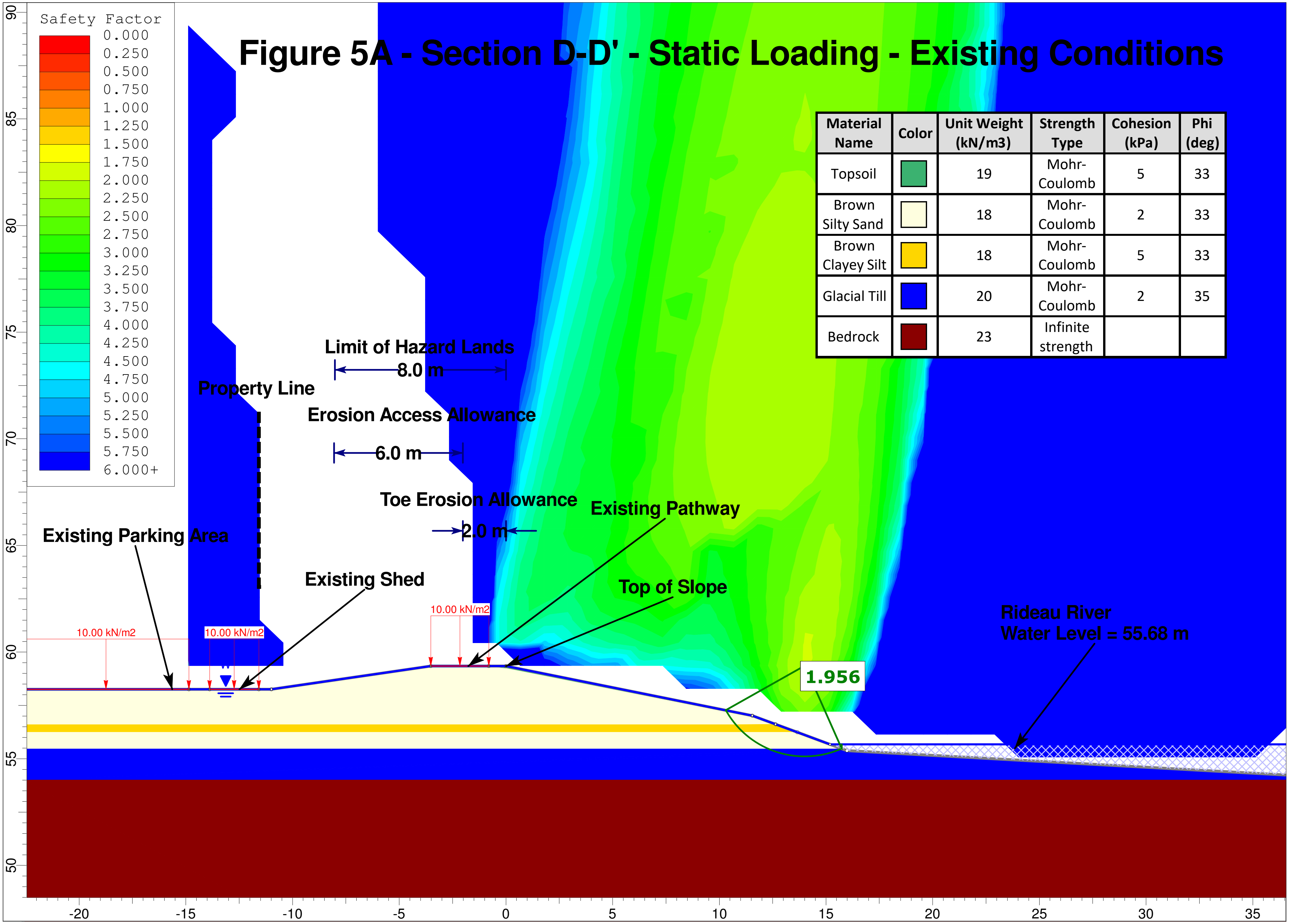
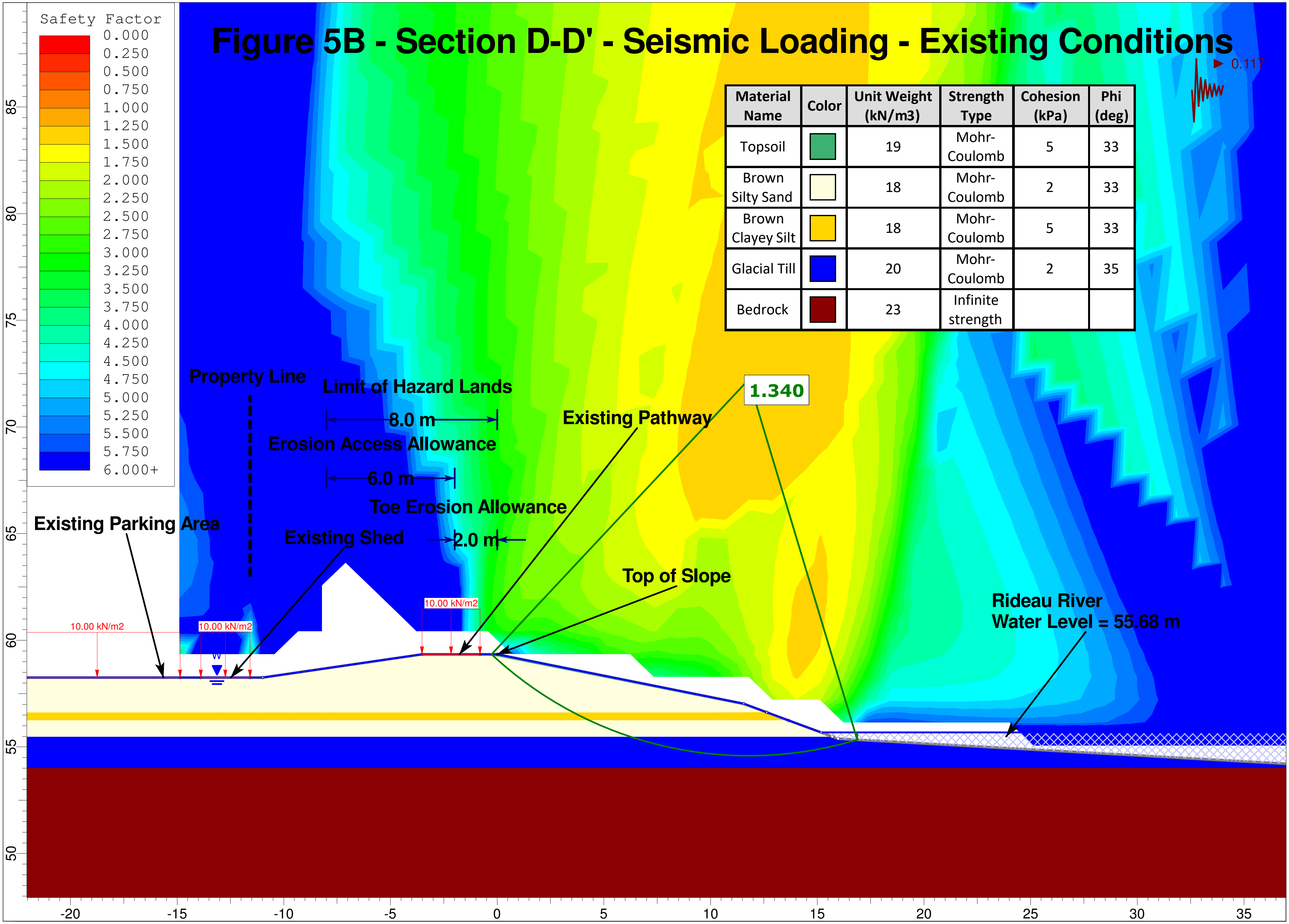


Figure 5A - Section D-D' - Static Loading - Existing Conditions

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	19	Mohr-Coulomb	5	33
Brown Silty Sand	⬜	18	Mohr-Coulomb	2	33
Brown Clayey Silt	🟨	18	Mohr-Coulomb	5	33
Glacial Till	🟦	20	Mohr-Coulomb	2	35
Bedrock	🟥	23	Infinite strength		

Figure 5B - Section D-D' - Seismic Loading - Existing Conditions



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	19	Mohr-Coulomb	5	33
Brown Silty Sand	⬜	18	Mohr-Coulomb	2	33
Brown Clayey Silt	🟨	18	Mohr-Coulomb	5	33
Glacial Till	🟦	20	Mohr-Coulomb	2	35
Bedrock	🟥	23	Infinite strength		

Photo 1: Existing conditions of the slope face – Facing to the north.

Photo 2: Existing conditions of the slope face – Facing to the south.

Photo 3: Existing conditions of the slope face – Facing to the west.

Photo 4: Minor to moderate erosion at the bottom of the slope was observed – Facing to the southwest.

Photo 5: Presence of bedrock outcrops at various locations at the bottom of the slope. Minor to moderate erosion at the bottom of the slope was observed – Facing to the west.

Photo 6: Presence of bedrock outcrops at various locations at the bottom of the slope – Facing to the west.

Photo 7: Existing conditions of the slope face – Facing to the north.

Photo 8: Existing conditions of the slope face – Facing to the west.

Photo 9: Presence of bedrock outcrops at various locations at the bottom of the slope – Facing to the west.

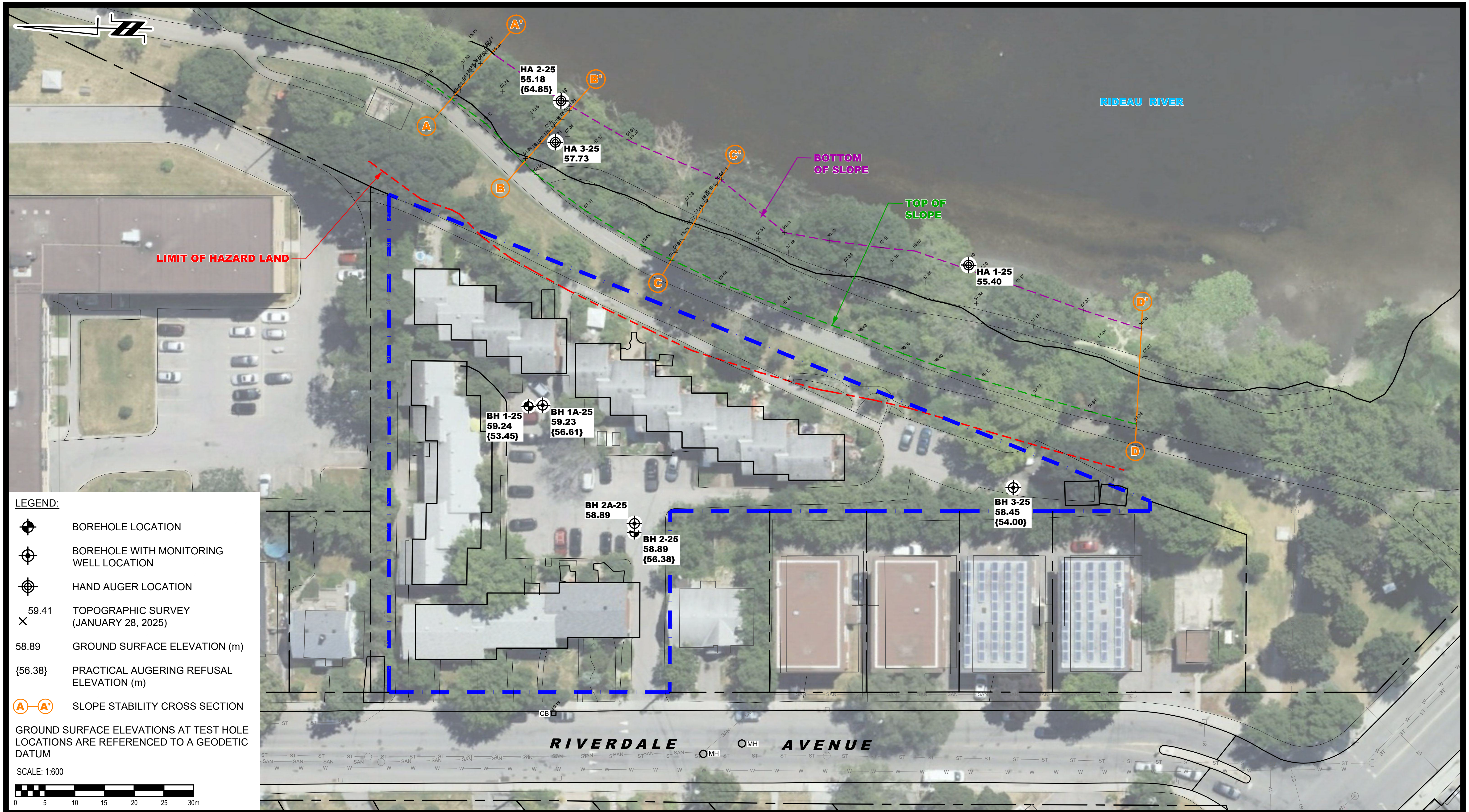
Photo 10: Presence of bedrock outcrops at various locations at the bottom of the slope – Facing to the north.

Photo 11: Presence of bedrock outcrops at various locations at the bottom of the slope – Facing to the west.

Photo 12: Presence of bedrock outcrops at various locations at the bottom of the slope – Facing to the west.

Photo 13: Existing conditions of the slope face – Facing to the south.

Photo 14: An existing asphalt-covered pathway passing along the top of the slope – Facing to the south.



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⌖	HAND AUGER LOCATION
59.41 ×	TOPOGRAPHIC SURVEY (JANUARY 28, 2025)
58.89	GROUND SURFACE ELEVATION (m)
{56.38}	PRACTICAL AUGERING REFUSAL ELEVATION (m)
Ⓐ—Ⓐ'	SLOPE STABILITY CROSS SECTION

GROUND SURFACE ELEVATIONS AT TEST HOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:600

0 5 10 15 20 25 30m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

J.S.S. RIVERDALE HOLDINGS INC.
GEOTECHNICAL INVESTIGATION & SLOPE STABILITY ASSESSMENT
PROPOSED RESIDENTIAL DEVELOPMENT
551 RIVERDALE AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Drawn by: YA
Checked by: YZ
Approved by: FAS
Date: 02/2025
Report No.: PG7423-1
Dwg. No.: PG7423-1
Revision No.:

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11x17