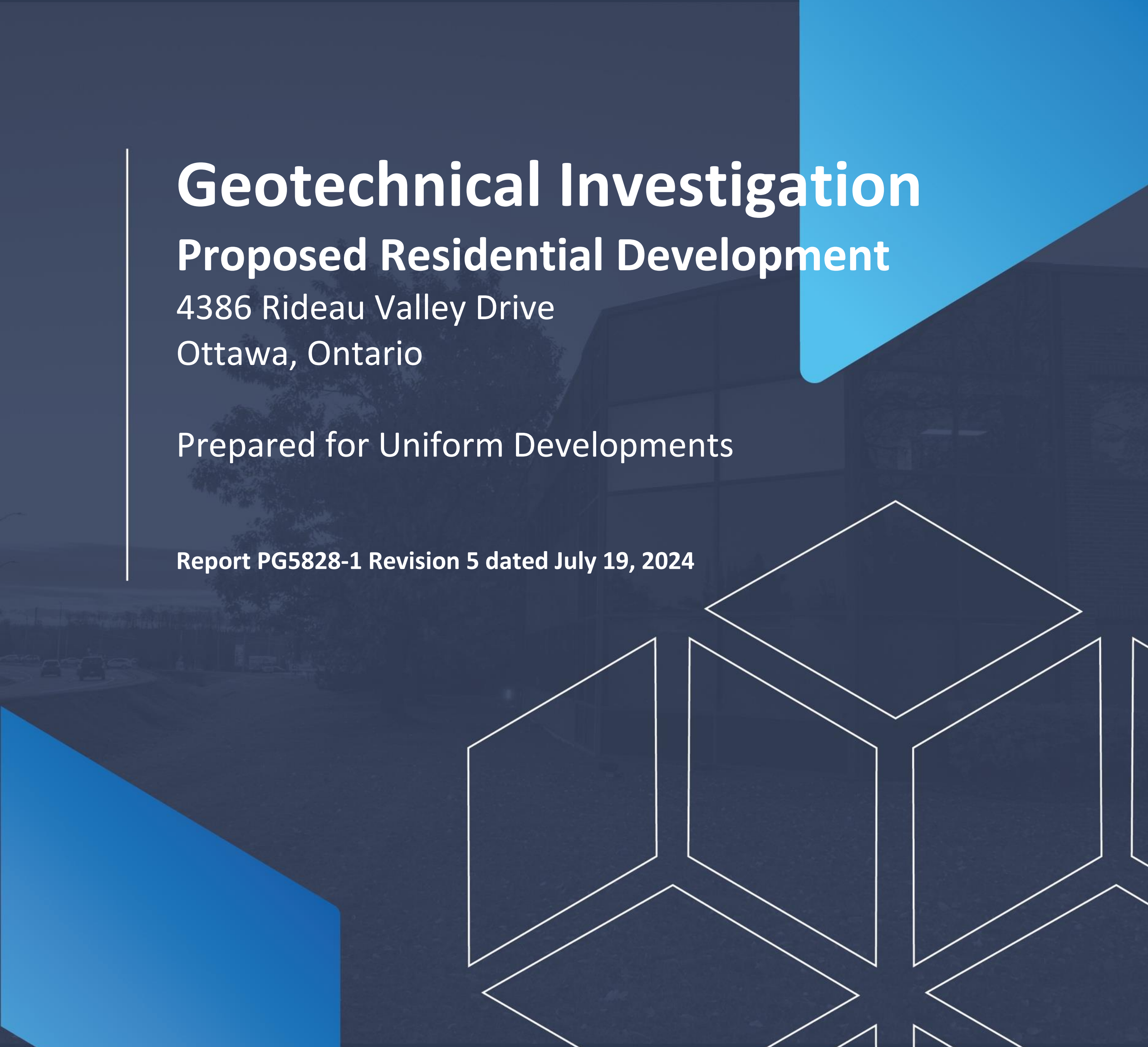

Geotechnical Investigation

Proposed Residential Development

4386 Rideau Valley Drive
Ottawa, Ontario

Prepared for Uniform Developments

Report PG5828-1 Revision 5 dated July 19, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Uniform Developments to conduct a geotechnical investigation for the proposed industrial building, located at 4386 Rideau Valley Drive, Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.
- Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

Based on the conceptual site plan, it is understood that the proposed development will consist of townhouses and single-family residential dwellings. Associated driveways, garages, roadways, and landscaping areas are also anticipated throughout the subject site. It is anticipated the proposed dwellings will be provided basement levels. Further, it is anticipated that the proposed development will be municipally serviced.

It is to be noted that as part of the proposed residential subdivision, it is anticipated that a river park will be constructed on 4386 Rideau Valley Drive.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on May 19 and 20, 2021 and consisted of advancing a total of 9 boreholes to a maximum depth of 6.7 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5828-1 - Test Hole Location Plan included in Appendix 2.

Also, a supplemental field investigation was completed for the proposed river park, which is to be located across 4386 Rideau Valley Drive on June 16, 2022, to assess the slope stability of the proposed park and to delineate the limit of hazard lands. At that time, a total of two boreholes were advanced down to a maximum depth of 5.9 m below existing ground surface. The results of this supplemental field investigation are presented in Appendix 3.

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

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. 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 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.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at boreholes BH 3-21 and BH 5-21. The DCPT 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

Boreholes BH 8-21 and BH 9-21 were fitted with 51 mm diameter PVC groundwater monitoring wells. The other boreholes were fitted with flexible piezometers to allow groundwater level monitoring. 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:

- ☐ 3.0 m of slotted 51 mm diameter PVC screen at 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 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 and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5828-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of 1 shrinkage test, 4 grain size distribution analyses and 8 Atterberg limit tests were completed on selected soil samples. The results of the testing are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing, and Atterberg Limits Results sheets presented in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was collected from BH 3-21 and submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of agricultural farmland and is currently occupied by a residential dwelling and associated structures at the southeast property boundary. The ground surface across the subject site slopes downward gradually from south to north and east to west.

The site is intersected by Mud Creek along its center and bordered to the west by Wilson Cowan Drain. The area along the creek is bordered by sloped terrain and valley corridors which were reviewed in the field at the time of completing the field investigation. The slope conditions were observed in the field to carry out a slope stability assessment and are discussed further in Subsection 6.8 of this report.

The site is bordered by a municipal maintenance property to the north, Rideau Valley Drive followed by Rideau River to the east, Bankfield Road to the south, and a residential subdivision to the west.

4.2 Subsurface Profile

Generally, the subsurface soil profile at the test hole locations consists of topsoil underlain by a deposit of silty clay. The topsoil was underlain by sand and further by silty clay at BH 5-21, BH 6-21 and BH 7-21 and by fill underlain by glacial till at BH 8-21.

The silty clay deposit generally consisted of a hard to very stiff brown weathered crust to depths ranging between 1.5 and 5.2 m below ground surface. The brown silty clay was observed to be underlain by a stiff grey silty clay at BH 1-21, BH 3-21, BH 4-21, BH 5-21, BH 6-21, and BH 1-22.

Glacial till was encountered below the clay deposit at BH 2-21, BH 9-21, BH 1-22, and BH 2-22. The glacial till deposit was generally observed to consist of compact to dense brown silty sand with gravel, cobbles and boulders.

Practical refusal to augering was encountered at an approximate depth of 4.4 m at borehole BH7-21. Practical refusal to DCPT was encountered at an approximate depth of 15 m, 8.8 m, and 4.24 at BH 3-21, BH 5-21, and BH 2-22, respectively.

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.

Field vane testing was completed within the silty clay deposits encountered in the test holes at the subject site. The shear strength values, as obtained from the field vane, were generally ranging between 50 to >200 kPa.

The remolded shear strength values as obtained from the field vane testing conducted in the test holes was observed to range between 20 to 80 KPa.

The sensitivity index of the encountered silty clay deposit was calculated based on the ratio between the undisturbed and remolded shear vane test measured in the field, for all the boreholes, and it was found to be generally below 4, indicating a normal sensitivity clay.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Dolomite of the Oxford formation, with an overburden drift thickness of 10 to 25 m depth.

Atterberg Limit and Shrinkage Tests

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay samples at selected locations throughout the subject site. Based on the results of the Atterberg limits, the encountered silty clay deposit is classified as clay with high plasticity according to the USCS. The results of the Atterberg limits tests are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH1-SS3	1.5-2.1	54	24	30	35.57	CH
BH2-SS2	0.7-1.3	39	17	22	29.01	CL
BH3-SS4	2.2-2.9	51	20	32	34.52	CH
BH4-SS3	1.5-2.1	49	23	26	36.13	CL
BH5-SS2	0.7-1.3	54	22	31	30.27	CH
BH6-SS3	1.5-2.1	62	27	34	43.76	CH
BH7-SS4	2.2-2.9	65	28	37	55.67	CH
BH9-SS2	0.7-1.3	34	17	17	22.41	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
CH: Inorganic Clay of High Plasticity CL: Inorganic Clay of Low Plasticity

The results of the shrinkage limit test indicate a shrinkage limit of 19.9% and a shrinkage ration of 2.05.

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on four (4) selected soil samples. The results of the grain size analysis are summarized in Table 2 and presented on the Grain-Size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH1-21	SS4	0.0	2.4	50.0	47.6
BH4-21	SS2	0.0	39.1	30.5	30.4
BH6-21	SS4	1.2	91.3	7.5	
BH9-21	SS3	21.5	52.6	25.9	

4.3 Groundwater

Groundwater levels were measured in the monitoring wells and piezometers installed at the borehole locations on May 26, 2021. The measured groundwater levels noted at that time are presented in Table 3.

Table 3 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH1-21	88.26	1.72	86.54	May 26, 2021
BH2-21	89.55	Dry	N/A	May 26, 2021
BH3-21	87.89	4.99	82.90	May 26, 2021
BH4-21	88.11	1.90	86.21	May 26, 2021
BH5-21	85.36	2.26	83.10	May 26, 2021
BH6-21	85.35	1.98	83.37	May 26, 2021
BH7-21	87.56	Dry	N/A	May 26, 2021
BH8-21	91.32	3.58	87.74	May 26, 2021
BH9-21	90.52	3.77	86.75	May 26, 2021

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS and are referenced to 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. 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 approximately 4 to 5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed residential development. It is anticipated that the proposed buildings will be supported by shallow foundations placed over very stiff brown silty clay, compact to dense glacial till or an approved engineered fill pad.

Permissible grade raise recommendations are discussed in Subsection 5.3. 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 under buildings. However, it should be noted that lightweight fill is not permitted under the ROWs.

Due to the presence of a low to medium sensitivity marine silty clay deposit across the site, the proposed development will be subjected to tree planting setback restrictions, as further detailed under Subsection 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 amounts of organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

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

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil (including the plastic sensitive silty clay deposit) could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as CCW MiraDRAIN 2000 or Delta-Teraxx

Proof Rolling

For the proposed driveways and roadways, proof rolling of the subgrade is required in areas where the existing fill, free of significant amounts of organics and deleterious materials, is encountered. It is recommended that the subgrade surface be proof rolled **under dry conditions and above freezing temperatures** 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 at the time of construction.

In-Fill Recommendations – Rear Yard of Lot 5 and Lot 6

It is understood that in-filling the face of the slope within the rear yards of Lot 5 and Lot 6 to match the surrounding slope and since the existing drainage swale feature will be in-filled by the proposed development. Based on this, it is recommended the following fill placement recommendations be followed for reinstating the slope throughout the swale footprint.

- All existing topsoil, organic soils and deleterious fill and materials should be stripped from the area that will be in-filled.

- It is recommended fill be placed upon benches excavated throughout the swale area to provide adequately wide surfaces for the placement and compaction of the fill material. The benches are recommended to be shaped to provide a 1.5H:1V profile extending upwards and away from the bottom of the swale and in a stepped fashion with maximum 500 mm high steps.

- It is recommended that the fill consist of a workable, site-generated brown silty clay fill placed in maximum 300 mm thick loose lifts **under dry conditions and in above freezing temperatures** to in-fill the slope. Every lift should be adequately compacted using a vibratory sheepsfoot roller and approved by Paterson personnel during placement.

- The grading along the slope should be provided to match the surrounding slope and to a maximum steepness of 3H:1V. In the even that adjacent grading is steeper than 3H:1V, it is recommended that the steepness of the in-fill be provided as 3H:1V.

- A minimum 300 mm thick layer of clayey topsoil mixed with hardy grass seed or hydroseed (weather permitting). All efforts should be taken to retain all vegetation surrounding the in-fill area throughout the in-fill effort.

- **Inspections During Construction:** Periodic inspections during the backfilling operation should be completed by Paterson personnel to confirm the above noted recommendations are undertaken as recommended at the time of construction.

  Reference should be made to Section 2A and 2B which consider the proposed grading in-fill as described herein.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Spread Foundations)

Based on the subsurface profile encountered, it is anticipated that the residential dwellings will be founded on shallow foundations placed on very stiff, brown silty clay, compact to dense glacial till or approved engineered fill. Using continuously applied loads, footings for the proposed development can be designed using the bearing resistance values presented in Table 4.

Table 4 - Bearing Resistance Values

Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Very Stiff Brown Silty Clay	150	225
Compact to Dense Glacial Till	150	225
Engineered Fill Pad	150	225

Note: Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, can be designed for silty clay bearing mediums using the above noted bearing resistance values.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen, or disturbed soil, whether in-situ or not, have been removed, prior to placement of concrete for footings. An engineered fill pad may be required where the existing fill is located at the proposed founding elevation for buildings located throughout southeastern portion of the subject site. It is recommended that the existing fill, where encountered at the design founding elevation, be sub-excavated to a suitable native, in-situ soil bearing medium.

The area may be raised to the proposed founding elevation using an imported engineered fill such as OPSS Granular B Type II placed in 300 mm thick loose lifts and compacted to 98% of the materials SPMDD. The placement of this engineered fill layer should be reviewed and approved at the time of construction by Paterson personnel.

The bearing resistance values will be reviewed against the grading plan and boreholes once available. Bearing resistance values for footing design should be confirmed on a per lot basis by the geotechnical consultant at the time of construction

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

Settlement

The total and differential settlement will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 to 20 mm, respectively.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, permissible grade raise restrictions are recommended for all structures placed on a silty clay bearing medium. The recommended grade raise restrictions are shown on Drawing PG5828-3 – Permissible Grade Raise Plan included in Appendix 2. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations.

If greater 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 of the soils surrounding the buildings. However, it should be noted that lightweight fill is not permitted under the ROWs.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered at this site. The soils encountered at the subject site consist of silty clays, which are cohesive in nature. These soils were evaluated for liquefaction susceptibility in accordance with the criteria prepared by Bray at al. 2004 which determines that all soils with a plasticity index exceeding 20% are not liquifiable (Figure 1). In general, the plasticity index results completed on samples taken from the silty clay layer were found to be above 20. Therefore, soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

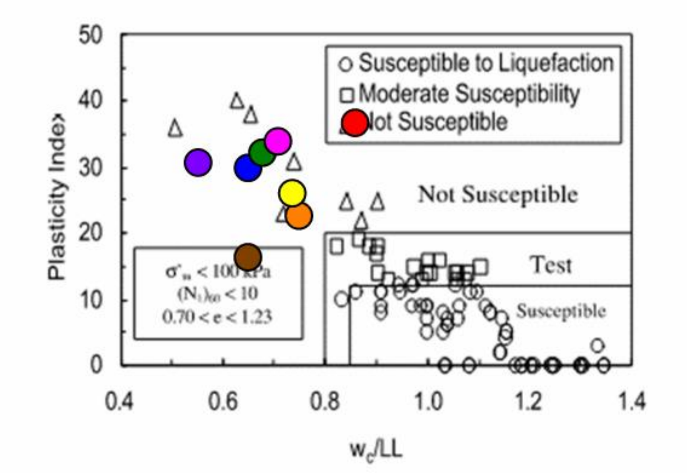

Figure 1. Criteria for evaluating liquefaction susceptibility of fine-grained soils (Bray et al. 2004).

Reference should be made to the Atterberg Limits Results sheet in Appendix 1 which provides the test results referenced in the above-noted chart.

5.5 Basement Slab Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native soils or approved engineered fill pad will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction. It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone crushed stone.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings). All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Pavement Design

For design purposes, the pavement structure presented in the following tables could be used for the design of driveways and local residential streets and roadways. The proposed pavement structures are presented in Tables 5 and 6 on the following page.

Table 5 – 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 6 – Recommended Pavement Structure – Local Residential Roadways

Thickness (mm)	Material Description
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear 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.	

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 geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity. Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to drainage lines.

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 the proposed residential development. The system should consist of a 150 mm diameter perforated corrugated plastic pipe wrapped in a geosock, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear 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 pit.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation Side Slopes

The excavations for the proposed development will be mostly through a hard to very stiff silty clay. Where excavations are above the groundwater level to a depth of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Flatter slopes could be required for deeper excavations or for excavations below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring systems should be used.

The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

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

Deep excavation is not anticipated for the proposed residential units. However, if deep services are anticipated at the subject site, then deep service trenches in excess of 3 m should be completed using a temporary shoring system, such as stacked trench boxes in conjunction with steel plates, designed by a structural engineer. The trench boxes should be installed to ensure that the excavation sidewalls are tight to the outside of the trench boxes and that the steel plates are extended below the base of the excavation to prevent basal heave, if required.

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material’s standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement.

The backfill material within the frost zone (about 1.5 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 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 should be carried in a manner to avoid the introduction of frozen materials, snow, or ice into the trenches.

6.7 Corrosion Potential and Sulphate

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

6.8 Slope Stability Assessment

The west and north boundaries of the site are adjacent to a valley of Wilson Cowan Drain to Mud Creek and the main channel of Mud Creek, respectively. The existing slope conditions were reviewed by Paterson field personnel as part of the geotechnical investigation on May 19, 2021. Four (4) slope cross-sections were studied as the worst-case scenarios. The cross sections were analyzed considering existing and post-development conditions, considering an average grade raise of approximately 2m. The cross-section locations are presented on Drawing PG5828-1 - Test Hole Location Plan in Appendix 2.

Field Observations

The existing slope conditions along the north and west boundaries of the site are detailed below. Reference may also be given to photographs taken as part of our site review in Appendix 2.

Slope Conditions Along the Western Boundary

The existing slope along the western portion of the subject site was generally observed to be covered with well rooted vegetation across its surface. The slope was observed to be approximately 4 m high and appeared to have a profile ranging between 2.5H:1V and 4H:1V. An approximately 4 to 15 m wide valley floor was observed across the creek length which appeared to decrease up to 2 m along some bends.

The width of the Wilson Cowan Drain was noted to be between 1.5 m and 2.0 m wide long its length and typically decreased to between 1.2 and 1.5 m at its bends. At the time of our visit, the water level appeared to be up to 1.0 m in depth in deeper areas and bends, and no more than 150 mm in depth in shallower areas.

The majority of the Wilson Cowan Drain bed appeared to be covered by an in-situ stiff grey silty clay. The bank channels were generally observed to be well vegetated such that bank material did not appear to be exposed directly to stream flow. Signs of erosion were documented by the project geo-fluvial consultant and should be referred to in the associated report

The creek was generally observed to consist of Wilson Cowan Drain to the Mud Creek channel and discharged into the main channel along the north-west portion of the subject site.

Slope Conditions Along the Northern Boundary

The existing slope bordering Mud Creek to the north of the subject site is generally heavily vegetated with brush and some trees. Mud Creek generally consists of an active watercourse which flows from west to east and discharges into the Rideau River located to the east of Rideau Valley Drive. The majority of the channel was observed to be fronted onto by a valley floor with the exception of the area of Cross Section C-C which was observed to be fronted onto by a slope at the creeks bend. The majority of the channel banks were observed to be affected by active erosion and were exposed directly to stream flow. Additional signs of erosion consisted of exposed tree roots, fallen trees, over-steepening and under-cutting of the bank at bends in the creek alignment.

The width of the creek was noted to be between 4.0 m and 6.0 m wide and decreased to widths of approximately 4.0 m at its bends. At the time of our visit, the water level appeared to be approximately 600 mm in depth across the majority of the channel’s footprint.

The slopes’ gradient was observed to slope downward towards Mud Creek gradually at an approximately 2H:1V to 15H:1V grade.

Slope Conditions Along the North-East Boundary

The existing slope bordering the area along the north-east of the subject site is generally heavily vegetated with brush and trees. The area appeared to consist of a tributary between the Mud Creek and the Rideau River. An approximately 50 m wide valley floor was observed across separating the main channel and the tributary. The slope fronting onto the channel or the valley floor was observed to be approximately 2.5 to 4 m high and appeared to have a profile ranging between 2.5H:1V and 4H:1V.

The width of watercourse was noted to be between 5 m and 20 m wide along its length and typically decreased to approximately 10 m at its bends. At the time of our visit, the water level appeared to be up to 300 mm in depth in deeper areas and bends, and no more than 150 mm in depth in shallower areas. The majority of the watercourse’s bed appeared to be covered by an in-situ stiff grey silty clay.

The bank channels were generally observed to be well vegetated with well-rooted vegetation and mature trees. However, some erosion consisting of exposed banks had been noted along the toe of the slope throughout bend areas.

Slope Stability Analysis

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

Subsoil conditions at the cross-section locations were determined based on test holes coverage conducted within the subject site. The subsurface profile across the proposed subdivision was observed to be generally consistent. Therefore, the soil profile used in the slope stability analysis for all cross sections was based on boreholes BH 1-21, BH 4-21, BH 5-21, and BH 6-21, which were in proximity to the watercourse and drain area. The soil profile considered in the slope stability analysis consists of 3m of very stiff brown silty clay crust underlain by firm grey silty clay. For a conservative review of the groundwater conditions, the silty clay deposit was noted to be fully saturated for our analysis and exiting at the toe of the slope and across the creek section. For a conservative review of the groundwater conditions, the silty clay deposit was noted to be fully saturated for our analysis and exiting at the toe of the slope and across the creek section.

Table 7 – Effective Stress Soil Parameters (Static – Drained Analysis)

Soil Layer	Depth (m)	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay/Site Excavated Silty Clay	-	17	33	5
Grey Silty Clay	4-5	16	33	10
Glacial Till	11	20	33	0

Table 8– Total Stress Soil Parameters (Seismic - Undrained Analysis)

Soil Layer	Elevation of Top of Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay/ Site Excavated Silty Clay	-	17	-	150
Grey Silty Clay	4-5	16	-	65
Glacial Till	11	20	33	0

Static Loading Analysis

The results are shown in Figures 2, 4, 6, 8, 10, 12, 14, & 16 in Appendix 2. The results indicate a slope with a factor of safety of 2.1 and 2.4 at Section A and Section B, respectively. The results also indicate slopes with factors of safety less than 1.5 beyond the top of slope at Section C and D. Based on these results, a stable slope setback varying between 1.3 and 5.3 m from the top of the slope are required to achieve a factor of safety of 1.5 for the limit of the hazard lands in the area of Sections C and D.

Seismic Loading Analysis

An analysis considering seismic loading and the groundwater at ground surface was also completed. A horizontal acceleration of 0.16g was considered for all slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the analyses including seismic loading are shown in Figures 3, 5, 7, 9, 11, 13, 15, and 17 in Appendix 2. The results indicate a slope with a factor of safety greater than 1.1 at all sections. However, it should be noted that the stable slope setback associated with our static loading analysis governs the required stable slope setback required for static conditions.

Toe Erosion and Access Allowances

Based on the soil profiles encountered at the borehole locations and the soil encountered throughout the watercourse, a stiff grey silty clay is anticipated to be subject to erosion activity by the watercourse within the main valley corridor.

Based on the anticipated soils, and the nature of the existing watercourse and drain, a toe erosion allowance of 5 m, and as advised in geo-fluvial study, may be applied from the watercourse edge for Mud Creek Watercourse and Wilson Cowan Drain.

Further, an access allowance of 6 m is required from the top of slope or geotechnical setback (where applicable). In areas where the watercourse edge has meandered to within 5 m of the toe of the existing slope, the toe erosion and access allowances should be applied in addition to geotechnical setback limit from the top of slope.

Limit of Hazard Lands

Based on the above, a setback taken from the top of the current slope has been provided as based on the above-noted observations and analysis. Reference should be made to Drawing PG5828-1 – Test Hole Location Plan for the proposed Limit of Hazard Lands setback for development considerations at the subject site. The existing vegetation on the slope faces should not be removed as it contributes to the stability of the slope and reduces erosion.

6.9 Landscaping Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed review of the soils in the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. The results of our Atterberg limit and sieve testing are presented in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture levels and consistency, the silty clay across the subject site is considered low to medium sensitivity clay.

The following tree planting setbacks are recommended for low to medium sensitivity silty clay deposits throughout the subject site.

Large trees (mature tree height over 14 m) can be planted at the subject site provided a tree to foundation setback equal to 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 height up to 7.5 m) and medium size trees (mature height 7.5 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 as indicated procedural changes below.

- ❒ A small tree must be provided with a minimum 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).

Swimming Pools

The in-situ soils are considered to be acceptable for swimming pools. Above ground swimming pools must be placed at least 4 m away from the residence foundation and neighboring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Aboveground Hot Tubs

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer’s specifications.

Installation of Decks or Additions

Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

In addition to the above recommendations, it should be noted that the following is should be considered for the proposed development:

☐ It is important to avoid directing uncontrolled water towards the slope (drainage, gutter, septic field, pool & hot tub drainage, etc.)

☐ It is important to avoid overloading the top of the slope (backfill, fill, miscellaneous waste, grass cuttings, branches, leaves, snow, etc.)

☐ It is important to avoid excavating at the base of the slope.

☐ It is important to maintain a healthy native vegetation cover.

☐ Any future additions, such as aboveground swimming pools or accessory buildings, should entail reassessment of slope stability unless this has been pre-confirmed via supplementary slope stability analyses during the design stage.

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined.

- Review detailed grading and site servicing plan(s) from a geotechnical perspective.
- Review detailed landscaping plan (s) from a geotechnical perspective.
- Observation of all bearing surfaces prior to the placement of concrete.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to placing backfilling material.
- Observation of clay seal placement at specified locations.
- Field density tests to determine the level of compaction has been achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Uniform Development 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
July 19, 2024
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Mrunmayi Anvekar, M.Eng.

Drew Petahtegoose, P.Eng.

Report Distribution:

- ☐ Uniform Developments (email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN-SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TESTING RESULTS

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9 Auriga Drive, Ottawa, Ontario, K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

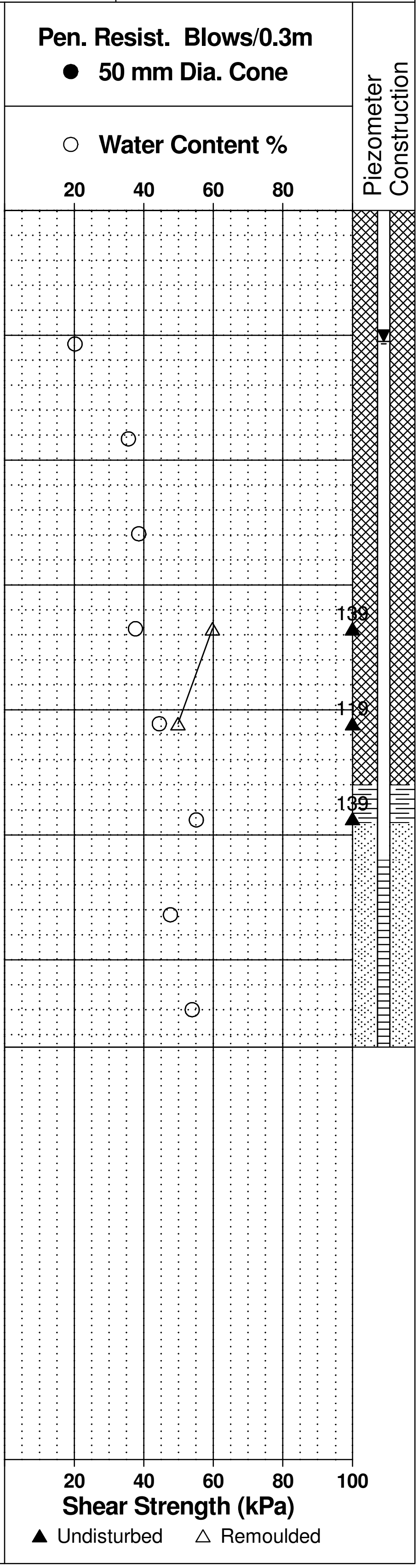
DATE May 19, 2021

FILE NO. PG5828

HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.30		AU	1		
Hard to very stiff, brown **SILTY CLAY,** trace sand		SS	2	100	13
		SS	3	100	6
		SS	4	100	8
		SS	5	100	7
		SS	6	100	5
		SS	7	83	P
- grey by 5.2m depth		SS	8	83	P
6.70		SS	9	83	P
End of Borehole (GWL @ 1.05m - May 26, 2021)					

DEPTH (m)	ELEV. (m)
0	88.26
1	87.26
2	86.26
3	85.26
4	84.26
5	83.26
6	82.26



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Proposed Residential Development
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FILE NO. PG5828

REMARKS

HOLE NO. BH 2-21

BORINGS BY Track-Mount Power Auger

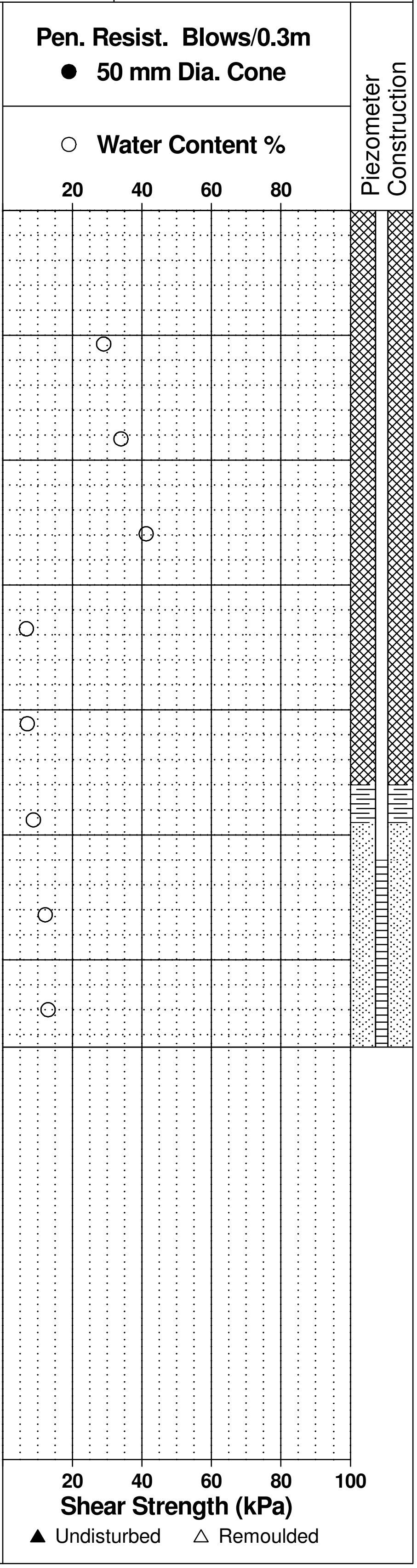
DATE May 19, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.30		AU	1		
Hard to very stiff, brown **SILTY CLAY,** some to trace sand		SS	2	100	11
		SS	3	58	8
2.80		SS	4	60	8
GLACIAL TILL: Dense to compact, brown silty sand with gravel, cobbles and boulders, trace clay		SS	5	67	46
		SS	6	62	27
		SS	7	60	21
		SS	8	58	15
6.70		SS	9	50	10

End of Borehole

(BH dry - May 26, 2021)

DEPTH (m)	ELEV. (m)
0	89.55
1	88.55
2	87.55
3	86.55
4	85.55
5	84.55
6	83.55



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Proposed Residential Development
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DATUM Geodetic

REMARKS

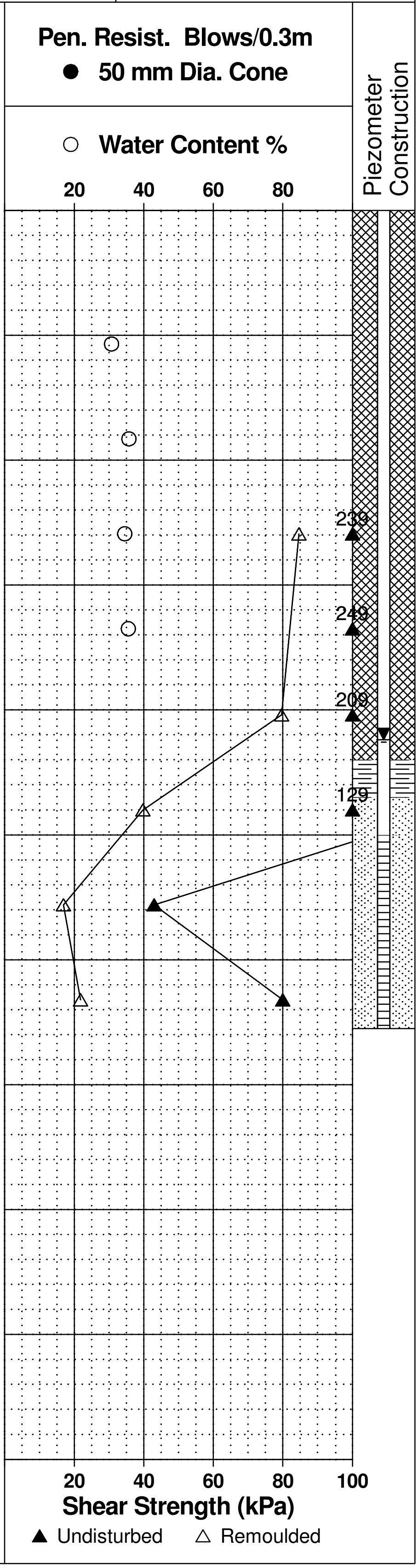
BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. PG5828

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.89
TOPSOIL 0.25							
		AU	1				
		SS	2	83	9	1	86.89
		SS	3	83	5	2	85.89
Hard to very stiff, brown **SILTY CLAY,** trace sand - sand content decreasing with depth		SS	4	83	P		
		SS	5	83	P	3	84.89
						4	83.89
- stiff and grey by 5.2m depth						5	82.89
						6	81.89
6.55							
Dynamic Cone Penetration Test commenced at 6.55m depth. Cone pushed to 11.0m depth						7	80.89
						8	79.89
						9	78.89
						10	77.89



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
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DATUM Geodetic

REMARKS

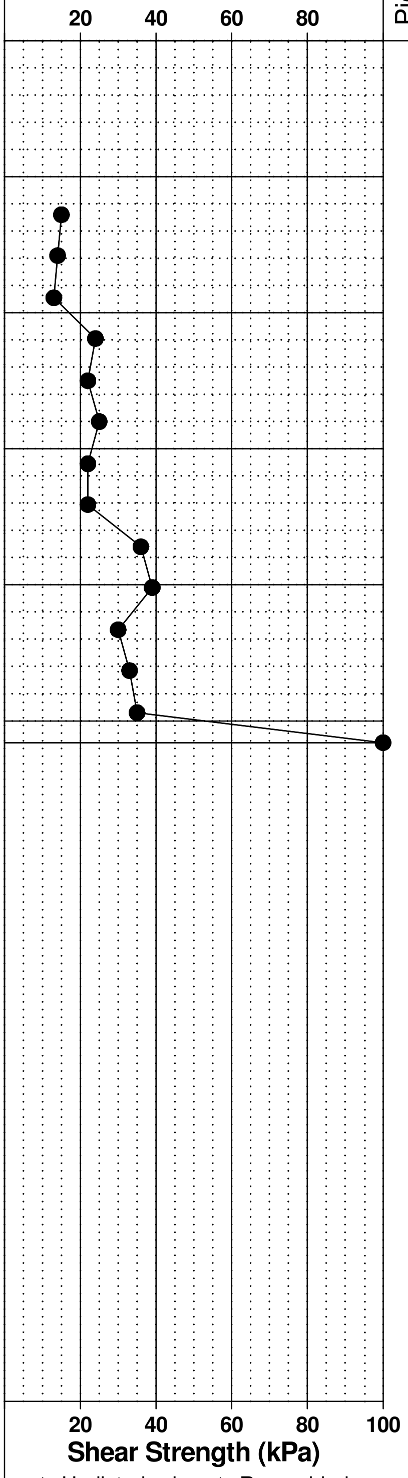
BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. PG5828

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						10	77.89		
						11	76.89		
						12	75.89		
						13	74.89		
						14	73.89		
						15	72.89		
15.16 End of Borehole Practical refusal to DCPT at 15.16m depth (GWL @ 4.24m - May 26, 2021)									



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5828**

REMARKS

HOLE NO. **BH 4-21**

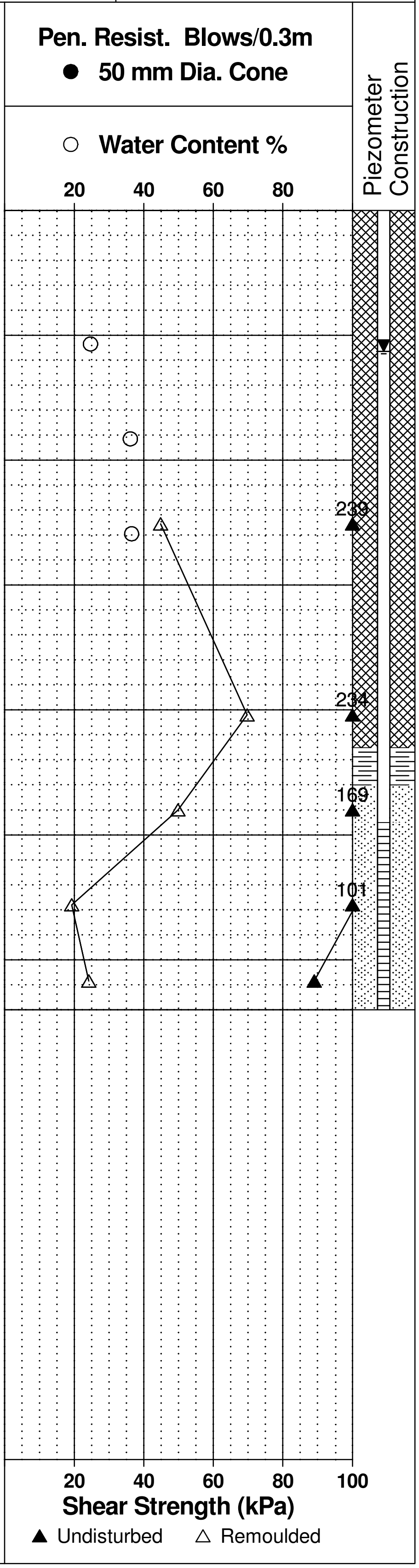
BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.36
Hard to very stiff, brown **SILTY CLAY,** some silty sand - sand content decreasing with depth	5.18
Stiff, grey **SILTY CLAY**	6.40
End of Borehole (GWL @ 1.13m - May 26, 2021)	

SAMPLE			
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	83	8
SS	3	83	5
SS	4	100	6

DEPTH (m)	ELEV. (m)
0	88.11
1	87.11
2	86.11
3	85.11
4	84.11
5	83.11
6	82.11



SOIL PROFILE AND TEST DATA

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Proposed Residential Development
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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

FILE NO. PG5828

HOLE NO. BH 5-21

SOIL DESCRIPTION

GROUND SURFACE

TOPSOIL 0.30

Compact, brown **SILTY SAND** 0.60

Hard to very stiff, brown **SILTY CLAY**

- stiff and grey by 4.3m depth

6.10

Dynamic Cone Penetration Test commenced at 6.10m depth. Cone pushed to 8.43m depth

8.84

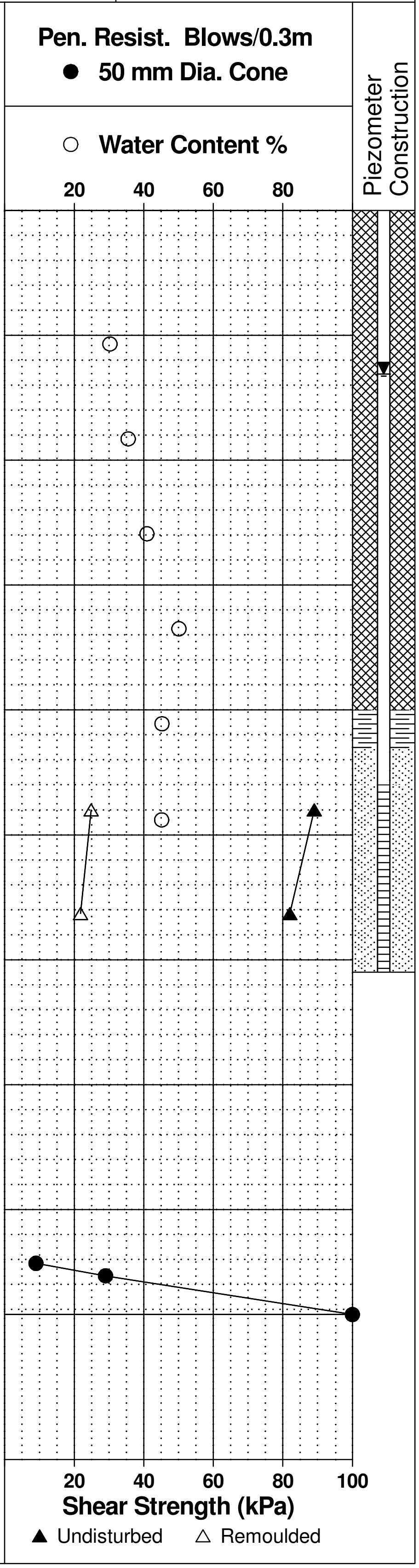
End of Borehole
Practical DCPT refusal at 8.84m depth.

(GWL @ 1.31m - May 26, 2021)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	10
SS	3	58	11
SS	4	83	9
SS	5	100	7
SS	6	100	5
SS	7	100	P

DEPTH (m)	ELEV. (m)
0	85.36
1	84.36
2	83.36
3	82.36
4	81.36
5	80.36
6	79.36
7	78.36
8	77.36



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SOIL PROFILE AND TEST DATA

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Proposed Residential Development
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DATUM Geodetic

REMARKS

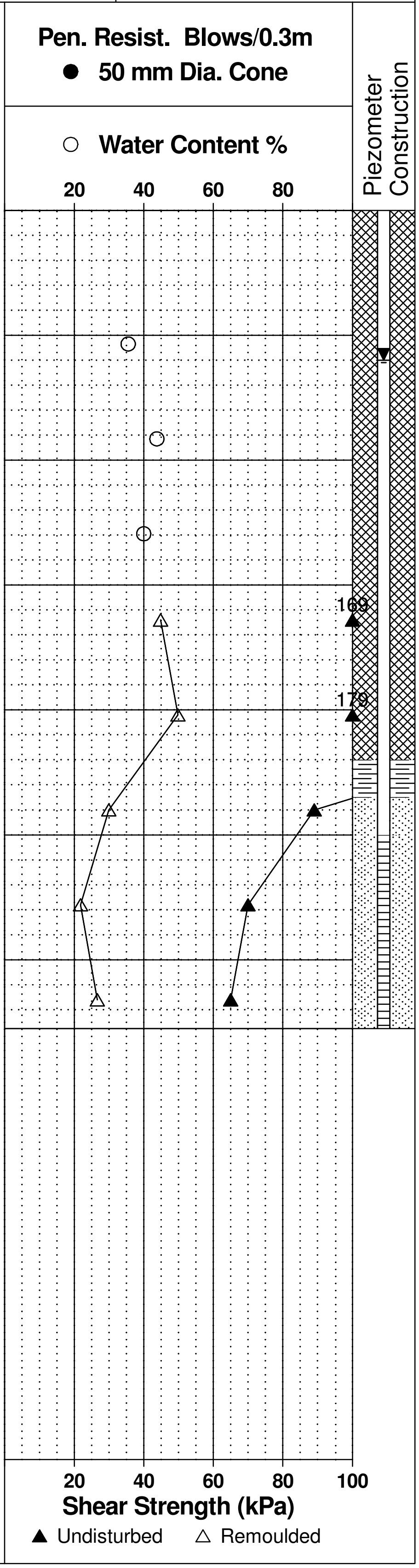
BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

FILE NO. PG5828

HOLE NO. BH 6-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	85.35
TOPSOIL 0.30						
Brown **SILTY SAND,** trace clay 0.60	AU	1				
	SS	2	83	6	1	84.35
	SS	3	83	6	2	83.35
Very stiff to stiff, brown **SILTY CLAY,** trace sand - sand content decreasing with depth	SS	4	83	5		
					3	82.35
					4	81.35
- grey by 4.6m depth						
					5	80.35
					6	79.35
6.55						
End of Borehole (GWL @ 1.20m - May 26, 2021)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. PG5828

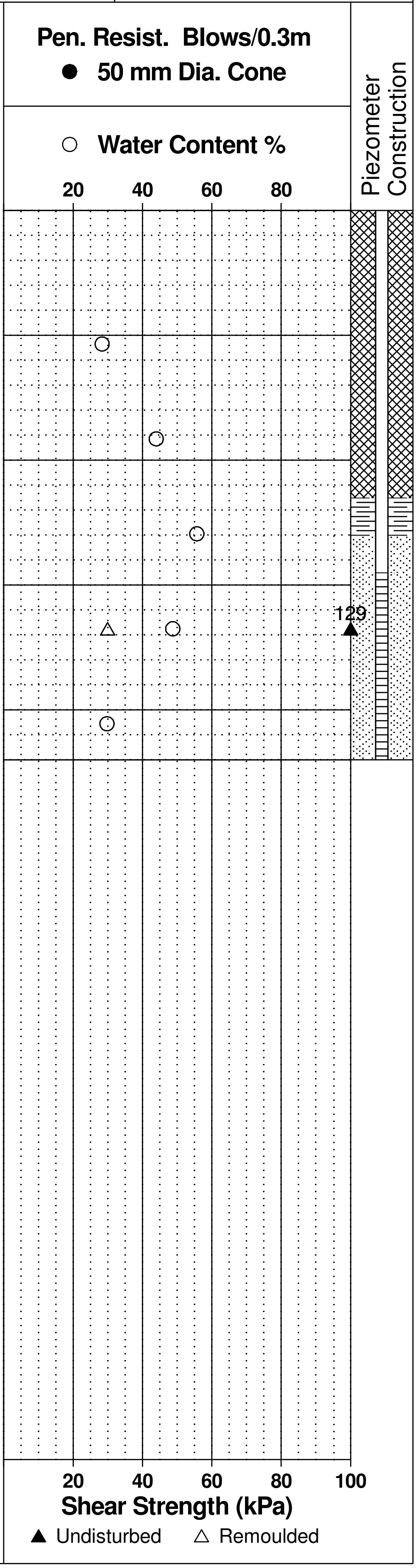
HOLE NO. BH 7-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	87.56
TOPSOIL 0.30						
Brown **SILTY SAND,** trace clay 0.76	AU	1				
Very stiff, brown **SILTY CLAY,** trace sand - sand content decreasing with depth	SS	2	75	7	1	86.56
	SS	3	83	7	2	85.56
	SS	4	83	3	3	84.56
- some sand, trace gravel by 4.1m depth	SS	5	100	P	4	83.56
4.40	SS	6	87	8		

End of Borehole

Practical refusal to augering at 4.4m depth

(BH dry - May 26, 2021)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

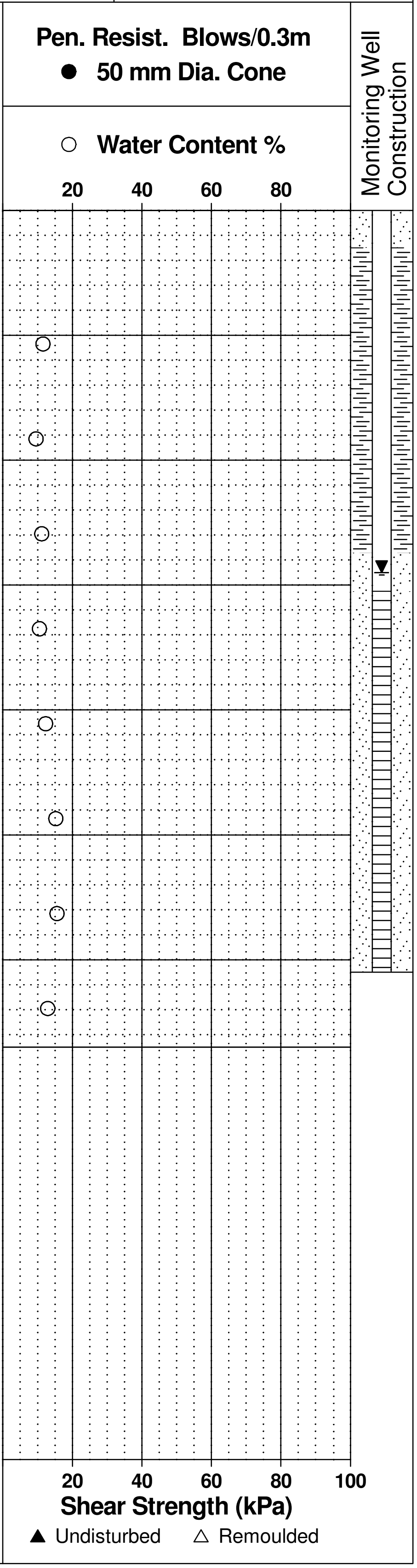
DATE May 20, 2021

FILE NO. PG5828

HOLE NO. BH 8-21

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.20				
FILL: Brown silty sand, some gravel, trace topsoil	AU	1		
1.07	SS	2	62	30
GLACIAL TILL: Dense to compact, brown silty sand with gravel, cobbles and boulders	SS	3	75	34
	SS	4	62	27
	SS	5	75	32
	SS	6	62	39
	SS	7	50	27
	SS	8	42	26
	SS	9	42	21
6.70				
End of Borehole (GWL @ 2.90m - May 26, 2021)				

DEPTH (m)	ELEV. (m)
0	91.32
1	90.32
2	89.32
3	88.32
4	87.32
5	86.32
6	85.32



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

REMARKS

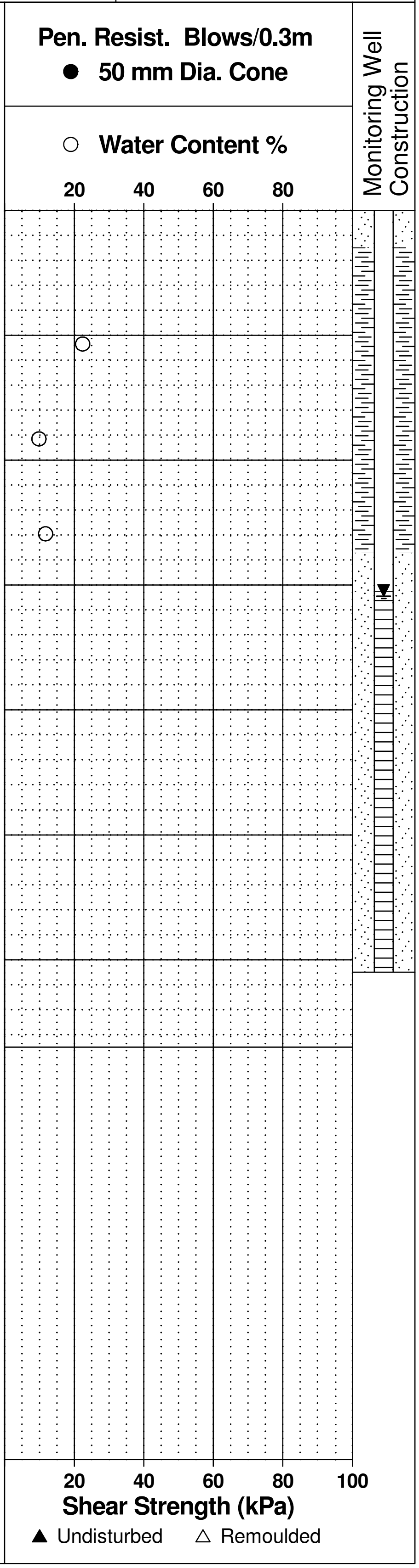
BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. PG5828

HOLE NO. BH 9-21

SOIL DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	90.52
TOPSOIL 0.30	■						
Stiff, brown **SILTY CLAY,** some to trace sand	▨	AU	1				
		SS	2	83	12	1	89.52
1.52							
						2	88.52
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders		SS	3	75	23	3	87.52
						4	86.52
						5	85.52
						6	84.52
6.70		SS	4	75	32		
End of Borehole (GWL @ 3.09m - May 26, 2021)							



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.

p - Push spoon sampling

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 = $(D30)^2 / (D10 \times D60)$
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)

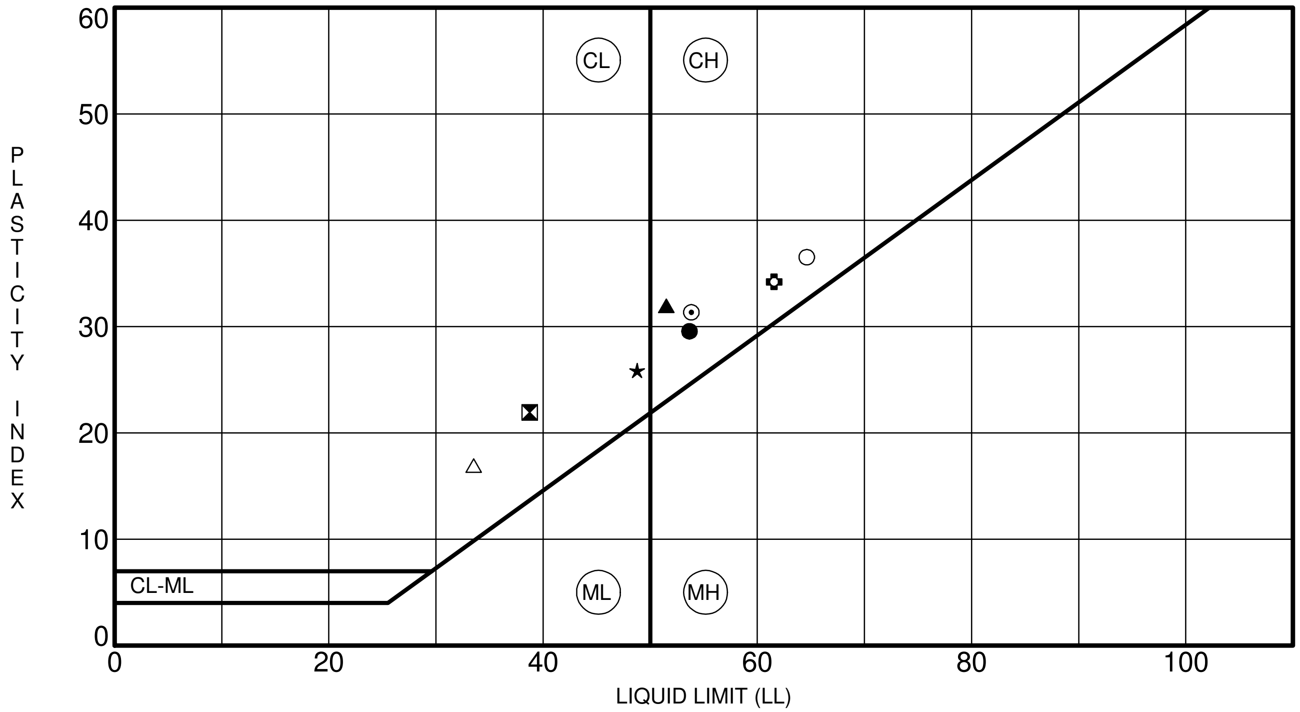
CONSOLIDATION TEST

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.



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 1-21 🔵 SS 3**	**54**	**24**	**30**		**CH - Inorganic clays of high plasticity**
⧗	**BH 2-21 🟠 SS 2**	**39**	**17**	**22**		**CL - Inorganic clays of low plasticity**
▲	**BH 3-21 🟢 SS 4**	**51**	**20**	**32**		**CH - Inorganic clays of high plasticity**
★	**BH 4-21 🟡 SS 3**	**49**	**23**	**26**		**CL - Inorganic clays of low plasticity**
⊙	**BH 5-21 🟣 SS 2**	**54**	**22**	**31**		**CH - Inorganic clays of high plasticity**
✤	**BH 6-21 ● SS 3**	**62**	**27**	**34**		**CH - Inorganic clays of high plasticity**
○	**BH 7-21 🔴 SS 4**	**65**	**28**	**37**		**CH - Inorganic clays of high plasticity**
△	**BH 9-21 🟤 SS 2**	**34**	**17**	**17**		**CL - Inorganic clays of low plasticity**

CLIENT **Uniform Urban Developments**

PROJECT **Geotechnical Investigation - Proposed Residential Development**

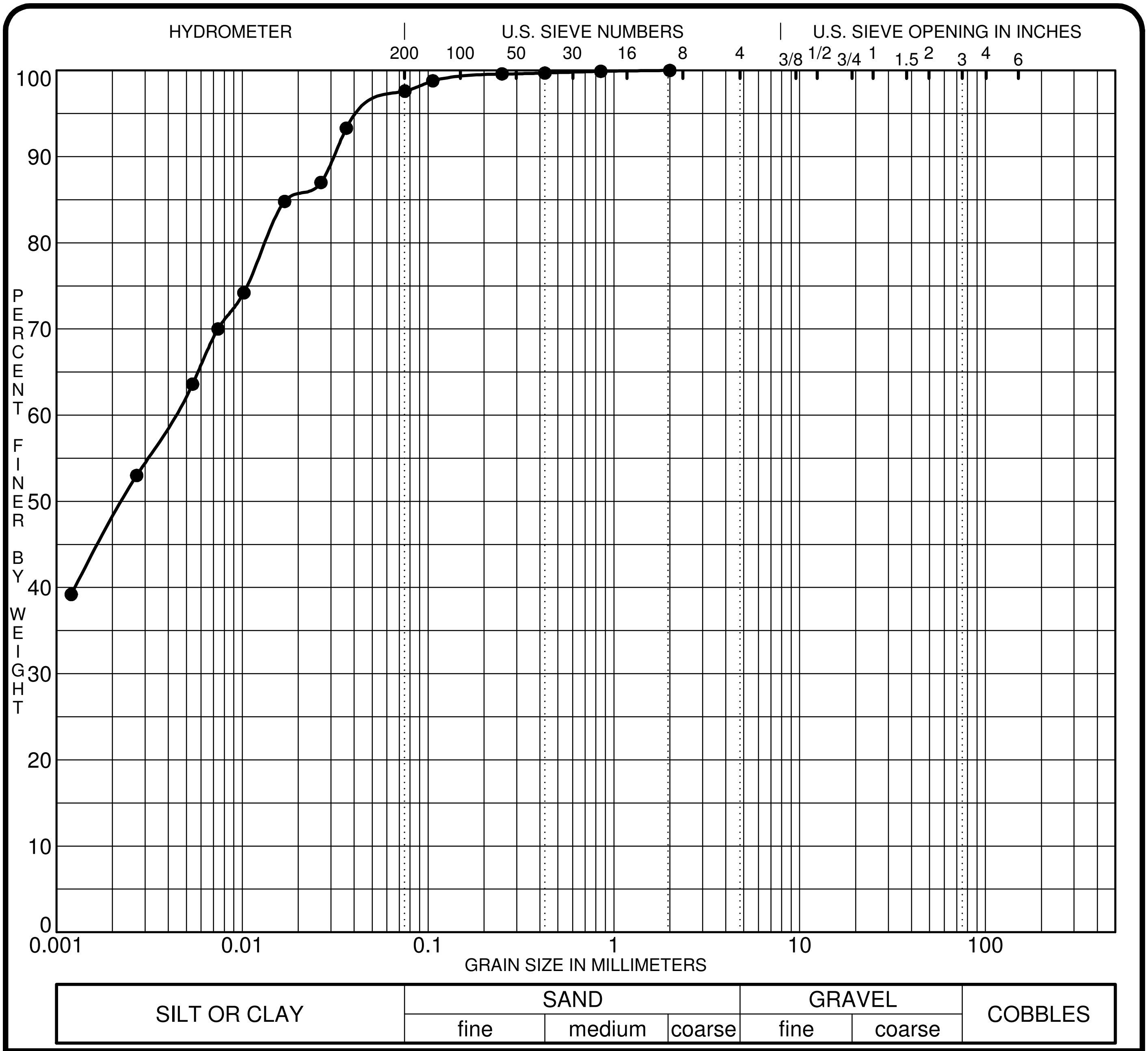
FILE NO. **PG5828**

DATE **20 May 21**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario, K2E 7T9

ATTERBERG LIMITS' RESULTS



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 1-21**	**SS 4**	**CH - Inorganic clays of high plasticity**						
☒									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 1-21**	**SS 4**	**2.00**	**0.00**			**0.0**	**2.4**	**97.6**	
☒										
▲										
★										

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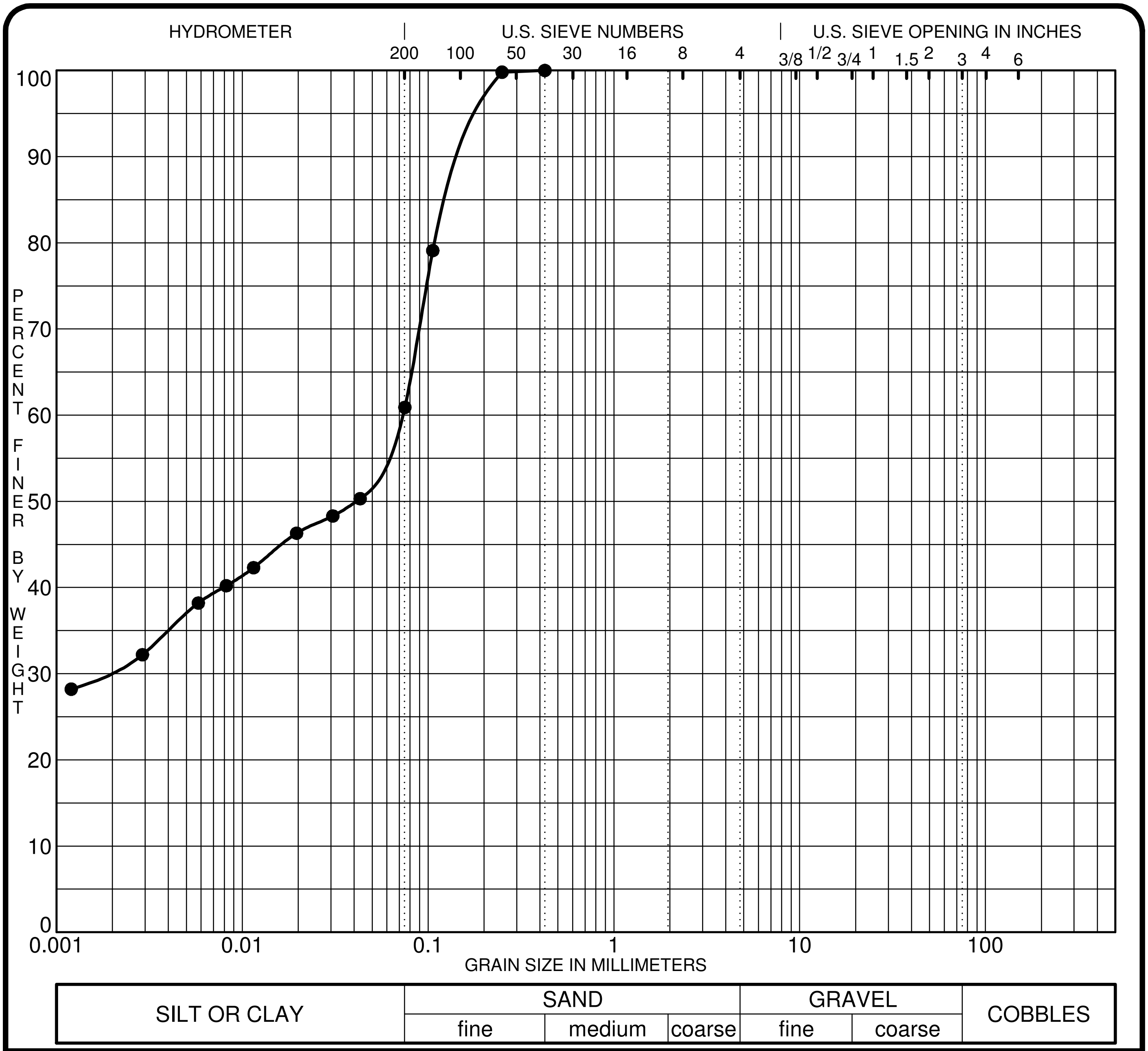
GRAIN SIZE DISTRIBUTION



Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 4-21 SS 2**	**CL - Inorganic clays of low plasticity**						
⊠								
▲								
★								

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 4-21 SS 2**	**0.43**	**0.07**	**0.002**		**0.0**	**39.1**	**60.9**	
⊠									
▲									
★									

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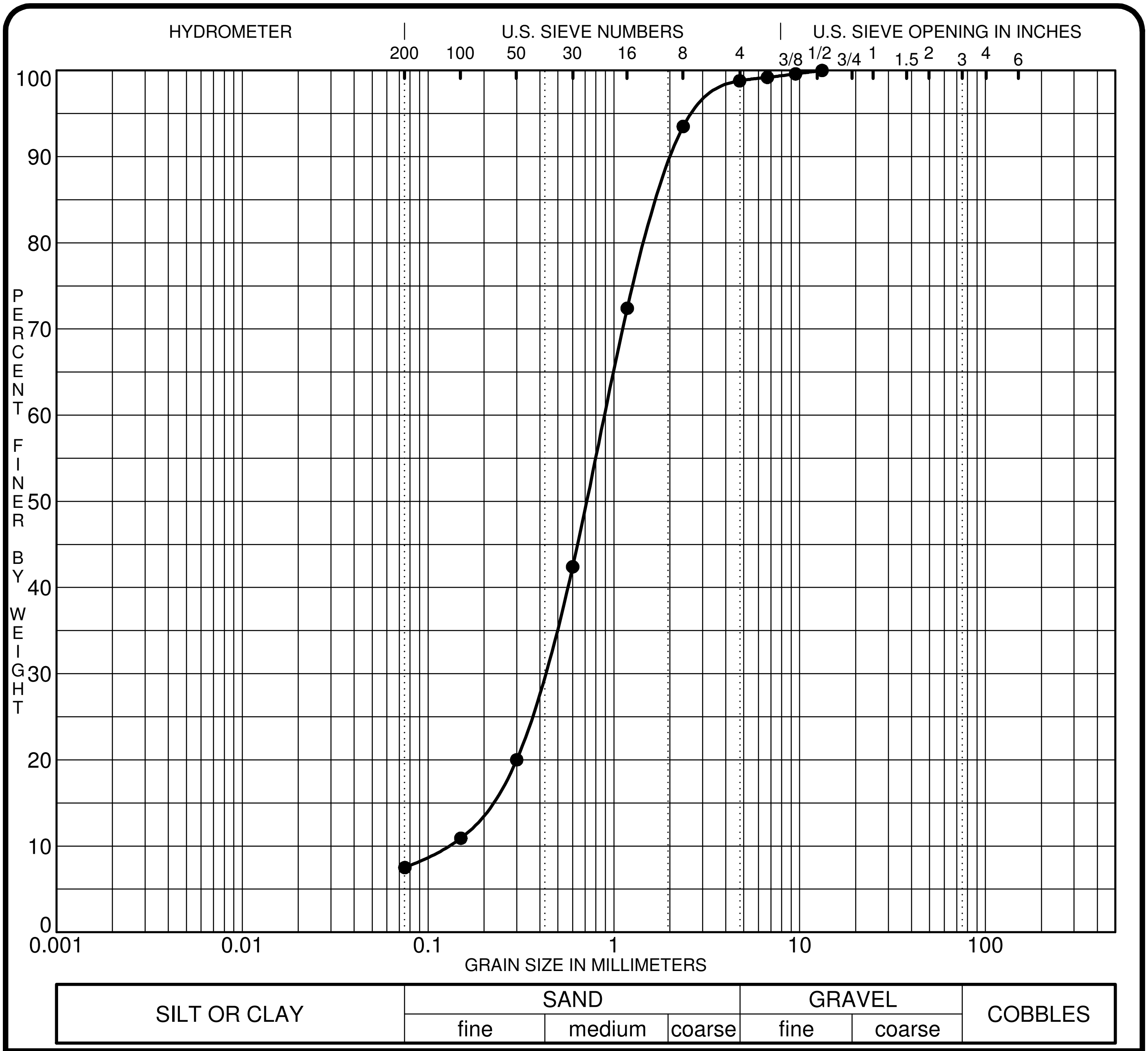
GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 6-21 SS 4**	**CH - Inorganic clays of high plasticity**					**1.50**	**7.1**
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 6-21 SS 4**	**13.20**	**0.89**	**0.409**	**0.1249**	**1.2**	**91.3**	**7.5**	
☒									
▲									
★									

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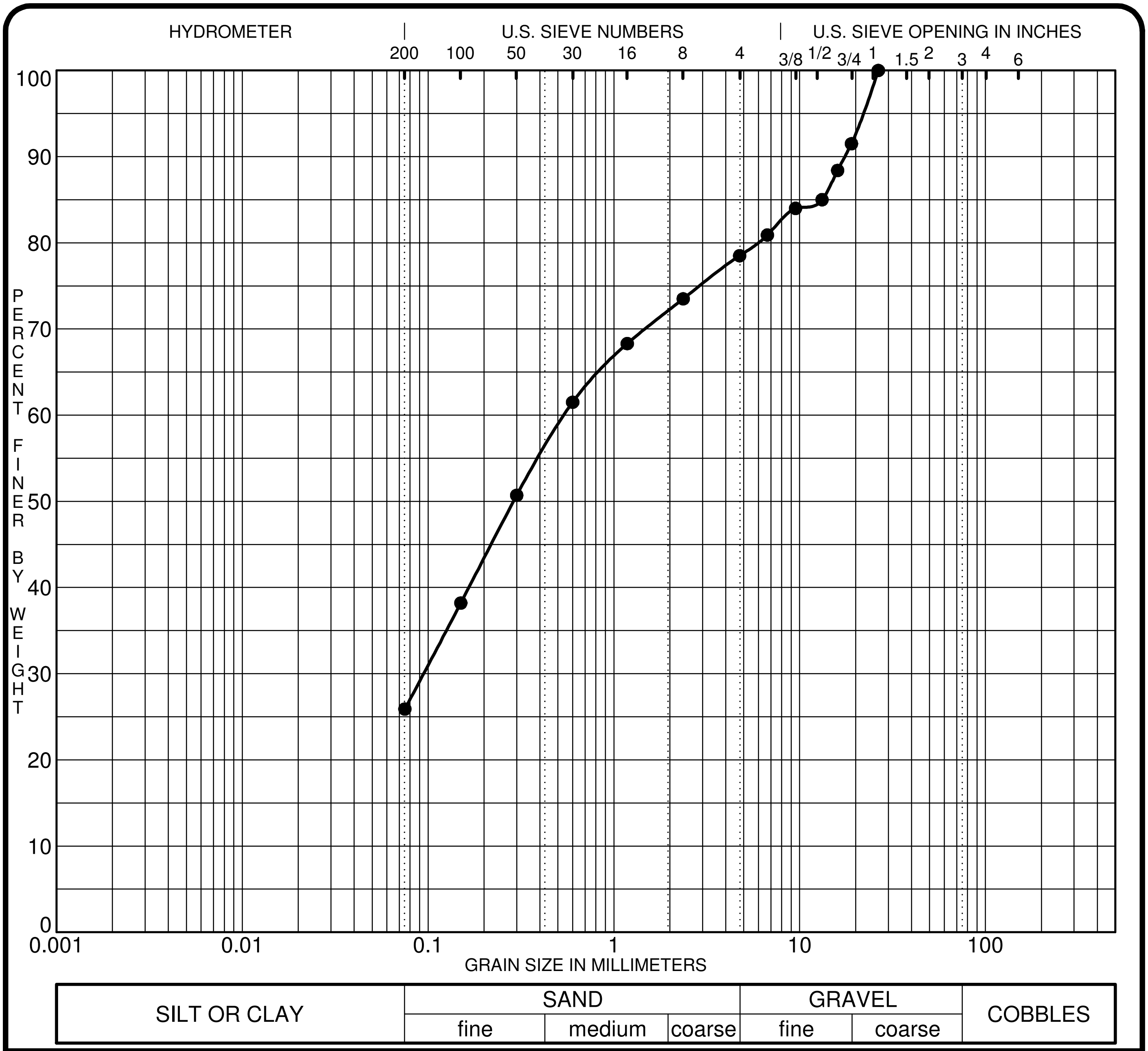
GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 9-21**	**SS 3**							
⌧									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 9-21**	**SS 3**	**26.50**	**0.54**	**0.094**		**21.5**	**52.6**	**25.9**	
⌧										
▲										
★										

CLIENT **Uniform Urban Developments**
FILE NO. **PG5828**
PROJECT **Geotechnical Investigation - Proposed Residential Development**
DATE **20 May 21**

patersongroup **Consulting Engineers**
9 Auriga Drive, Ottawa, Ontario, K2E 7T9

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29757

Report Date: 28-May-2021

Order Date: 21-May-2021

Project Description: PE5828

	Client ID:	BH3-21, SS3	-	-	-
	Sample Date:	20-May-21 09:00	-	-	-
	Sample ID:	2121708-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.54	-	-	-
Resistivity	0.10 Ohm.m	59.3	-	-	-
Anions					
Chloride	5 ug/g dry	9	-	-	-
Sulphate	5 ug/g dry	23	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 TO FIGURE 17 – SLOPE STABILITY ANALYSIS CROSS SECTIONS

PHOTOGRAPHS FROM SITE VISIT – MAY 19, 2021

DRAWING PG5828-1 – TEST HOLE LOCATION PLAN

DRAWING PG5828-3 – PERMISSIBLE GRADE RAISE PLAN

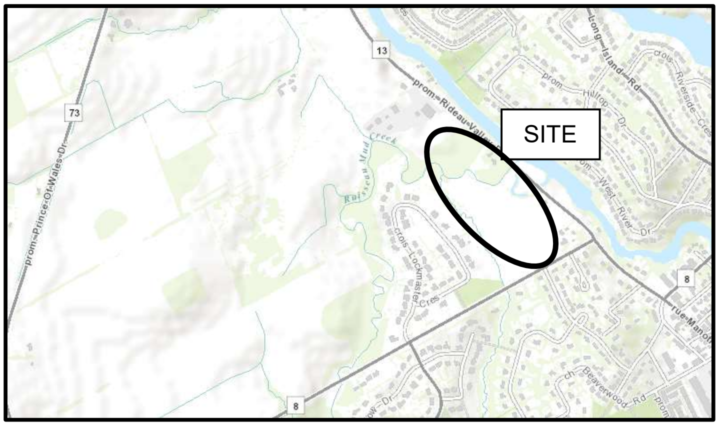


FIGURE 1

KEY PLAN

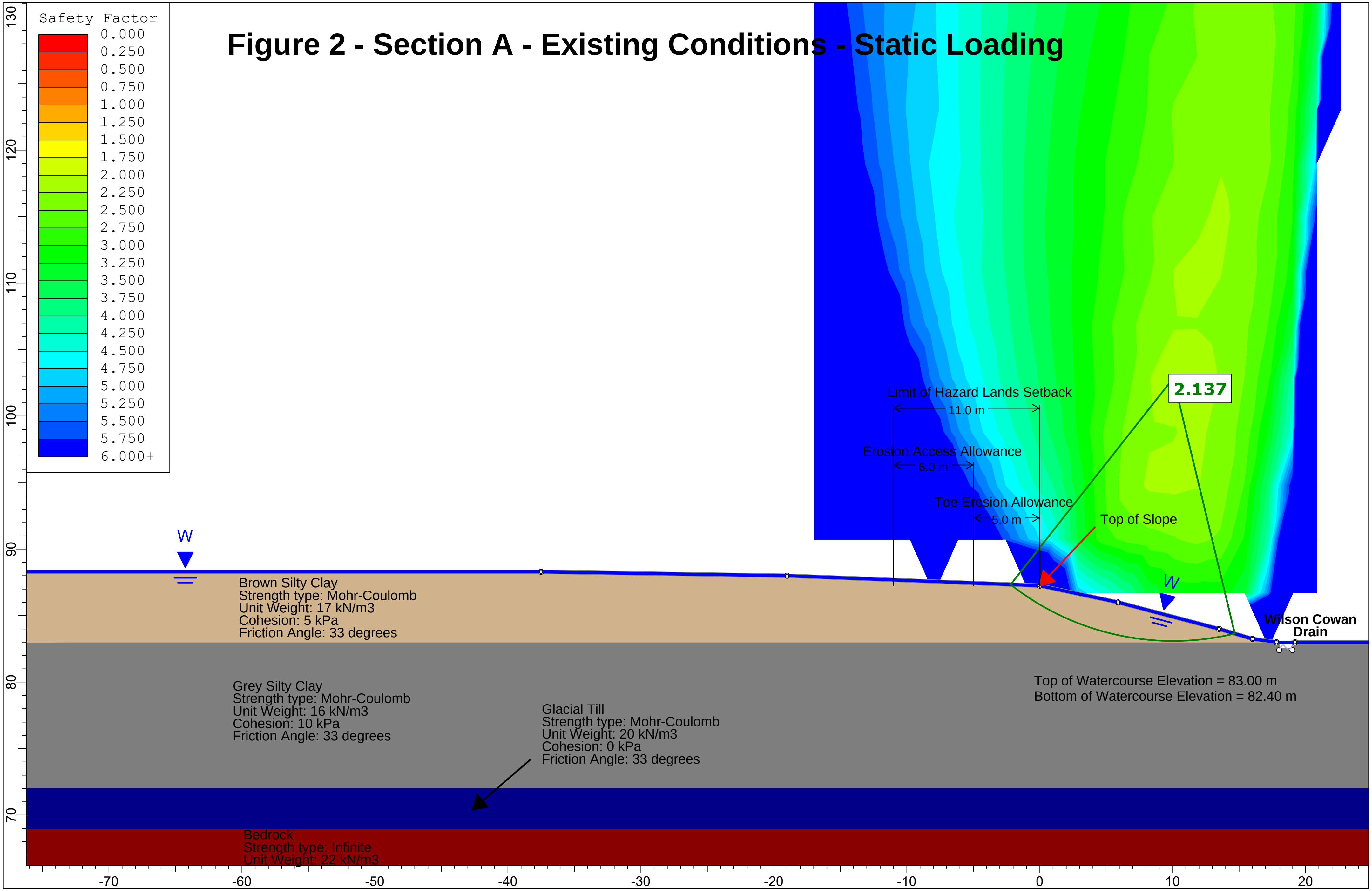


Figure 2 - Section A - Existing Conditions - Static Loading

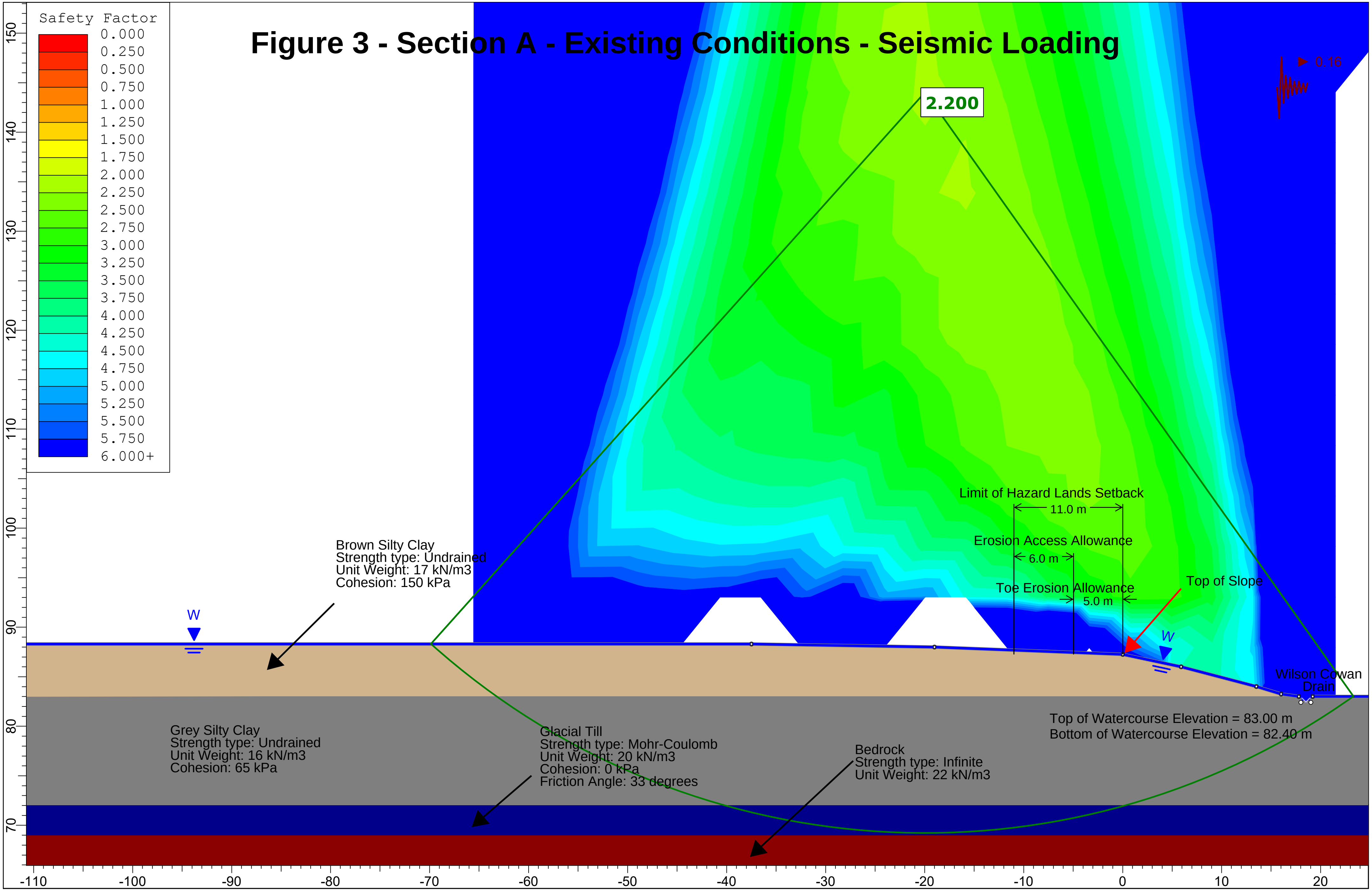


Figure 3 - Section A - Existing Conditions - Seismic Loading

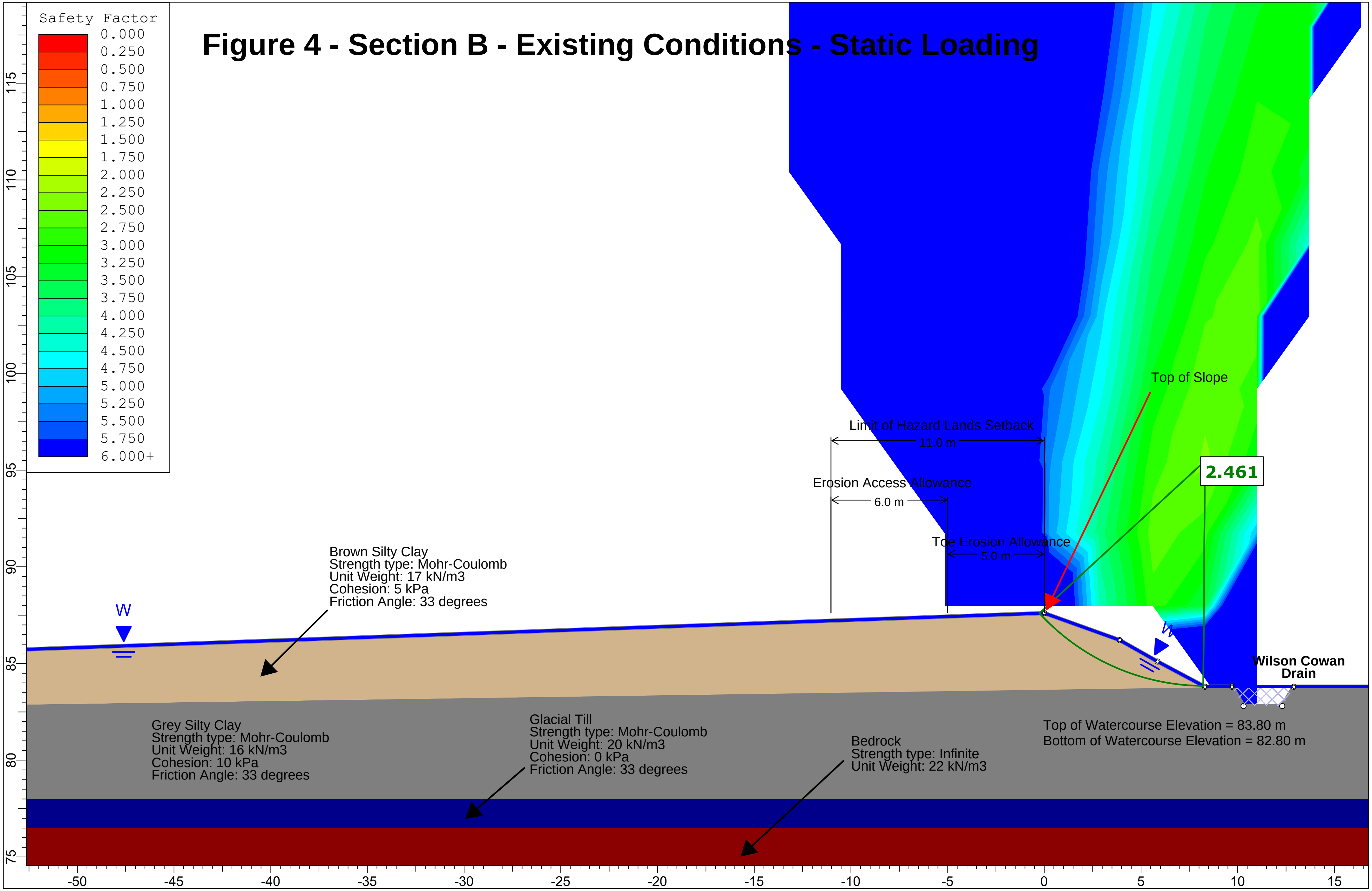
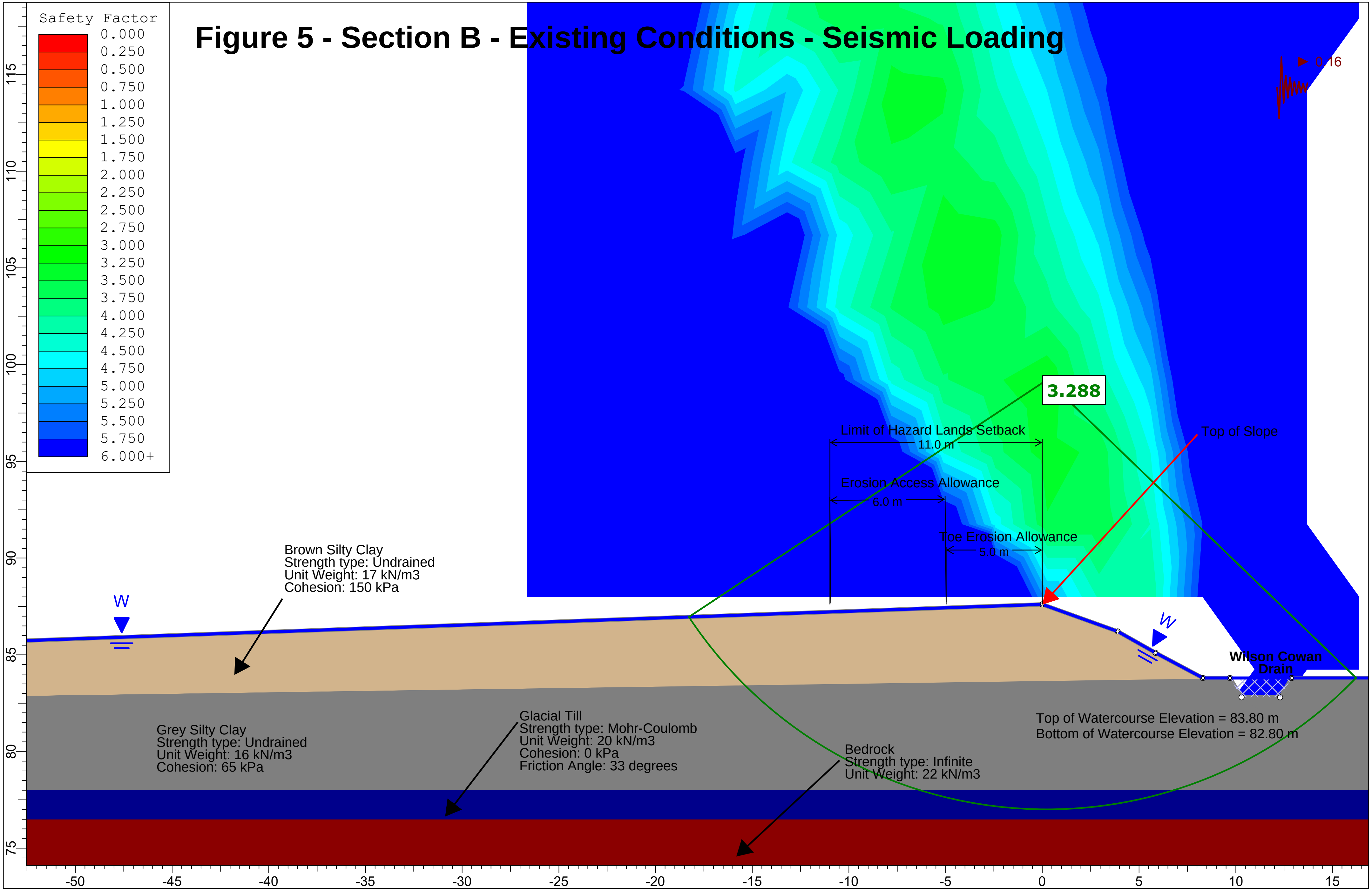


Figure 4 - Section B - Existing Conditions - Static Loading

Figure 5 - Section B - Existing Conditions - Seismic Loading



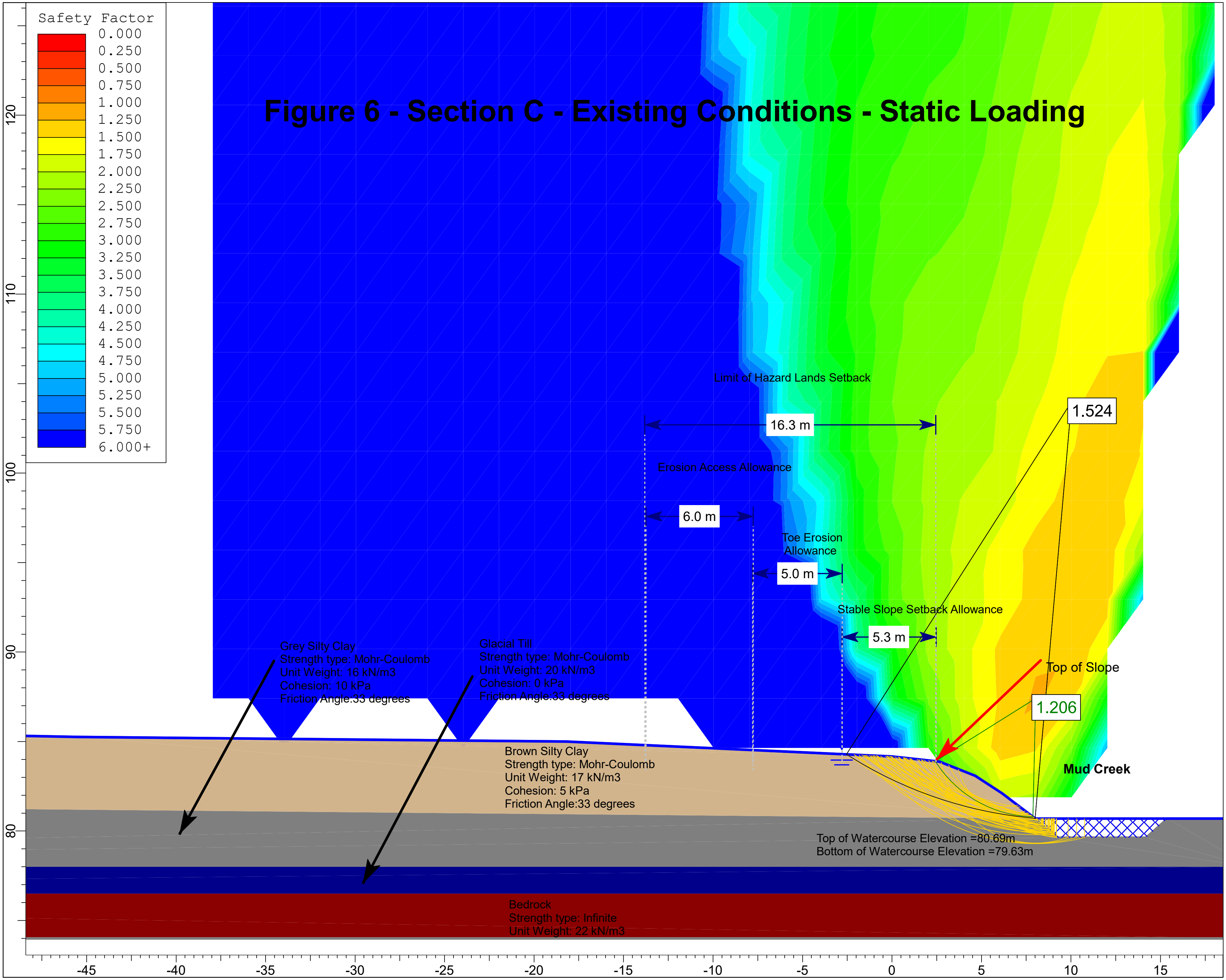
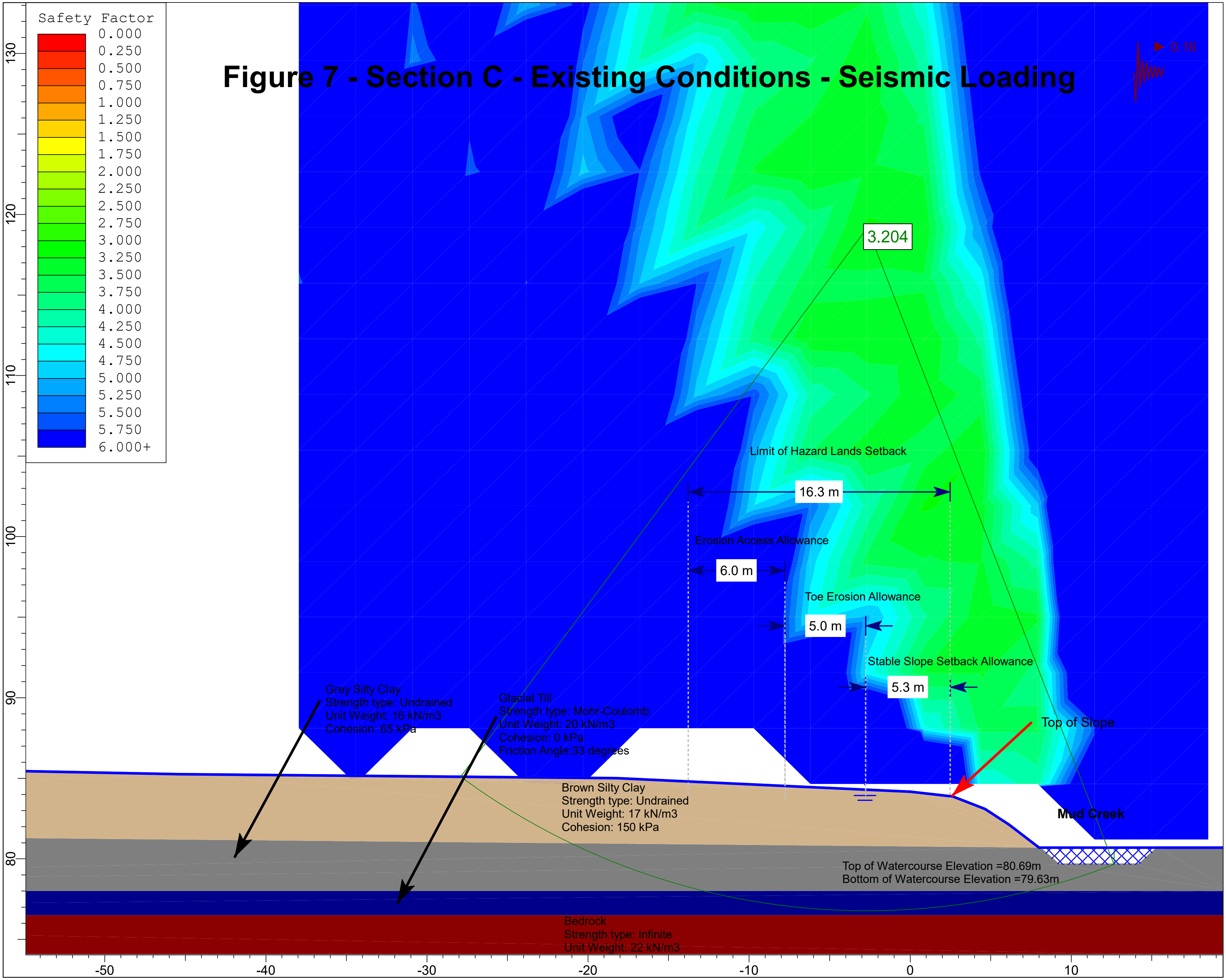


Figure 6 - Section C - Existing Conditions - Static Loading

Figure 7 - Section C - Existing Conditions - Seismic Loading



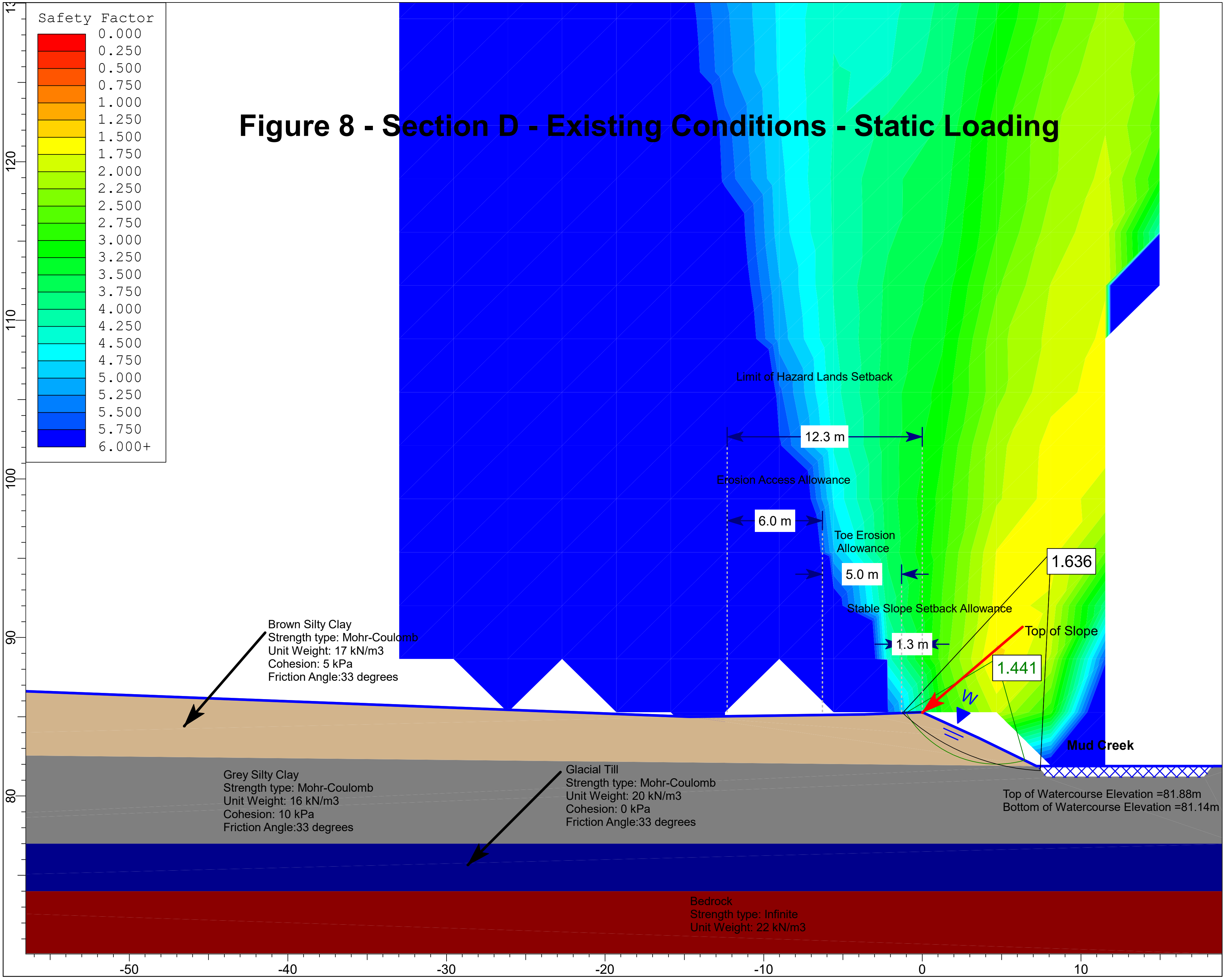


Figure 8 - Section D - Existing Conditions - Static Loading

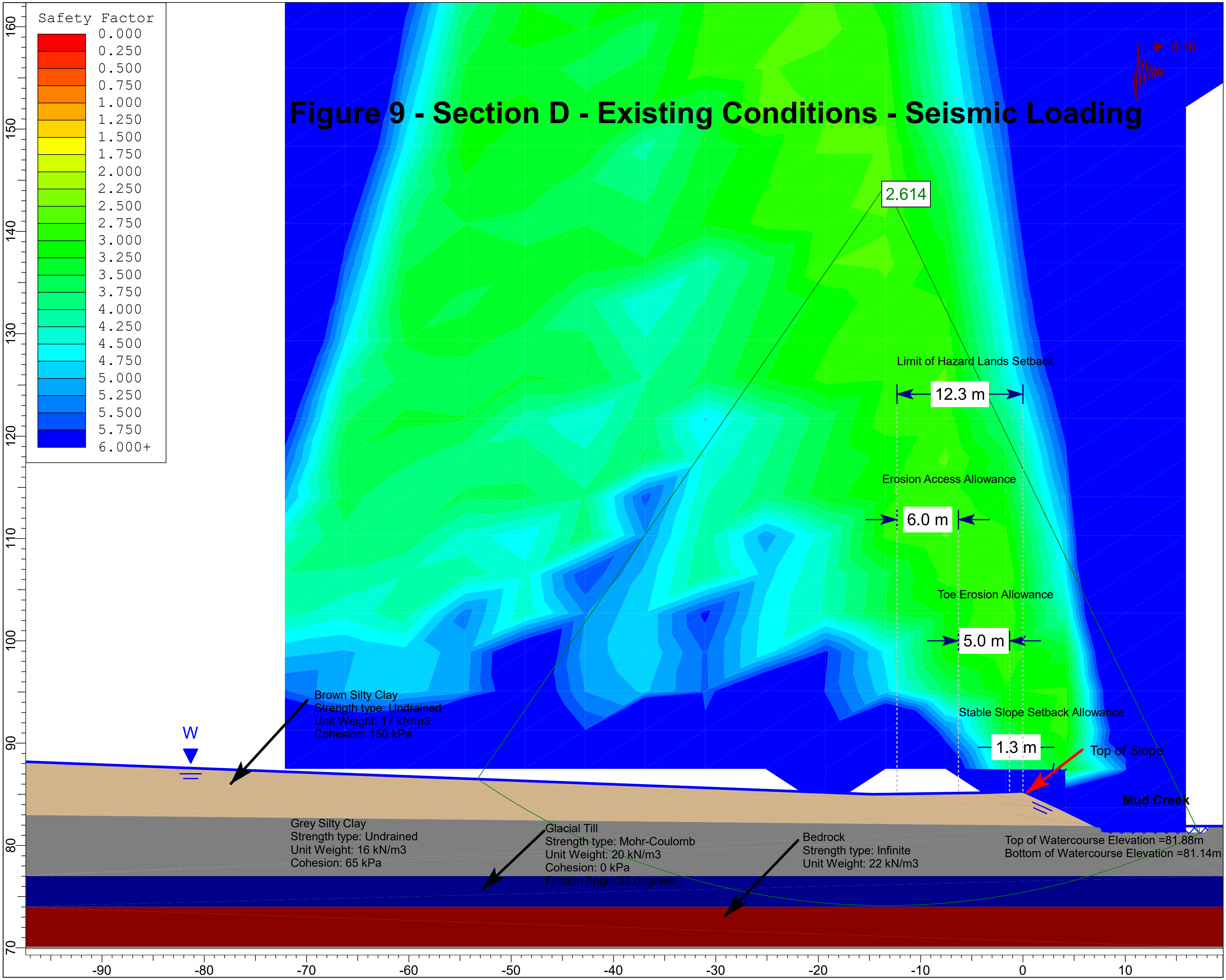
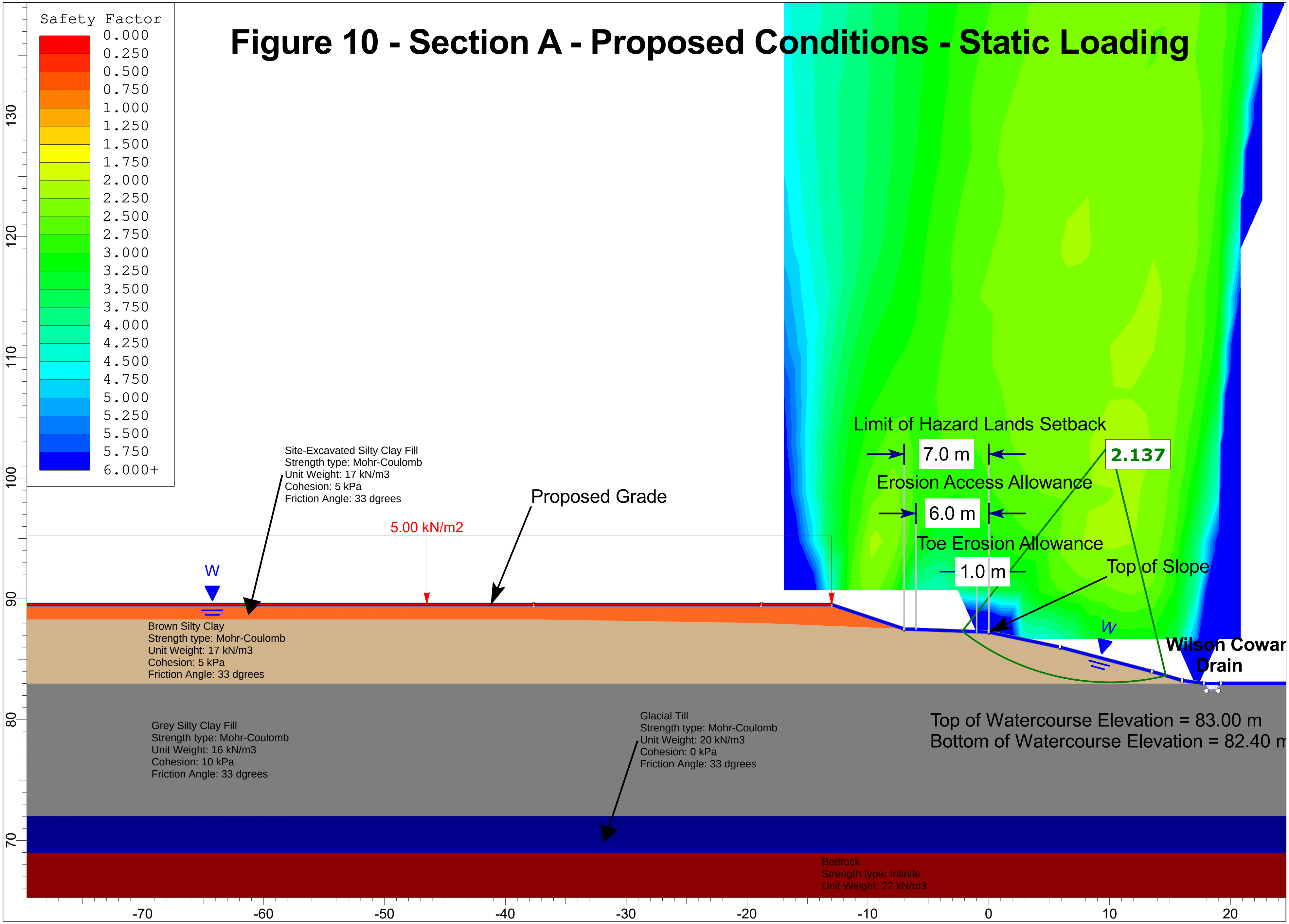


Figure 9 - Section D - Existing Conditions - Seismic Loading

Figure 10 - Section A - Proposed Conditions - Static Loading



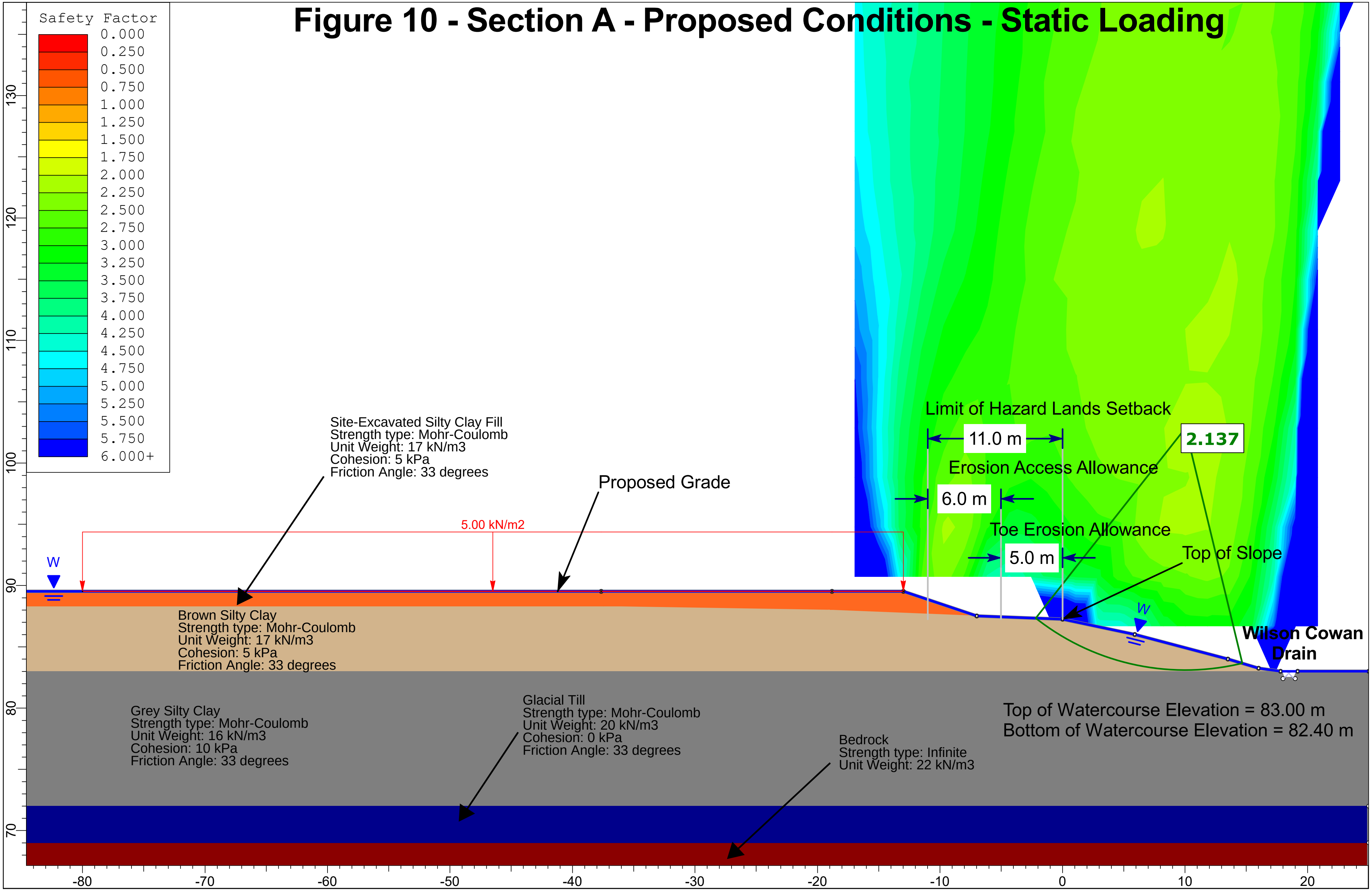
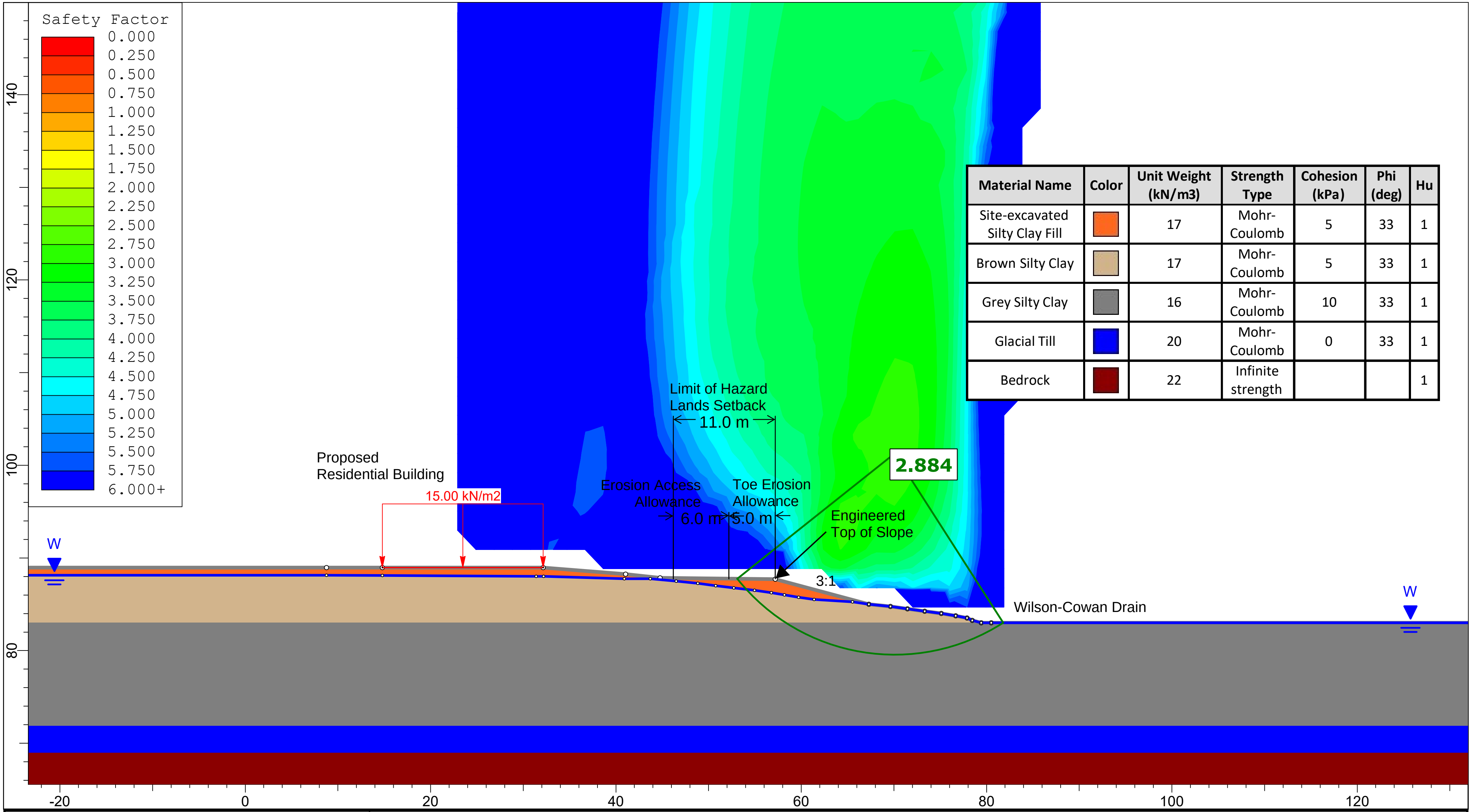


Figure 10 - Section A - Proposed Conditions - Static Loading

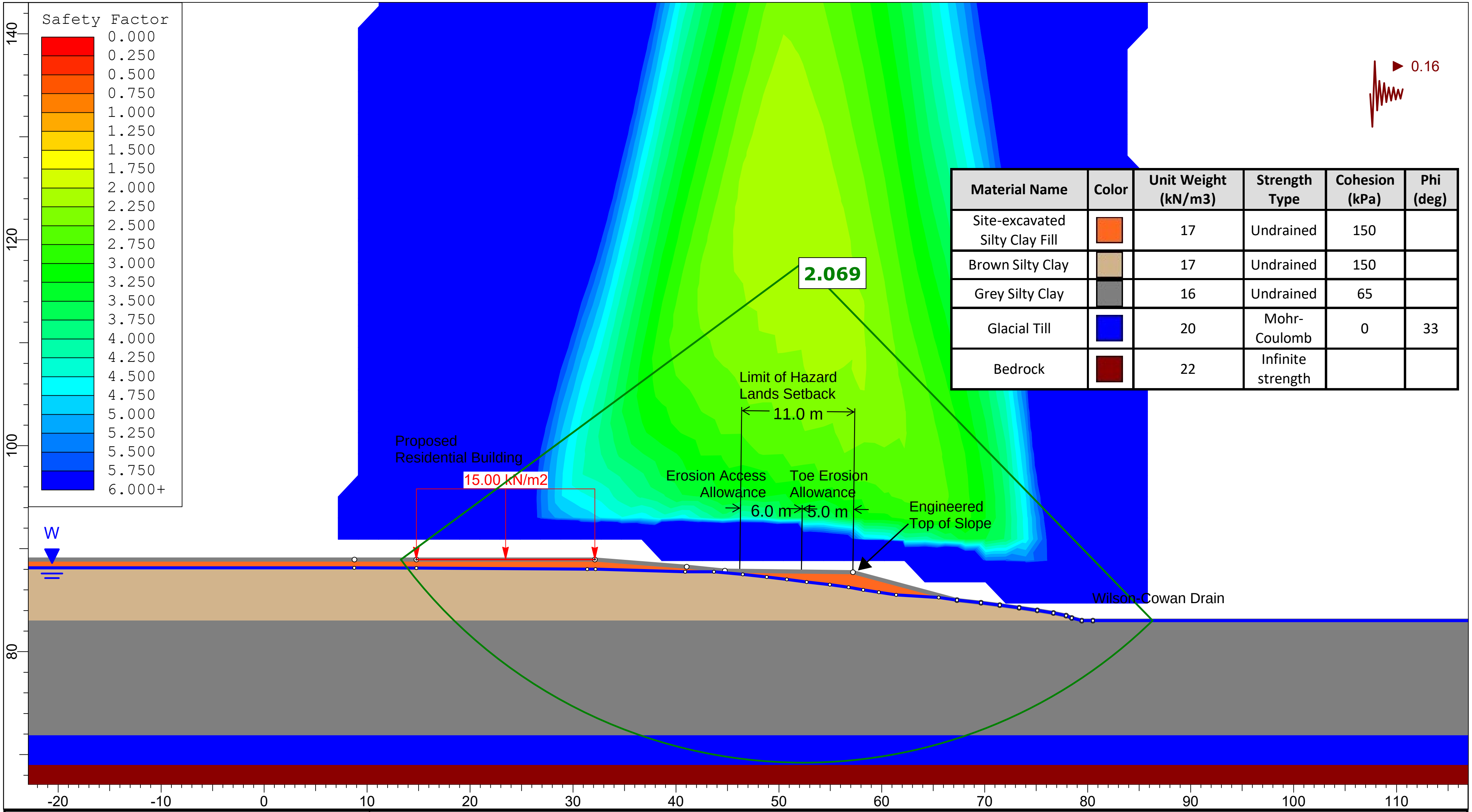


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Hu
Site-excavated Silty Clay Fill	🟧	17	Mohr-Coulomb	5	33	1
Brown Silty Clay	🟫	17	Mohr-Coulomb	5	33	1
Grey Silty Clay	⬛	16	Mohr-Coulomb	10	33	1
Glacial Till	🟦	20	Mohr-Coulomb	0	33	1
Bedrock	🟥	22	Infinite strength			1

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG5828 - 4386 Rideau Valley Drive, Ottawa, ON		
Group	Figure 2A - Section A-A - Static Analysis		
Drawn By	MA	*Company*	Uniform Developments
Date	2024-07-12	*File Name*	Slope Stability Assessment



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Site-excavated Silty Clay Fill	🟧	17	Undrained	150	
Brown Silty Clay	🟫	17	Undrained	150	
Grey Silty Clay	⬛	16	Undrained	65	
Glacial Till	🟦	20	Mohr-Coulomb	0	33
Bedrock	🟥	22	Infinite strength		

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG5828 - 4386 Rideau Valley Drive, Ottawa, ON
Group	Figure 2B - Section A-A - Seismic Analysis
Drawn By	MA
Company	Uniform Developments
Date	2024-07-12
File Name	Slope Stability Assessment

Figure 11 - Section A - Proposed Conditions - Seismic Loading

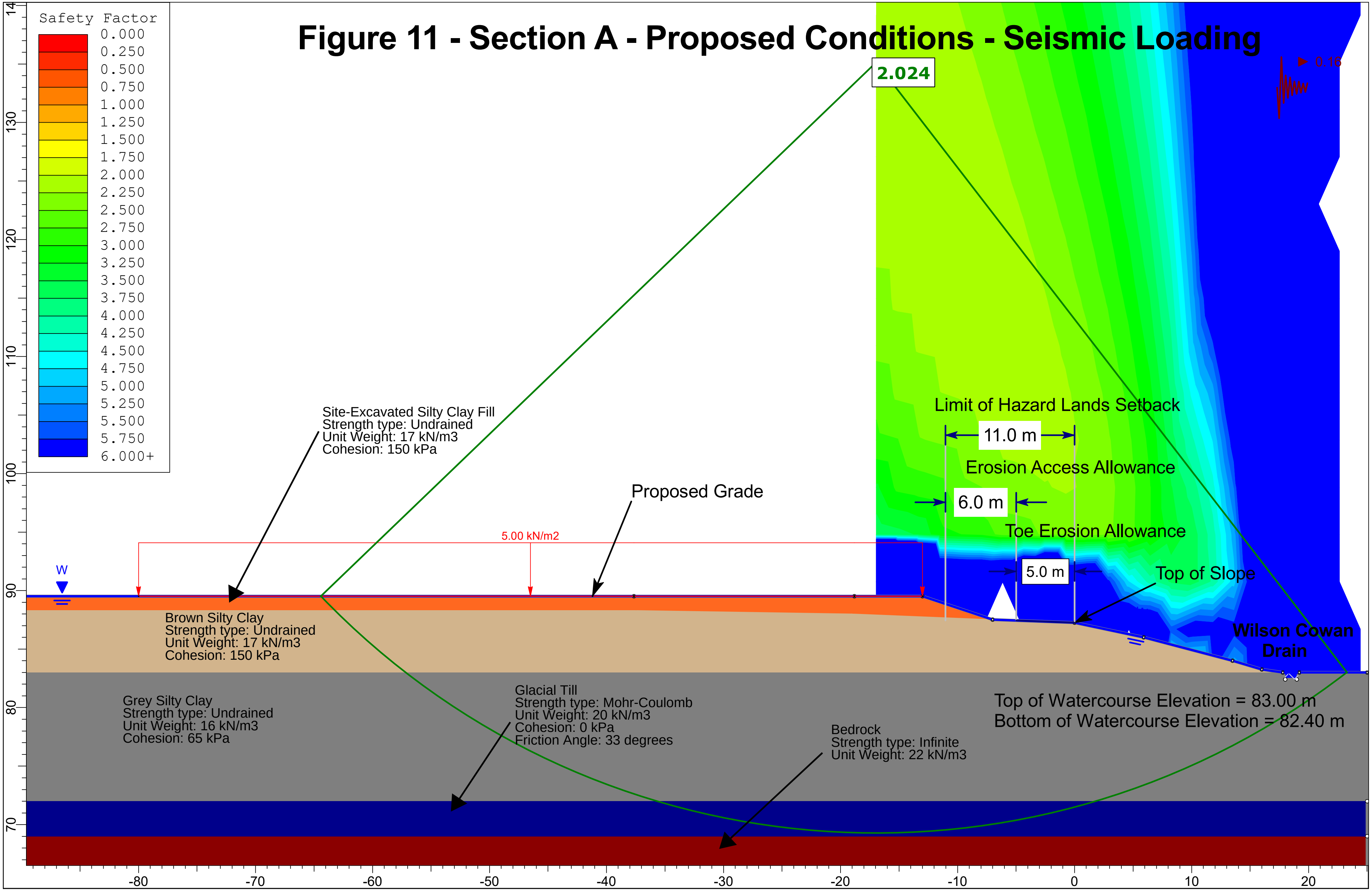


Figure 12 - Section B - Proposed Conditions - Static Loading

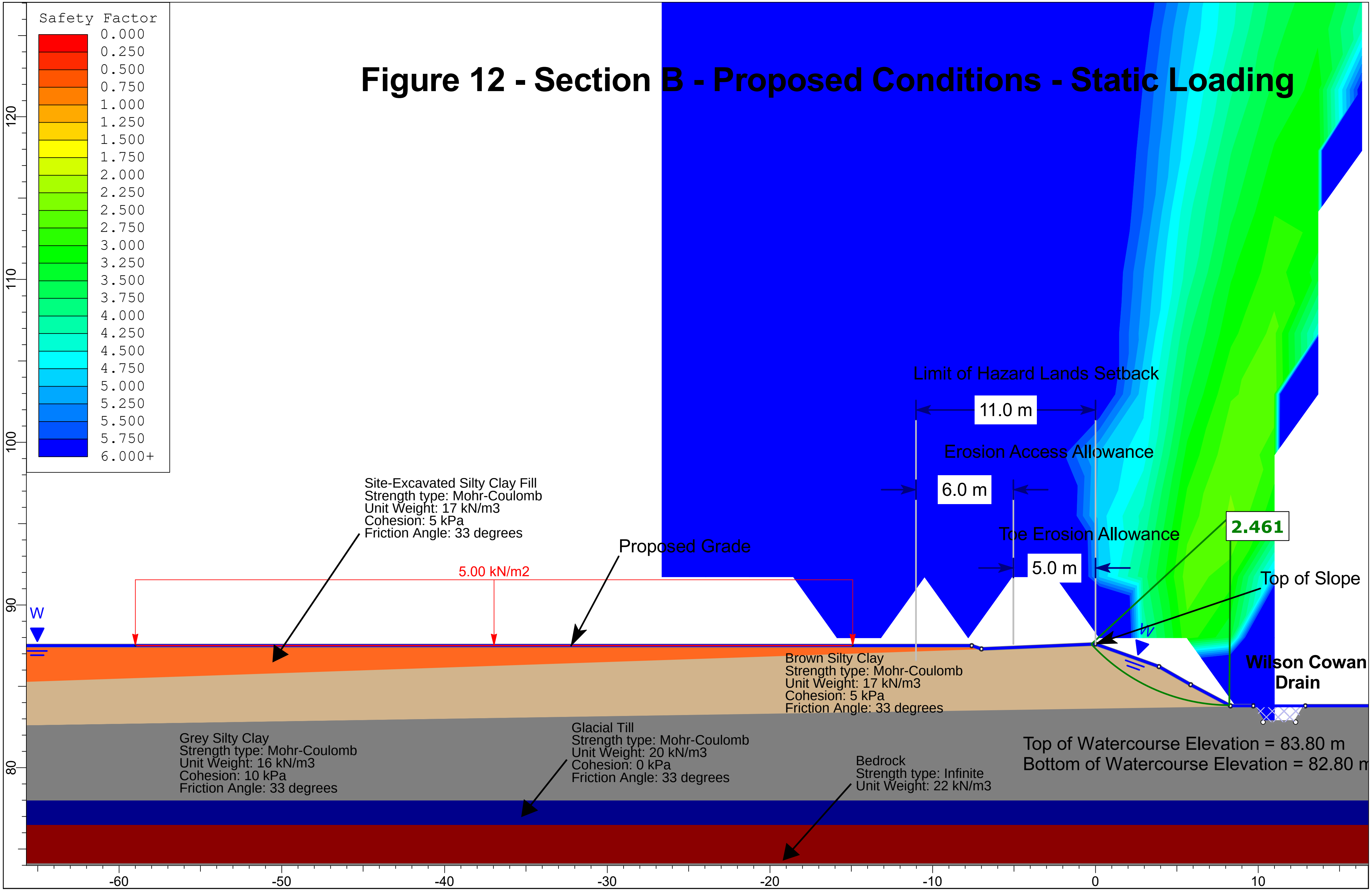
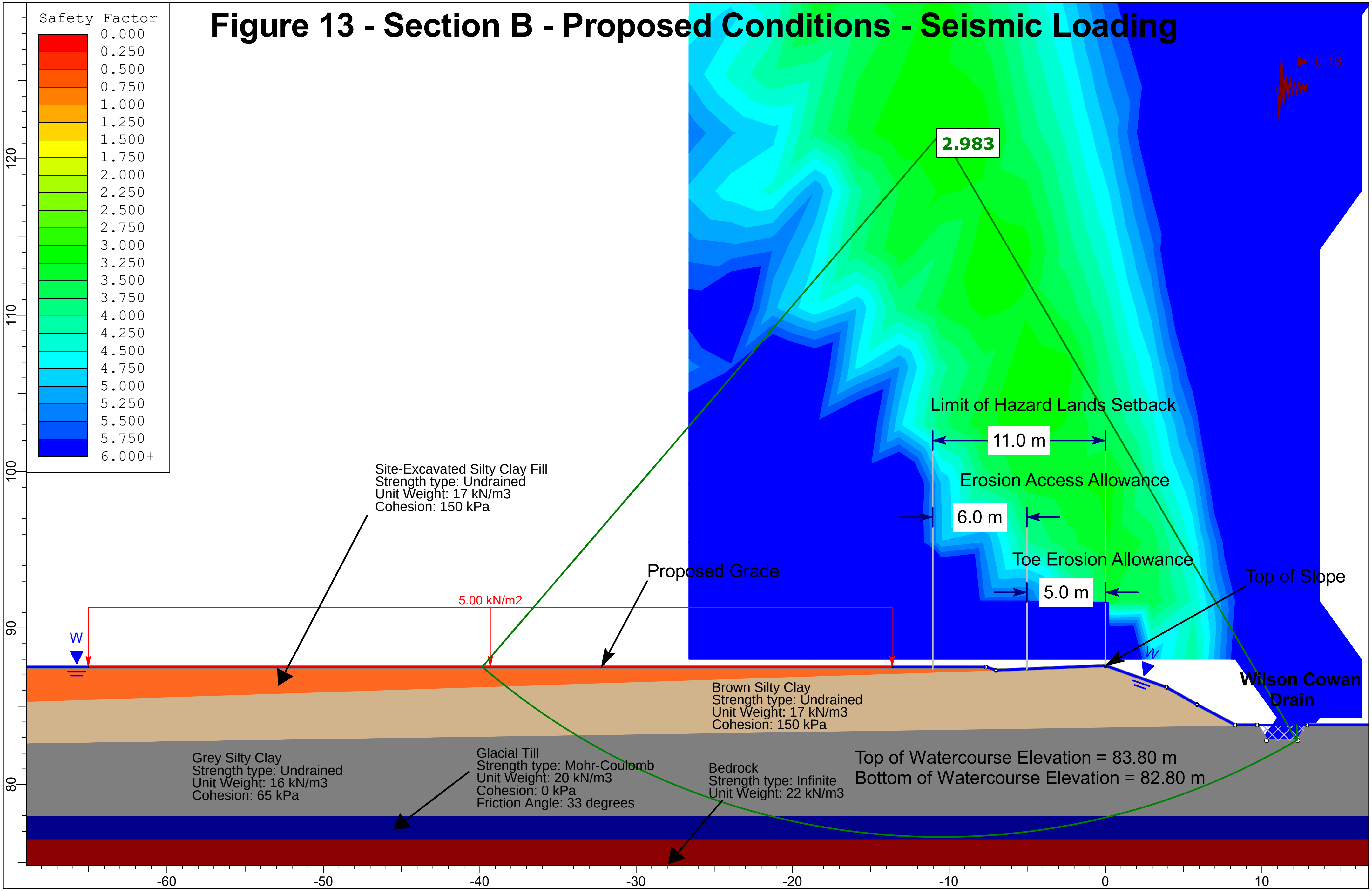


Figure 13 - Section B - Proposed Conditions - Seismic Loading



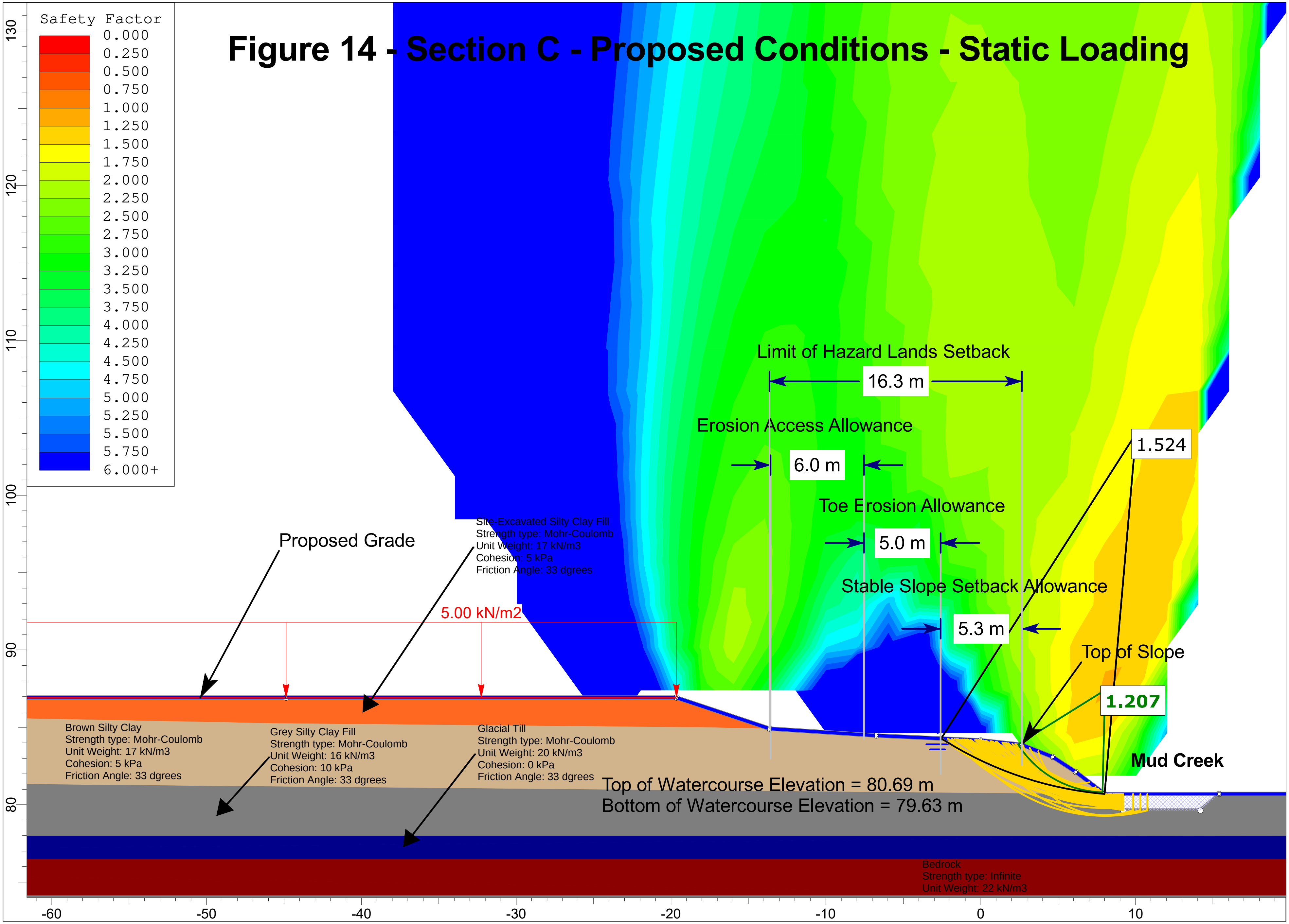


Figure 14 - Section C - Proposed Conditions - Static Loading

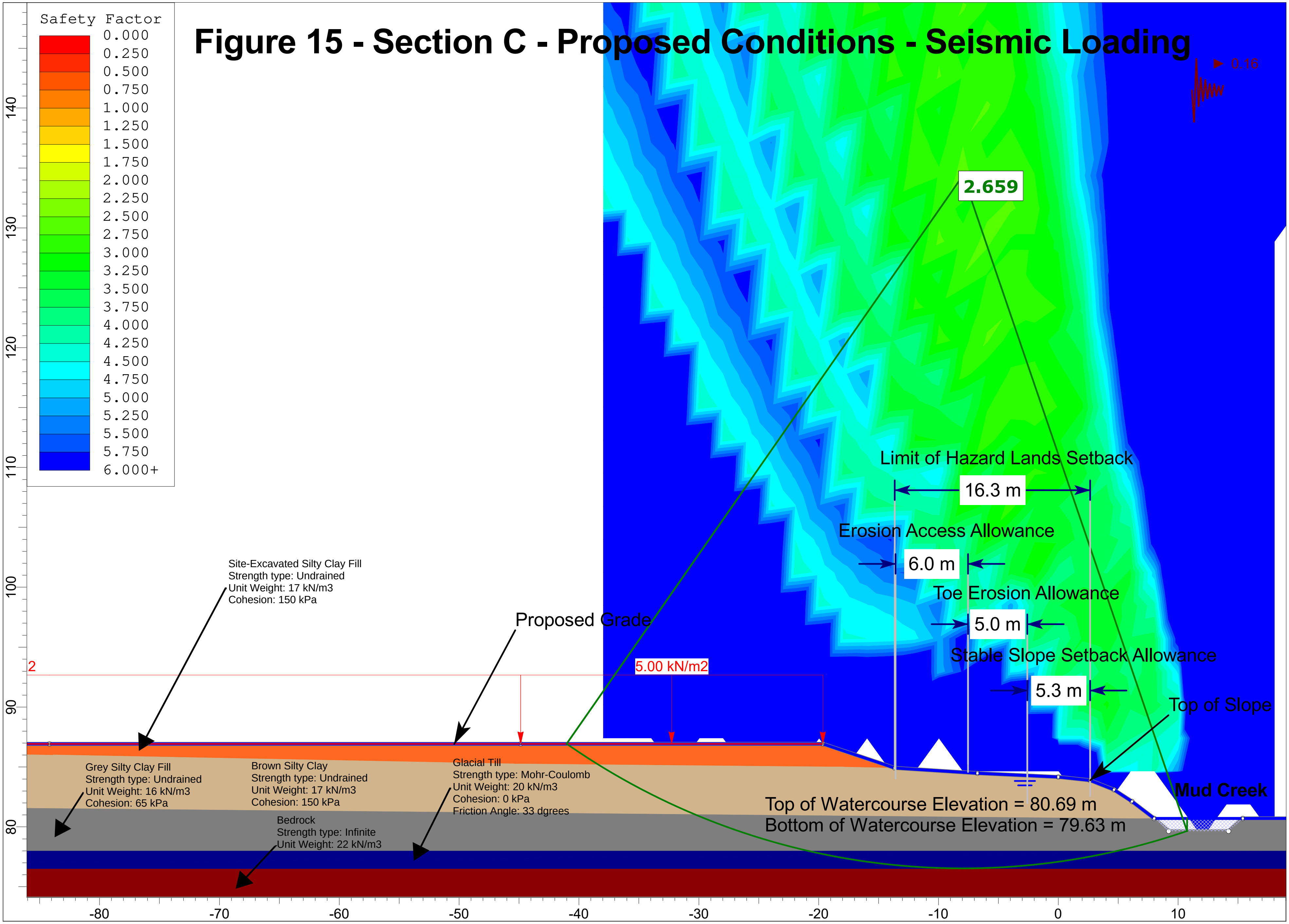


Figure 15 - Section C - Proposed Conditions - Seismic Loading

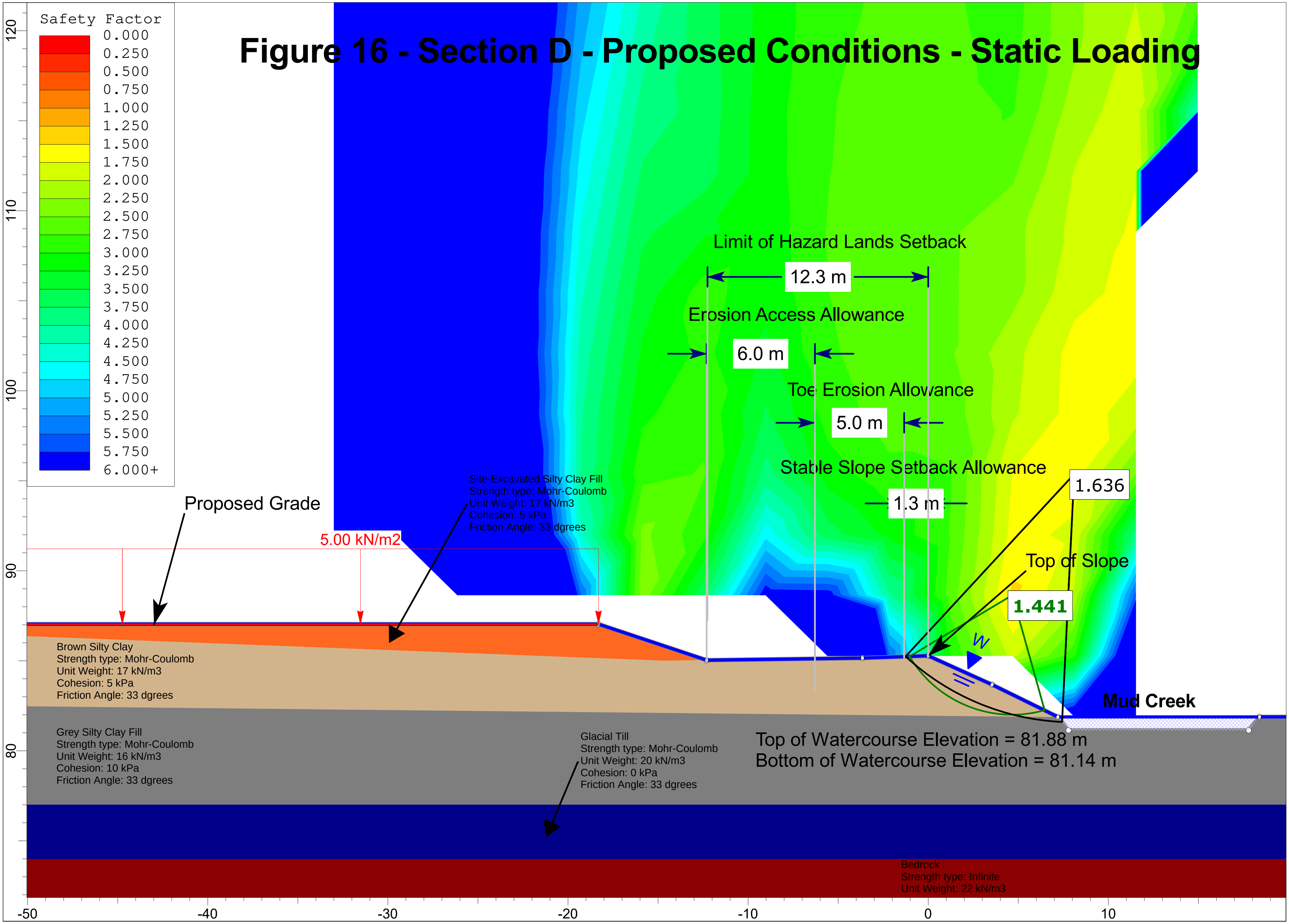


Figure 16 - Section D - Proposed Conditions - Static Loading

Figure 17 - Section D - Proposed Conditions - Seismic Loading

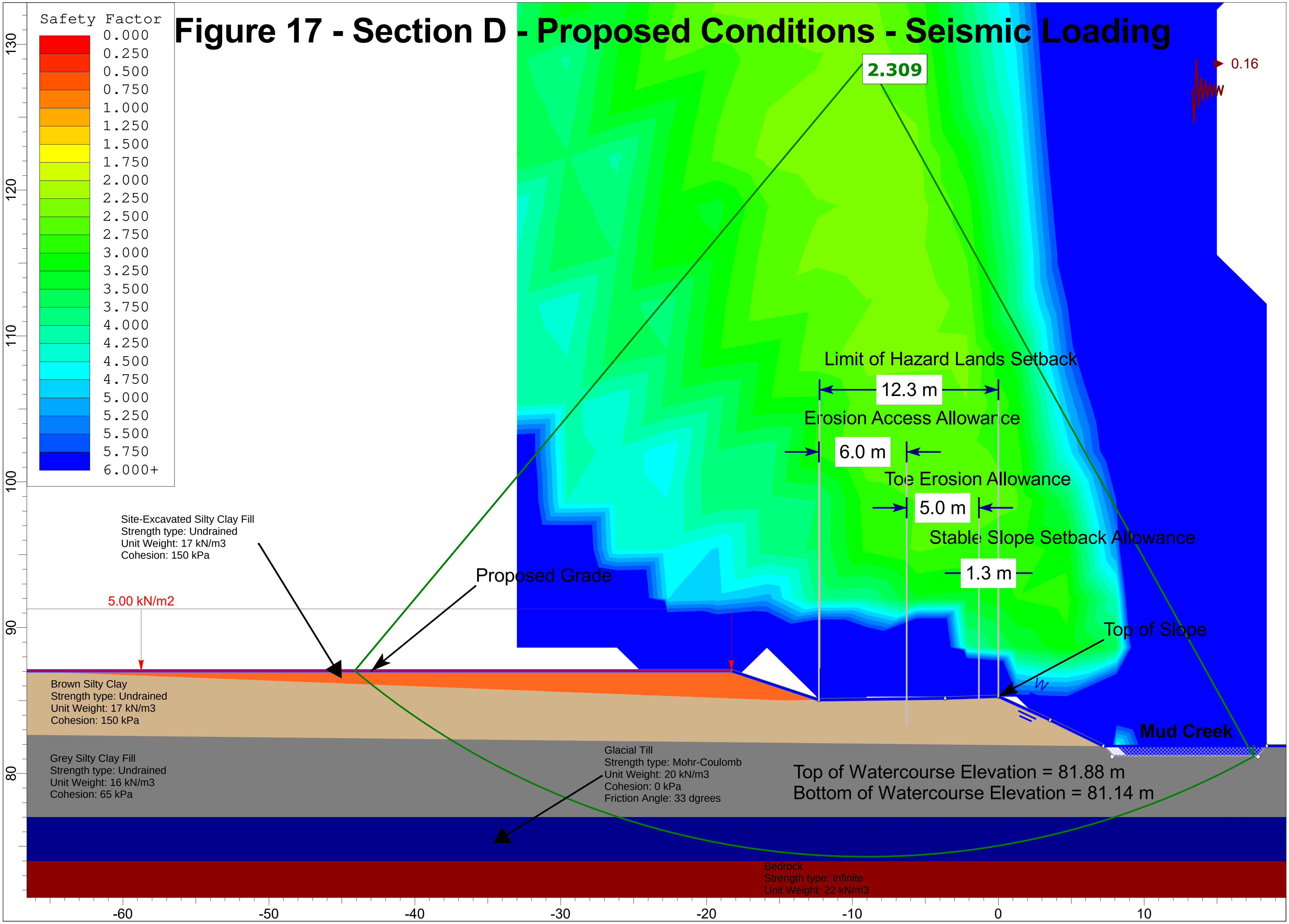


Photo 1: Area located at the bottom of the slope along the south-west portion of the subject site. Area is well vegetated and sloped gradually towards the valley floor.

Photo 2: Area along Wilson Cowan Drain and south-west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Water throughout Wilson Cowan Drain appeared to be flowing very slowly and/or ponding.

Photo 3: Area along Wilson Cowan Drain and south-west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Gradual slope observed from subject site to the valley floor.

Photo 4: Area along Wilson Cowan Drain and south-west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Water throughout Wilson Cowan Drain appeared to be flowing very slowly and/or ponding.

Photo 5: Area along Wilson Cowan Drain and south-west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Water throughout Wilson Cowan Drain appeared to be flowing very slowly and/or ponding.

Photo 6: Area along Wilson Cowan Drain and west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Gradual slope observed from subject site to the valley floor.

Photo 7: Area along Wilson Cowan Drain and west portion of the subject site. Area appeared to be well vegetated with a slightly steeper bank along Wilson Cowan Drain at the time of site visit. Gradual slope observed from subject site to the valley floor. Active erosion was not observed.

Photo 8: Area along Wilson Cowan Drain and west portion of the subject site. Area appeared to be well vegetated with a slightly steeper bank along Wilson Cowan Drain at the time of site visit. Gradual slope observed from subject site to the valley floor. Active erosion was not observed.

Photo 9: Area along Wilson Cowan Drain and north-west portion of the subject site. Area appeared to be well vegetated with a gentle flow throughout Wilson Cowan Drain at the time of site visit. Gradual slope observed from subject site to the valley floor.

Photo 10: Area of intersection of Wilson Cowan Drain along west portion of subject site and Mud Creek. Area of Mud Creek appeared to have banks exposed to streams flow. Mature trees noted to have previously fallen across creek alignment. Some over-steepening of banks also observed at the time of site visit.

Photo 11: Area of Mud Creek along north-west portion of subject site. Area appeared to have banks exposed to streams flow and lack of well rooted vegetation along bank. Some over-steepening of banks also observed. Creek appeared to be flowing very slowly at the time of site visit.

Photo 12: Area of Mud Creek along north-west portion of subject site. Area appeared to have banks exposed to streams flow and along with slumping and oversteepening of banks at the time of our site visit.

Photo 13: Area of Mud Creek along north-west portion of subject site. Area of valley floor appeared to have well rooted vegetation with relatively steep banks along creek. No active erosion observed along photographed portion of creek at the time of site visit.

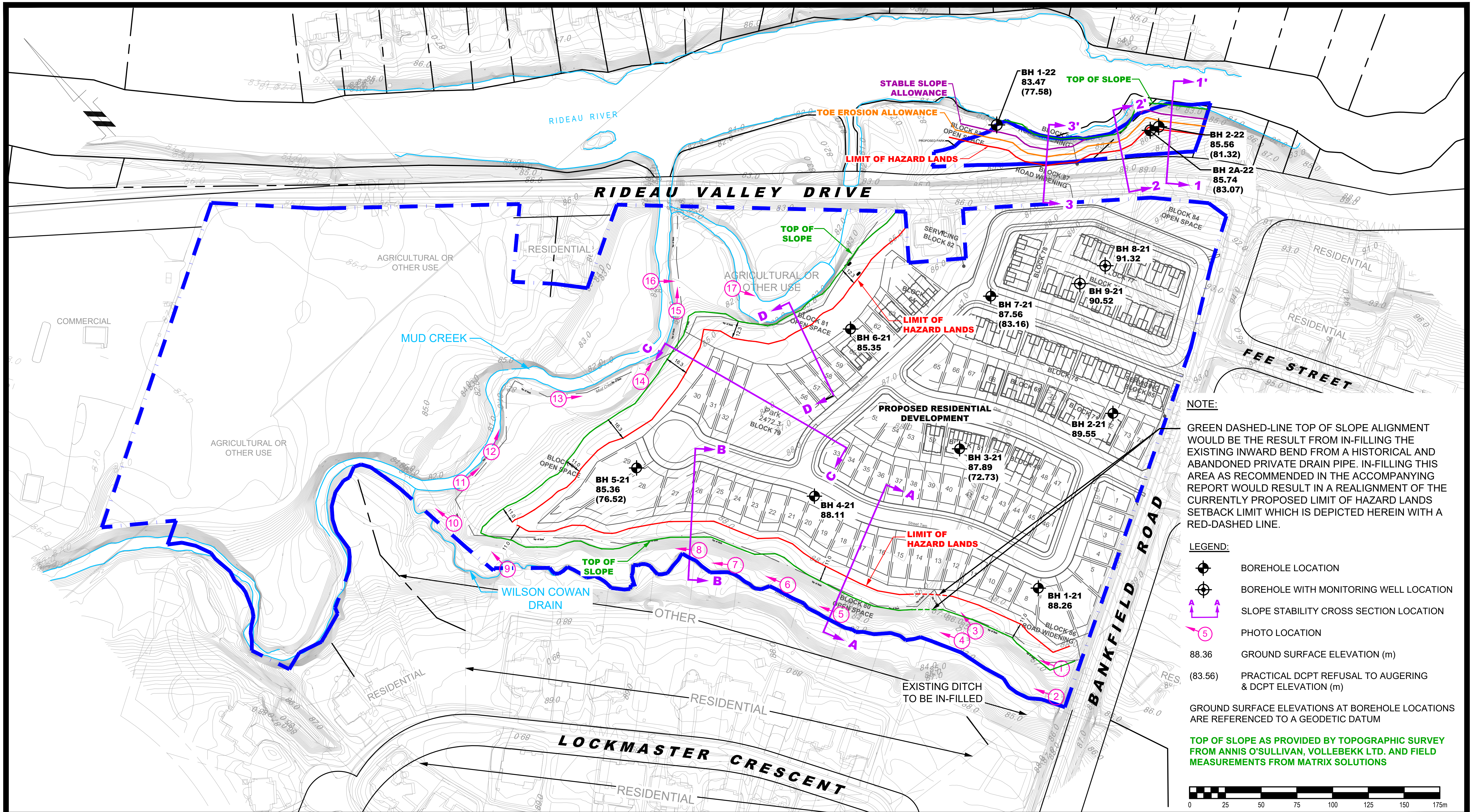
Photo 14: Area of Mud Creek along north-west portion of subject site. Area of valley floor appeared to have well rooted vegetation with relatively steep banks along creek. Some active erosion and fallen trees observed along photographed portion of creek.

Photo 15: Area of Mud Creek along northern portion of subject site. Photographed area appeared to have banks exposed to streams flow along with slumping and undercutting of banks at the time of our site visit.

Photo 16: Area of Mud Creek valley floor along north-east portion of subject site. Photographed area appeared to contain well rooted vegetation and mature trees.

Photo 17: Area of Mud Creek valley floor along north-east portion of subject site. Photographed area appeared to contain well rooted vegetation and mature trees. No erosion observed along toe of slope at time of site visit.

Photo 17b: Close-up of Photo 17 - Area of Mud Creek valley floor along north-east portion of subject site. Photographed area appeared to contain well rooted vegetation and mature trees. No erosion observed along toe of slope at time of site visit.



NOTE:

GREEN DASHED-LINE TOP OF SLOPE ALIGNMENT WOULD BE THE RESULT FROM IN-FILLING THE EXISTING INWARD BEND FROM A HISTORICAL AND ABANDONED PRIVATE DRAIN PIPE. IN-FILLING THIS AREA AS RECOMMENDED IN THE ACCOMPANYING REPORT WOULD RESULT IN A REALIGNMENT OF THE CURRENTLY PROPOSED LIMIT OF HAZARD LANDS SETBACK LIMIT WHICH IS DEPICTED HEREIN WITH A RED-DASHED LINE.

LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
A A	SLOPE STABILITY CROSS SECTION LOCATION
⑤	PHOTO LOCATION
88.36	GROUND SURFACE ELEVATION (m)
(83.56)	PRACTICAL DCPT REFUSAL TO AUGERING & DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

TOP OF SLOPE AS PROVIDED BY TOPOGRAPHIC SURVEY FROM ANNIS O'SULLIVAN, VOLLEBEKK LTD. AND FIELD MEASUREMENTS FROM MATRIX SOLUTIONS

PATERSON GROUP

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

4	SITE PLAN UPDATED	04/04/2024	SK
3	WATER COURSES NAME UPDATED	20/10/2023	MS
2	CONCEPTUAL PLAN UPDATED	14/10/2022	MS
1	TOP OF SLOPE REVISED AS PER SITE SPECIFIC TOPOGRAPHIC SURVEY FROM NOVATECH	08/23/2022	MS
NO.	REVISIONS	DATE	INITIAL

UNIFORM DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4386 RIDEAU VALLEY DRIVE

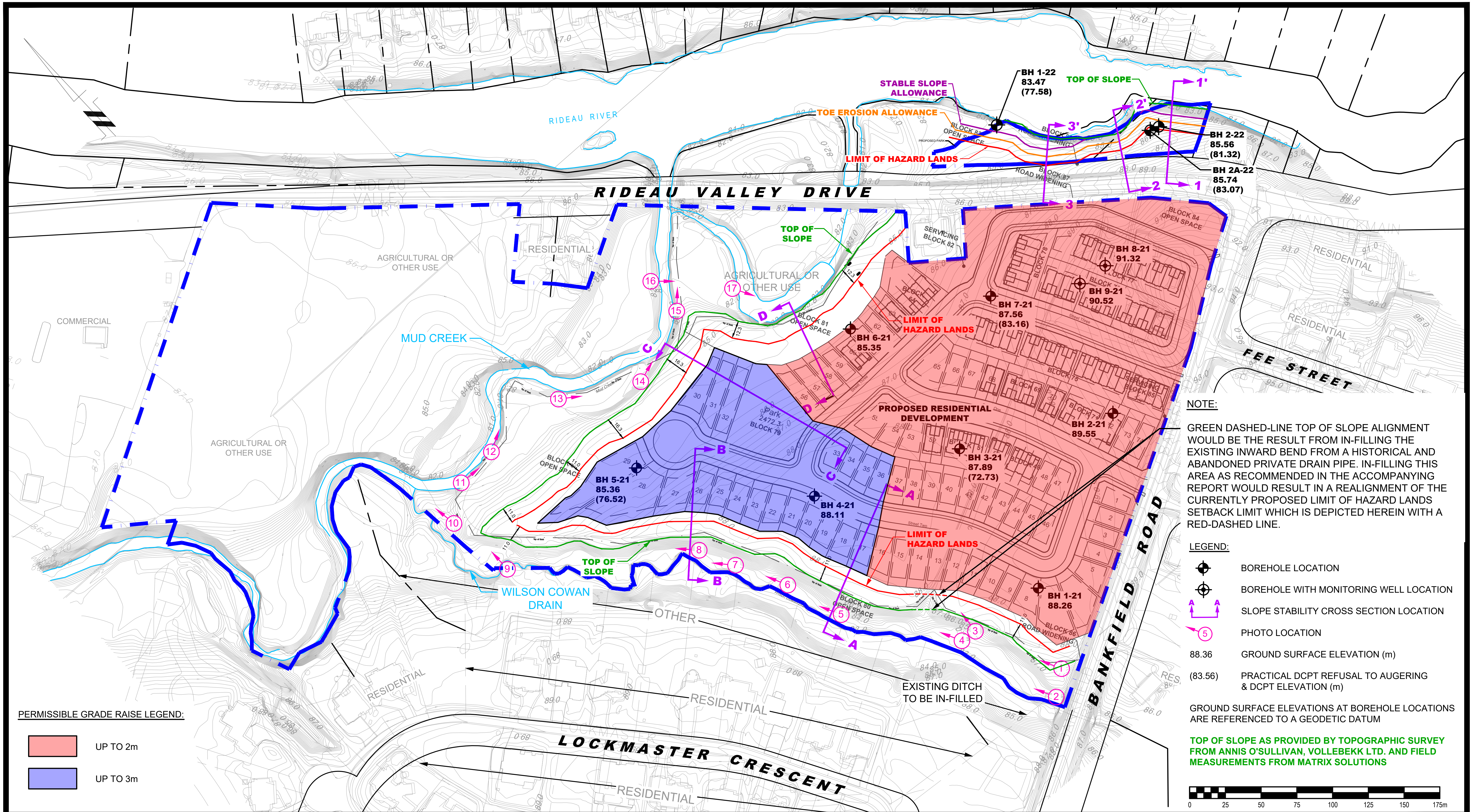
OTTAWA, **ONTARIO**

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2500
Date: 06/2021
Drawn by: JM
Report No.: PG5828-1
Checked by: KP
Dwg No.: PG5828-1
Approved by: DJG
Revision No.: 4

p:\autocad drawings\geotechnical\pg58xx\pg5828\pg5828-1 thlp (rev.04) (july 2024).dwg



NOTE:

GREEN DASHED-LINE TOP OF SLOPE ALIGNMENT WOULD BE THE RESULT FROM IN-FILLING THE EXISTING INWARD BEND FROM A HISTORICAL AND ABANDONED PRIVATE DRAIN PIPE. IN-FILLING THIS AREA AS RECOMMENDED IN THE ACCOMPANYING REPORT WOULD RESULT IN A REALIGNMENT OF THE CURRENTLY PROPOSED LIMIT OF HAZARD LANDS SETBACK LIMIT WHICH IS DEPICTED HEREIN WITH A RED-DASHED LINE.

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
A A	SLOPE STABILITY CROSS SECTION LOCATION
⑤	PHOTO LOCATION
88.36	GROUND SURFACE ELEVATION (m)
(83.56)	PRACTICAL DCPT REFUSAL TO AUGERING & DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

TOP OF SLOPE AS PROVIDED BY TOPOGRAPHIC SURVEY FROM ANNIS O'SULLIVAN, VOLLEBEKK LTD. AND FIELD MEASUREMENTS FROM MATRIX SOLUTIONS

0 25 50 75 100 125 150 175m

PERMISSIBLE GRADE RAISE LEGEND:

🟥	UP TO 2m
🟦	UP TO 3m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

UNIFORM DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4386 RIDEAU VALLEY DRIVE

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:2500	Date: 06/2021
Drawn by: NFRV	Report No.: PG5828-1
Checked by: KP	Dwg No.: PG5828-3
Approved by: DJG	Revision No.:

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APPENDIX 3

RELEVANT REPORTS

PATERSON GROUP

memorandum

re: **Geotechnical Response to City Comments**
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

to: Uniform Urban Developments Ltd. – **Mr. Ryan MacDougall –** rmacgougall@uniformdevelopments.com

date: October 17, 2023

file: PG5828-MEMO.01

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide responses to the geotechnical-related comments from the City of Ottawa listed in the letter dated May 1, 2023 (File Nos. D02-02-220118, D07-16-22-0026) regarding the proposed residential development at the aforementioned site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5828-1 Revision 3 dated October 17, 2023.

Geotechnical Investigation Comments

Comment 2.11

Please refer to the watercourses as Mud Creek and the Wilson Cowan Drain, rather than Mud Ruisseau Creek and tributary, to remain consistent with other reports and plans submitted.

Response:

Noted. Reference to the watercourses has been modified in our revised geotechnical report mentioned above, as requested.

Comment 2.12

Please expressly state whether any of the clay soils on site may be ‘sensitive marine clays’, or not. [page 8 of 65].

Response:

As noted under subsection 6.9-Landscaping Considerations in our original geotechnical report, and based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples.

In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture levels and consistency, the silty clay across the subject site is considered low to medium sensitivity clay.

Having said that, it should be noted that page 8 has been revised to indicate the presence of low to medium sensitivity marine silty clay deposit in the subject site under subsection 5.1 in the above-mentioned revised geotechnical report, as requested.

Comment 2.15

Do the results of your study of the Slope Stability study align with the results from the Geo-fluvial Study? [page 18 of 65].

Response:

Paterson reviewed the geo-fluvial study completed by Matrix Solutions, dated November 2022, for the proposed residential development. Based on our review of the above-noted study, it appears that the results of our slope stability study are in general agreement with the results of the geofluvial study for the majority of the proposed limit of hazard lands with the exception of the recommended toe erosion allowance along Wilson Cowan Drain. Paterson is recommending 1m for toe erosion along that drain based on the nature and size of the drain (i.e. not a permanent watercourse) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season, as opposed to 5m for toe erosion as suggested by the geofluvial study. Furthermore, the geofluvial study did not provide photographs depicting active erosion along the Wilson Cowan drain. Further justification for the toe erosion allowance has been included in our geotechnical report under subsection 6.8. Having said that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan and the erosion limit proposed by Matrix solutions as well as the limit of hazard lands proposed by Paterson are both outside the limits of the proposed development.

Comment 2.16

Please provide further detail regarding the area proposed to be filled in the rear of Lots 5 & 6.

Response:

Backfilling of the slope face in the vicinity of the rear yards of lots 5 and 6 can be completed in a stepped fashion to provide a finish grade with a slope face of minimum 3H:1V. Site preparation and backfilling should be completed under dry weather conditions (specifically for the clay placement portion of the program) and above freezing temperatures, and in accordance with our geotechnical recommendations provided under section 5.2 of the revised geotechnical report noted above.

Comment 2.17

Please explain what the shrinkage limit and other Atterberg limits results infer.

Response:

Due to the presence of a silty clay deposit at the subject site, Paterson completed a review of the soils on the site to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Based on our review of the results of the shrinkage limit and Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples indicating that the silty clay across the subject site is considered ***low to medium sensitivity marine clay***, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Reference should be made to subsection 6.9- Landscaping Considerations in our above-mentioned revised geotechnical report.

Comment 2.18

Please state why the June 16, 2022, results were not included.

Response:

The geotechnical investigation conducted on June 16, 2022 pertained to the proposed park, located across Rideau Valley Drive which was done after submitting the geotechnical report for the residential development. Having said that, the results of the geotechnical investigation conducted for the proposed park have been added to the above-mentioned revised geotechnical report. Furthermore, the geotechnical letter mentioned above has been added as an addendum to Appendix 3 of the above-mentioned geotechnical report.

Comment 2.19

Consolidation results not found in the report.

Response:

No consolidation tests were completed on the encountered silty clay deposit at the subject site. Consolidation testing is not possible within the silty clay deposit, where encountered within the subject site, due to the stiffness of the overall deposit. Consolidation testing in the Ottawa area is typically carried out on soft to firm silty clay samples which are recovered from Shelby tubes taken during the field investigation. To accurately complete consolidation testing, the soft to firm (undrained shear strength of 12 to 50 kPa) silty clay samples are required to be undisturbed. The consistency of the silty clay encountered at the subject site was determined to be generally hard to stiff (undrained shear strength ranging between 50 to >200 kPa), based on in-situ vane testing completed as part of our geotechnical

investigation. Due to the consistency, advancement of Shelby tubes and subsequent recovery of an undisturbed silty clay sample is not possible.
Damage to either the piston sampler or the thin-walled Shelby tube is expected based on our experience with silty clay of similar consistency. Therefore, in our professional opinion, the available information collected from the boreholes drilled at the subject site is sufficient for us to provide a permissible grade raise for the proposed subdivision, without the need for a consolidation test. Reference should be made to subsection 5.3-Foundation Design, in our revised geotechnical report.

Comment 2.20

Sensitivity results are required.

Response:

The sensitivity index of the encountered silty clay deposit was calculated based on the ratio between the undisturbed and remolded shear vane test measured in the field, for all the boreholes, and it was found to be generally below 4, indicating a normal sensitivity clay. Please refer to subsection 4.2 in the revised above-mentioned geotechnical report fur further discussion regarding the sensitivity index calculation for the encountered silty clay deposit.

Comment 2.21

Atterberg limits results are required from a number of elevations in each borehole.

Response:

Atterberg limits tests were conducted at the encountered silt clay deposit in each borehole at the subject site. The soil samples were recovered from elevations below the anticipated design underside of footing elevation and 3.5 m depth below anticipated finished grade, and are considered to be sufficient from a geotechnical perspective to provide valuable information and satisfy the requirements for the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) in assessing the sensitivity of the silty clay deposit for tree planting.

Comment 2.22

A longer-term, or year-long groundwater level analysis is required.

Response:

Based on our understanding, LID measures are not considered for the subject site. Therefore, year-long groundwater level is not required from a geotechnical perspective at the subject site.

Comment 2.23

Groundwater cannot be stated to be expected to lower based on the LID directive documents without analysis showing that it will be so (with similitude, if necessary/appropriate).

Response:

Reference should be made to our response to comment 2.22 above. Furthermore, it is unclear what the reviewer is referring to LID directives. Further clarification is required. In any case, post-development groundwater level lowering is conservatively anticipated following construction of site servicing at residential developments, as observed by Paterson from previous similar jobs.

Comment 2.24

For section 5.1, please note that lightweight fill is not permitted in ROWs.

Response:

Noted. Lightweight fill is not permitted in ROWs. Please refer to subsections 5.1 and 5.3 in the revised above-mentioned geotechnical report.

Comment 2.25

It is suggested that the plastic, sensitive soils be restricted in section 5.2 under the heading Fill Placement.

Response:

Our recommendation for fill placement under subsection 5.2 clearly state that fill placed beneath the building areas should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. It is further stated in our report under section 5.2 that placement of a non-specified existing fill along with site-excavated soil (including the plastic sensitive soils) is permitted only under landscape areas where settlement of the ground surface is of minor concern.

Comment 2.26

Section 5.3, under the heading Bearing Resistance Values (Conventional Shallow Foundation), should be reviewed against the grading plan and the boreholes.

Response:

Noted. A statement was added to the report to indicate that the bearing capacity will be reviewed against the grading plans for the proposed residential subdivision, once available. Reference should be made subsection 5.3 in the above-mentioned revised geotechnical report.

Comment 2.27

The comments that the subject site are not susceptible to liquefaction requires an exhaustive discussion: whichever approach the consultant takes will require proof of similitude and full copies of papers provided to the City showing unequivocal support.

Response:

The soils encountered at the subject site consist of silty clays, which are cohesive in nature. These soils were evaluated for liquefaction susceptibility in accordance with the criteria prepared by Bray at al. 2004 which determines that all soils with a plasticity index exceeding 20% are not liquifiable (Figure 1). In general, the plasticity index results completed on samples taken from the silty clay layer were found to be above 20. Therefore, the encountered soils are not susceptible to liquefaction. Reference should be made to subsection 5.4- Design for Earthquakes in the abovementioned revised geotechnical report, for further details on liquefaction susceptibility at the subject site.

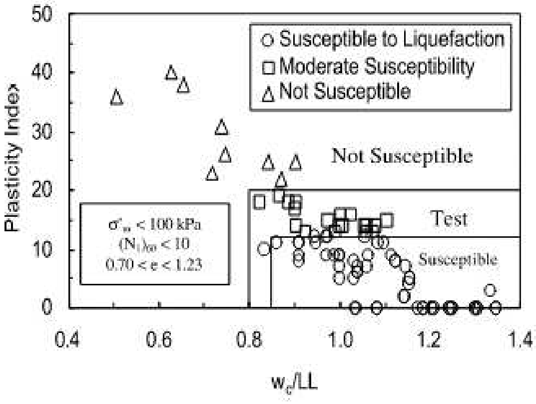


Figure 1. Criteria for evaluating liquefaction susceptibility of fine-grained soils (Bray et al. 2004).

Comment 2.28

The comments under the heading of Foundation Drainage, within section 6.1, Foundation Drainage and Backfill, appear to be from another report; please review the report and confirm that all other comments are for the address intended.

Response:

Recommendations for foundation drainage for the proposed residential development are provided under section 6.1-Foundation Drainage and Backfill, of the above revised geotechnical report. These recommendations are applicable to the proposed residential development at the subject site.

Comment 2.29

For the end of section 6.3 please state if deep excavations will be occurring.

Response:

Based on the available conceptual plans, it is understood that the proposed subdivision will consist of single and townhouse style residential houses. Therefore, deep excavation for buildings is generally not anticipated at the subject site. Furthermore, the detailed design servicing plans were not provided at the time of writing the report. However, recommendations for deep excavations for construction of services, if deemed needed, are included in subsection 6.3- Excavation Side Slopes in the revised geotechnical report for the subdivision, referenced above.

Comment 2.30

Please state why the horizontal acceleration of 0.16g was included under the heading of Seismic Loading Analysis (as opposed to another value).

Response:

Per the City of Ottawa Slope Stability Guidelines for Development Applications, the seismic coefficient to be used in the analyses is typically half the peak ground acceleration (PGA) specified in the National Building Code of Canada (NBCC). The PGA at the location of the subject site, based on the 2015 NBCC is approximately 0.266. Therefore the seismic coefficient at the location of the subject site is 0.133. However, based on previous versions of the NBCC, the PGA for the Ottawa area is 0.32, thus using a seismic coefficient of 0.16 is generally a more conservative approach, and is considered acceptable from a geotechnical perspective.

Comment 2.31

A toe erosion allowance of 1 m is not acceptable. The comments on “active erosion was not observed” are contested in a number of the photographs in Appendix 2. The toe erosion allowance, under the heading of Toe Erosion and Access Allowances shall be revised as per Table 3 of the Ministry of Natural Resources, and Forestry (MNRF) Technical Guide- River and Stream Systems: Erosion Hazard Limit due to the active erosion and the soils of the boreholes. It is noted that the Fluvial Geomorphic and Erosion Hazard Assessment completed by Matrix Solutions Inc. recommended a 5 m toe erosion allowance for the Wilson Cowan Drain. Based on the penetration resistance blows of the Soil Profile and Test Data Sheets the soils on site may be Soft/Firm Cohesive Soils, loose granular, (sand, silt) fill, in the MNRF Guide.

Response:

Based on our field review and engineering analysis, active erosion was not encountered along the western watercourse at Wilson Cowan drain. It is to be clarified that the photographs depicting active erosion in Appendix 2 of the geotechnical report are for the Mud Creek watercourse, as indicated in the description, not for Wilson Cowand Drain, where no active erosion was recorded. In addition, Paterson recommended a 1m toe erosion allowance along the Wilson Cowan Drain based on the nature and size of the drain (i.e. not a permanent watercourse, anthropogenic not natural) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season. Therefore, based on our review, the recommended toe erosion allowance from the watercourse edge of 5 m for Mud Creek (main channel) and 1 m for Wilson Cowan Drain (western tributary), respectively is considered acceptable from a geotechnical perspective. Further justification for the toe erosion allowance has been included in our geotechnical report under section 6.8. In addition, Paterson revised the limit of hazard lands to show both the geotechnical limit of hazard lands setback based on our slope stability analysis, as well as the erosion hazard limit based on the Matrix Solutions geofluvial study, which considered a 5m toe erosion for Wilson Cowan Drain. Having said that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan.

Comment 2.32

The sensitivity results in section 6.9 should be derived from vane shear results.

Response:

For tree planting setbacks, the sensitivity of the clay was based on the Atterberg limit test results, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Sensitivity index which is calculated from the vane shear results is not used to determine tree planting setbacks, as per the City of Ottawa Guidelines for Tree Planting in Sensitive Marine Clays.

Comment 2.33
Please state if above ground swimming pools were contemplated in the section headed Swimming Pools in section 6.9.

Response:
Above ground swimming pools are contemplated *under section 6.9* in our geotechnical investigation report.

Comment 2.35
Section 7 should also include review of trees in proximity to foundations.

Response:
Noted. A statement has been added under section 7 indicating the requirement for completing a landscaping plan review by the geotechnical consultant. Please refer to the revised above-mentioned geotechnical report.

Comment 2.36
In Appendix 1 please add a determination, in the Symbols and Terms, of an n value of P.

Response:
The Symbols and Terms of ‘p’ reference in Appendix 1 is used to describe the “push spoon”, which we conducted to collect soil samples for testing. The definition of p has been added to the symbols list in Appendix 1.

Comment 2.37
It is suggested that a number of borehole logs should be modified due to the presence of a blow count record of P, yet the description is listed as “hard to very stiff”, for example, BH 1-21.

Response:
As explained in our response for comment 2.36, P (or push spoon) is not an SPT test. A push spoon sample is completed to collect a soil sample for visual observation and further testing. Therefore, it does not measure the consistency of the soil and it should not be correlated with N values.

Comment 2.38
Please discuss how the shear strength of BH 1-21 is 119 kPa at 4 m depth (with an N count of 5, while, at 5 m depth the shear strength is 139 with a blow count of P).

Response:
Please refer to our response to comment 2.37 and 2.38 above. It is erroneous to correlate P with the N value obtained from the SPT for clayey soils.

Comment 2.39
Please include DCPT results from 6.55 to 11 for borehole BH 3-21

Response:
The DCPT was pushed from 6.55 to 11 at the location of BH 3-21 with no recorded penetration resistance, which is typical for the grey silty clay deposit in Ottawa.

Comment 2.40
Please provide documentation confirming bedrock elevation.

Response:
Based on available geological mapping, the bedrock in the subject area consists of Dolomite of the Oxford formation, with an overburden drift thickness of 10 to 25 m depth. Bedrock was not encountered within the maximum investigated depth of 6.4m. The proposed residential development is anticipated to consist of single and townhouse style residential homes, of slab-on-grade construction, and founded on shallow footings. Therefore, there is no requirement to determine the elevation of bedrock for the proposed residential development at the subject site, from a geotechnical perspective.

Comment 2.41
Please add DCPT results from 6.1 to 8.4 m to BH 5-21.

Response:
Refer to our response to comment 2.39 above.

Comment 2.42
Please include laboratory results for the sections shown on Appendix 2.

Response:
It is to be noted that the subsoil conditions at the analyzed cross-sections were inferred based on nearby boreholes, completed within the subject site, as well as on the results of the insitu vane shear tests, as discussed under section 6.8 of the above-mentioned revised geotechnical report.

Comment 2.43
The soil annotations on Figure 3 appear to be floating.

Response:
Noted. The annotations for soil layers in Figure 3 have been modified in the above-mentioned revised report.

Comment 2.44
Please include bathymetric survey data used for Figure 4 (amongst others).

Response:
The bottom elevations of the watercourses at the studied cross sections has been determined using a high precision GPS, during our site visit to review the slope conditions. These elevations have been added to the slope cross sections included in the revised geotechnical report referenced above.

Comment 2.45
The annotation in the red area is not legible.
Response:
Noted. The annotation in the red area has been enhanced to be legible. Please refer to the revised geotechnical report mentioned above.

Comment 2.46

Some non-circular slip circles should be analyzed (considering the soil types).

Response:

The analysis of the stability of the slopes was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. According to standard practice for slope stability analysis, a simple circular failure surface method is applicable for a slope in a homogenous soil layer. On the other hand, a non-circular failure surface would be investigated in case of a heterogeneous multi-soil layered slope. Based on the encountered subsurface conditions along the north and west slopes at the subject site, it is not required to complete a non-circular slip circle analysis for the subject slopes, from a geotechnical perspective.

Comment 2.47

It is suggested that additional cross-sections are required along north and west sides of the subdivision lands.

Response:

Based on our review of the existing slope conditions, five (5) slope cross-sections were studied as the worst-case scenarios and are considered sufficient, based on the observed side slopes and on the existing conditions. From a geotechnical perspective, additional cross-sections are not required along north and west sides of the subdivision lands. However, additional analysis considering proposed loading conditions, including the porposed grade raises, buildings & roads has been added to the revised geotechnical report.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
October 17, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Maha Saleh, M.A.Sc., P.Eng.

David J. Gilbert, P.Eng.

PATERSON GROUP

memorandum

re: **Geotechnical Response to RVCA Comments**
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

to: Uniform Urban Developments Ltd. – **Mr. Ryan MacDougall –** rmacgougall@uniformdevelopments.com

date: October 17, 2023

file: PG5828-MEMO.02

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide responses to the geotechnical-related comments from the RVCA listed in the letters dated April 27, 2023 and May 1, 2023 (File: 23-NEP-SUB-0041) regarding the proposed residential development at the aforementioned site as well as the porposed Park block to be located east of Rideau Valley Drive, along Rideau River. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5828-1 Revision 3 dated October 17, 2023 and PG5828-LET.01 Revision 2 dated October 17, 2023.

It should be noted that Paterson completed the previous and current slope stability analyses for the slopes along Mud Creek, Wilson Cowan Drain, and Rideau River at the subject sites based on current practice for slope stability analysis in Ottawa, and in accordance with the City of Ottawa Slope Stability Guidelines for Development Applications. The adopted methodology as well as the selection of soil parameters for the encountered soil properties have been done taking into account our vast experience in the area and in similar applications.

Discussion Topic 1: Geotechnical Investigation Report for the Proposed Residential Development, 4386 Rideau Valley Drive, Ottawa, Ontario; prepared by: Paterson Group; report no: PG5828-1; Rev no: 2; dated 14-Oct-2022.

Comment 1

In section 6.9 – General landscaping comments should include additional best practices recommendations, such as but not limited to:

- *i.) It is important to avoid directing uncontrolled water towards the slope (drainage, gutter, septic field, pool & hot tub drainage, etc.)*
- *ii.) It is important to avoid overloading the top of the slope (backfill, fill, miscellaneous waste, grass cuttings, branches, leaves, snow, etc.)*
- *iii.) It is important to avoid excavating at the base of the slope.*
- *iv.) It is important to maintain a healthy native vegetation cover.*
- *v.) Any future additions, such as aboveground swimming pools or accessory buildings, should entail reassessment of slope stability unless this has been pre-confirmed via supplementary slope stability analyses during the design stage.*

Response:

Noted. Additional considerations regarding the above items have been added to Subsection 6.9- Landscaping Considerations in the above mentioned revised geotechnical report.

Comment 2

Section 6.8 – Slope Conditions Along the Western Boundary: It is recommended to provide Paterson Group with the Matrix Solution report, since the field inspection was conducted before the fluvial geomorphological study. This will ensure that Paterson has all the relevant information and can make informed decisions and recommendations in their report.

Response:

The slope stability analysis completed by Paterson for the porposed development takes into account our field observations of the existing slope conditions along Mud Creek and Wilson Cowan Drain, made during our site visit on May 19, 2021. Having said that, Paterson reviewed the geo-fluvial study completed by Matrix Solutions, dated November 2022, for the proposed development. Based on our review of the above-noted study, it appears that the results of our slope stability study are in general agreement with the results of the geofluvial study for the majority of the proposed limit of hazard lands. The main deviation from the above-noted geofluvial study is the recommended toe erosion allowance along Wilson Cowan Drain. Paterson recommended a 1m toe erosion allowance along that drain based on the nature and size of the drain (i.e. not a permanent watercourse, anthropogenic not natural) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season, as opposed to the 5m toe erosion allowance suggested by the geofluvial study. It is to be noted that the geofluvial study did not provide photographs depicting active erosion along the Wilson Cowan Drain nor did Paterson note any active erosions during our previous site visit. Further justification for the toe erosion allowance has been included in our geotechnical report under section 6.8. In addition, Paterson revised the limit of hazard lands to show both the geotechnical limit of hazard lands setback based on our slope stability analysis, as well as the erosion hazard limit based on the Matrix Solutions geofluvial study. Having aid that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan.

Comment 3

Section 6.8 – Slope Stability analysis: Soil strength parameters (c and Φ) for drained (effective stress conditions) and undrained (total stress conditions), as well as information for the rational on how they were established should be provided within the body of the report (how are they inferred from in situ and laboratory testing, any correlations used?). There is currently not sufficient information to accept that soil strength parameters used by the consultant reflect accurately the site conditions.

Response:

The soil strength parameters for drained and undrained conditions used in the slope stability analysis were chosen based on the subsurface conditions observed in the test holes located within the proximity of the slopes, and our general knowledge of the geology in the area. Furthermore, the adopted soil strength parameters are within the range of recommended values for different soil layers based on the City of Ottawa's slope stability guidelines and academic literature such as M.A. Klugman and P. Chung, 1976. Further discussion on the selection of the soil strength parameters has been added to Subsection 6.8- Slope Stability Assessment, in the above mentioned geotechnical report.

Comment 4

Section 6.8 – Slope Stability analysis: We noted that soil strength parameters for grey softer clays under the drained static analyses were higher than for the upper brown clays (desiccated crust), please explain rational, as in standard practice the contrary is observed.

Response:

Based on the City of Ottawa's slope stability guidelines and academic literature such as M.A. Klugman and P. Chung, 1976, brown clay has lower cohesion values compared to grey clay. Due to the loss of water in Brown silty clay and weathering of the silty clay particle, the cohesion values are decreased in comparison with the grey clay. However, it should be noted that our calculations and assumptions in the slope stability models are in the range of recommended values for different soil layers based on the above noted guidelines.

Comment 5

We noted that only drained analyses were undertaken for the static conditions. It is generally geotechnical best practice to undertake both drained and undrained analyses when in presence of clayey soils, even if the drained conditions governed.

Response:

Paterson completed the slope stability assessment for the slopes along Mud Creek and Wilson Cowan Drain, within the subject site, in accordance with best practice for slope stability analysis in Ottawa as well as the City of Ottawa's slope stability guidelines. Based on the City guidelines for slope stability analysis, the potential for a drained failure should be checked for the case of slow loading (i.e. realistic condition of natural slope) whereas that of undrained failure should be checked for the case of sudden or short term loading (i.e. seismic loading). Completing an undrained analysis under static loading would always provide a higher safety factor compared to the same undrained analysis completed under seismic loading, because it would be the same analysis minus the seismic load.

The critical scenario in this case is the undrained analysis under seismic loading. Reference should be made to Subsection 6.8 -Slope Stability Assessment in the abovementioned geotechnical report for further details on the analysis methodology.

Comment 6

Please provide information within the body of the report to support that the clay is not sensitive.

Response:

The sensitivity index of the encountered silty clay deposit was calculated based on the ratio between the undisturbed and remolded shear vane test measured in the field, for all the boreholes, and it was found to be generally below 4, indicating a normal sensitivity clay. Please refer to Subsection 4.2-Subsurface Profile, in the abovementioned geotechnical report.

Comment 7

Additionally, the sections should display the water level used in the stability. Generally, it should consider the design low water level (present flow) as well as the 100-year flood level.

Response:

The water level used in the analysis is displayed on the cross sections in the previous and current geotechnical reports. The slope stability analysis was completed for the worst-case scenario at several cross sections, considering a conservative review of the groundwater conditions, where the silty clay deposit was considered to be fully saturated and the groundwater level was taken at ground surface, which is common practice for completing slope stability analysis for natural slopes in Ottawa. The 100- year flood level is typically completed for storm ponds in confined excavations and would generally yield a higher safety factor for slope stability as compared to the current water level in the watercourse due to the balancing of the hydrostatic pressure.

Comment 8

Section 5.3 – Permissible Grade Raise Restriction allow for up to 2 m of fill to be added. This scenario should be analysed where fill is proposed to ensure that this would not negatively affect the Factor of Safety (FoS). It would be important to consider potential water seepage/perched water table at the interface of the fill and impermeable existing clay layer that could result after the placement of the fill material (expected to be more permeable).

Response:

Paterson completed additional slope stability analyses for the proposed conditions considering an approximate average grade raise of 2m at the location of the studied cross sections areas. The new slope stability cross sections account for the proposed grade raise as well as the proposed buildings/roads within the development. Based on our slope stability analysis, a stable slope setback varying between 1.3 and 5.3 m from the top of the slope are required to achieve a factor of safety of 1.5 for the limit of the hazard lands along Mud Creek. The results of the new slope stability analysis have been added to the abovementioned geotechnical report. Reference should be made to Drawing PG5828-1 – Test Hole Location Plan for the proposed Limit of Hazard Lands setback for development considerations at the subject site.

Comment 9

Where applicable, on lots along the slopes, surcharge from proposed structures/roads should be incorporated within the analyses.

Response:

Refer to our response for Comment 8 above.

Comment 10

Section 6.8 – Limit of Hazard Lands: The consultant established a toe erosion allowance of 5 m along Mud Creek and 1m along Wilson-Cowan drain based on their review of erosion on site with a future 6 m erosion access allowance. This is supplemented with a stable slope allowance where needed. Please update with a toe allowance of 5 m along all watercourses as recommended in the Fluvial Geomorphic and Erosion Hazard Assessment prepared by Matrix Solution Inc.

Response:

Refer to our response for Comment 2 above.

Slope Stability Assessment; Proposed River Park, 4386 Rideau Valley Drive - Ottawa, Ontario; prepared by: Paterson Group Report PG5828-LET.01 Rev. 1 dated: July 5th, 2022.

Comment 11

The study may have to be revised such as to address the following: a. Section 2.0 – Slope Stability analysis: Please confirm if the Rideau Valley Road is present within the analysis sections. We would generally recommend that it be labelled, modelled as fill with proper traffic transient loading conditions.

Response 11

The slope stability analysis does not include the Rideau Valley Road since it is located far enough from the top of slope and will have negligible influence on the slope stability of the subject slope.

Comment 12

Soil strength parameters (c and Φ) for drained (effective stress conditions) and undrained (total stress conditions), as well as information for the rational on how they were established should be provided within the body of the report (how are they inferred from in situ and laboratory testing, any correlations used?). There is currently not sufficient information to accept that soil strength parameters used by the consultant reflect accurately the site conditions.

Response

Refer to our response for Comment 3 above.

Comment 13

We noted that soil strength parameters for grey softer clays under the drained static analyses were higher than for the upper brown clays (desiccated crust), please explain rational, as in standard practice the contrary is observed.

Response:

Refer to our response for Comment 4 above.

Comment 14

We noted that only drained analyses were undertaken for the static conditions. It is generally geotechnical best practice to undertake both drained and undrained analyses when in presence of clayey soils, even if the drained conditions governed.

Response:

Refer to our response for Comment 5 above.

Comment 15

Please provide information within the body of the report to support that the clay is not sensitive.

Response:

Refer to our response for Comment 6 above.

Comment 16

Additionally, the sections should display the water level used in the stability. Generally, it should consider the design low water level (present flow) as well as the 100-year flood level.

Response:

Reference should be made to our response for Comment 7 above.

Erosion Hazard General Comments

Comment *17*

As mentioned in the Geotechnical Investigation comments above, it is important to avoid directing water and discharging it in an uncontrolled manner towards the slopes.

Response:

Noted. Reference should be made to the revised letter report.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

Maha Saleh, M.A.Sc., P.Eng.

LICENSED PROFESSIONAL ENGINEER
October 17, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

PATERSON GROUP

memorandum

re: **Geotechnical Response to City Comments**
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

to: Uniform Urban Developments Ltd. – **Mr. Ryan MacDougall –**
rmacgougall@uniformdevelopments.com

date: July 4, 2024

file: PG5828-MEMO.03

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide responses to the geotechnical-related comments from the City of Ottawa listed in the letter dated June 14, 2024 (File Nos. D02-02-22-0118, D07-16-22-0026) regarding the proposed residential development at the aforementioned site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5828-1 Revision 5 dated July 18, 2024.

Geotechnical Investigation Comments

(City01): Comment 2.11 *Please refer to the watercourses as Mud Creek and the Wilson Cowan Drain, rather than Mud Ruisseau Creek and tributary, to remain consistent with other reports and plans submitted.*

Paterson's Previous Response: Noted. Reference to the watercourses has been modified in our revised geotechnical report mentioned above, as requested.

(City02): Outstanding: *There are still some references to 'Mud Ruisseau' in your report. (Pages 71 thru 75 of 114, "Photographs From Site Visit – May 19, 2021").*

Response:
Noted. Reference to the watercourses has been modified in our revised geotechnical report mentioned above.

(City01): Comment 2.15 *Do the results of your study of the Slope Stability study align with the results from the Geo-fluvial Study? [page 18 of 65].*

Paterson's Previous Response: Paterson reviewed the geo-fluvial study completed by Matrix Solutions, dated November 2022, for the proposed residential development. Based on our review of the above-noted study, it appears that the results of our slope stability study are in general agreement with the results of the geofluvial study for the majority of the proposed limit of hazard lands with the exception of the recommended toe erosion allowance along Wilson Cowan Drain. Paterson is recommending 1m for toe erosion along that drain based on the nature and size of the drain (i.e. not a permanent watercourse) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season, as opposed to

5m for toe erosion as suggested by the geofluvial study. Furthermore, the geofluvial study did not provide photographs depicting active erosion along the Wilson Cowan drain. Further justification for the toe erosion allowance has been included in our geotechnical report under subsection 6.8. Having said that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan and the erosion limit proposed by Matrix solutions as well as the limit of hazard lands proposed by Paterson are both outside the limits of the proposed development.

(City02): Outstanding: *The Slope and Hazard Land layouts do not agree with that provided in the City's 'Slope Stability Guidelines (Dec-2004)', Figures 12 and 13. See attached. In addition, as the Fluvial report recommends a 5-metre toe erosion, this is the value that the City feels is applicable. Further the fluvial geomorphology report should be taken as superior to the geotechnical report for fluvial issues.*

Response:
This comment has been acknowledged. The toe erosion along the Wilson Cowan Drain has been revised to 5.0m. Please refer to the above-mentioned revised report.

(City01): Comment 2.22 *A longer-term, or year-long groundwater level analysis is required.*

Paterson's Previous Response: Based on our understanding, LID measures are not considered for the subject site. Therefore, year-long groundwater level is not required from a geotechnical perspective at the subject site.

(City02): Outstanding: *An accurate seasonal high groundwater level is necessary for the general design of subdivisions. All as per the Sewer Design Guidelines (Section 8.3.13) and the City's Low Impact Development Technical Guidance Report (Section 2.3.3, sheet 14 of 68).*

Please note that 'Low Impact Development' within subdivisions is also required as per the MECP Bulletin: 'Interpretation Bulletin, Ontario Ministry of Environment and Climate Change Expectations Re: Stormwater Management, February 2015'.

> *"Low impact development stormwater management is relevant to all forms of development, including new development, redevelopment, infill, and retrofit development." (page 2 of 7)*
>
> *"Infiltration of stormwater is needed to maintain ground water sources of drinking water, and to maintain stream base flows. At the same time, ground water quality must be protected from contamination, requiring the appropriate selection of LID measures, which would be determined by the hydrogeology*

of an area." (page 3 of 7)

The City notes that the 'Conceptual Site Servicing & Stormwater Management Report' provided with this application already provides some general guidance on LID Design. See Section 4.4.3 (sheet 21 of 324). This information should be referenced here.

Response:
????

(City01): Comment 2.23 *Groundwater cannot be stated to be expected to lower based on the LID directive documents without analysis showing that it will be so (with similitude, if necessary/appropriate).*

Paterson's Previous Response**:** Reference should be made to our response to comment 2.22 above. Furthermore, it is unclear what the reviewer is referring to LID directives. Further clarification is required. In any case, post-development groundwater level lowering is conservatively anticipated following construction of site servicing at residential developments, as observed by Paterson from previous similar jobs.

(City02): Outstanding: *See City of Ottawa response to Comment 2.22 (above) and the LID Technical Guidance Report declines estimations of groundwater lowering with development.*

Response:
????

(City01): Comment 2.27 *The comments that the subject site are not susceptible to liquefaction requires an exhaustive discussion: whichever approach the consultant takes will require proof of similitude and full copies of papers provided to the City showing unequivocal support.*

Paterson's Previous Response: The soils encountered at the subject site consist of silty clays, which are cohesive in nature. These soils were evaluated for liquefaction susceptibility in accordance with the criteria prepared by Bray at al. 2004 which determines that all soils with a plasticity index exceeding 20% are not liquifiable (Figure 1). In general, the plasticity index results completed on samples taken from the silty clay layer were found to be above 20. Therefore, the encountered soils are not susceptible to liquefaction. Reference should be made to subsection 5.4- Design for Earthquakes in the abovementioned revised geotechnical report, for further details on liquefaction susceptibility at the subject site.

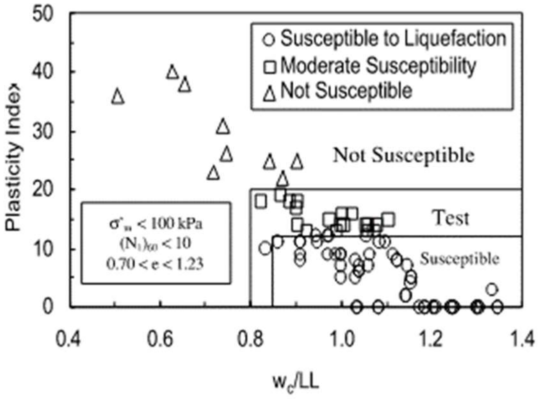


Figure 1. Criteria for evaluating liquefaction susceptibility of fine-grained soils (Bray et al. 2004).

(City02): Outstanding: *While the City understands the comparison implied here, we need to see testing or other data that confirms that this specific site meets these requirements.*

Response:
During our site investigation, Paterson conducted several field and laboratory tests to evaluate soil liquefaction potential. These included the Standard Penetration Test (SPT), which measures soil resistance to penetration using a hammer-driven sampler. Field vane testing was also completed within the silty clay deposits encountered in the test holes to assess soil strength under pore water pressure conditions. Shear strength values obtained from the field vane ranged between 50 and >200 kPa.
Additionally, Plasticity Index (PI) tests were conducted on selected soil samples to assess cohesive soil plasticity based on liquid and plastic limits. As previously indicated, the results showed a plasticity index above 20%. Based on these findings, the conducted field and laboratory testing provide sufficient evidence from a geotechnical perspective to confirm that the soils at the subject site are not susceptible to liquefaction.

(City01): Comment 2.31 *A toe erosion allowance of 1 m is not acceptable. The comments on “active erosion was not observed” are contested in a number of the photographs in Appendix 2. The toe erosion allowance, under the heading of Toe Erosion and Access Allowances shall be revised as per Table 3 of the Ministry of Natural Resources, and Forestry (MNRF) Technical Guide- River and Stream Systems: Erosion Hazard Limit due to the active erosion and the soils of the boreholes.*
It is noted that the Fluvial Geomorphic and Erosion Hazard Assessment completed by Matrix Solutions Inc. recommended a 5 m toe erosion allowance for the Wilson Cowan Drain. Based on the penetration resistance blows of the Soil Profile and Test Data Sheets the soils on site may be Soft/Firm Cohesive Soils, loose granular, (sand, silt) fill, in the MNRF Guide.

Paterson's Previous Response**:** Based on our field review and engineering analysis, active erosion was not encountered along the western watercourse at Wilson Cowan drain. It is to be clarified that the photographs depicting active erosion in Appendix 2 of the geotechnical report are for the Mud Creek watercourse, as indicated in the description, not for Wilson Cowand Drain, where no active erosion was recorded. In addition, Paterson recommended a 1m toe erosion allowance along the Wilson Cowan Drain based on the nature and size of the drain (i.e. not a permanent watercourse, anthropogenic not natural) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season. Therefore, based on our review, the recommended toe erosion allowance from the watercourse edge of 5 m for Mud Creek (main channel) and 1 m for Wilson Cowan Drain (western tributary), respectively is considered acceptable from a geotechnical perspective. Further justification for the toe erosion allowance has been included in our geotechnical report under section 6.8. In addition, Paterson revised the limit of hazard lands to show both the geotechnical limit of hazard lands setback based on our slope stability analysis, as well as the erosion hazard limit based on the Matrix Solutions geofluvial study, which considered a 5m toe erosion for Wilson Cowan Drain. Having said that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan.

(City02): Outstanding: *As discussed in comment 2.15 above, as the Fluvial report recommends a 5-metre toe erosion, this is the value that the City recognizes.*

Response:
This comment has been acknowledged. The toe erosion along the Wilson Cowan Drain has been revised to 5.0m. Please refer to the above-mentioned revised report.

(City01): Comment 2.37 *It is suggested that a number of borehole logs should be modified due to the presence of a blow count record of P, yet the description is listed as "hard to very stiff", for example, BH 1-21.*

Paterson's Previous Response: As explained in our response for comment 2.36, P (or push spoon) is not an SPT test. A push spoon sample is completed to collect a soil sample for visual observation and further testing. Therefore, it does not measure the consistency of the soil and it should not be correlated with N values.

(City02): Outstanding: *The N values provided on BH 1-21 at the 4m, 5m, and 6m depths states that the N value are 'P' (or push, or no resistance implying very soft soils). This seems to contradict the description of the soil as hard to very stiff soils. Please review and advise.*

Response:
As we previously explained, P (or push spoon) is not an SPT test and is completed just to collect a soil sample for visual observation and further testing only. It does not measure the consistency of the soil, and therefore, it should not be correlated with N values. *The description of the soil as hard to very stiff soils is obtained from our field observations and the completed* field vane testing within the silty clay deposits. Shear strength values obtained from the field vane at this borehole location and at 4m and 5m depth ranged between 139 kPa and 119 kPa, respectively. Please reference the symbols and terms in Appendix 1 in the above-mentioned report for the consistency guide or range based on the undrained shear strength values.

(City01): Comment 2.38 *Please discuss how the shear strength of BH 1-21 is 119 kPa at 4 m depth (with an N count of 5, while, at 5 m depth the shear strength is 139 with a blow count of P).*

Paterson's Previous Response: Please refer to our response to comment 2.37 and 2.38 above. It is erroneous to correlate P with the N value obtained from the SPT for clayey soils.

(City02): Outstanding: *The N values provided on BH 1-21 at the 4m, 5m, and 6m depths states that the N value are 'P' (or push, or no resistance implying very soft soils). This seems to contradict the description of the soil as hard to very stiff soils. Please review and advise.*

Response:
Please refer to our response to comments 2.37 above.

(City01): Comment 2.39 *Please include DCPT results from 6.55 to 11 for borehole BH 3-21*

Paterson's Previous Response: The DCPT was pushed from 6.55 to 11 at the location of BH 3-21 with no recorded penetration resistance, which is typical for the grey silty clay deposit in Ottawa.

(City02): Outstanding: *The DCPT results suggest soft soils. This seems to contradict the description of the soil as hard to very stiff soils. Please review and advise.*

Response:
As explained, at BH 3-21, the DCPT showed no recorded penetration resistance from depths of 6.55 to 11 meters, indicating stiff consistency of the soil at this borehole location, typical for grey silty clay deposits in Ottawa. However, hard to very stiff soils were measured at BH 1-21, BH 4-21, BH 5-21, BH 6-21, and BH 1-22 at depths ranging from 3 to 5m, characteristic of brown silty clay deposits.
Overall, our investigation revealed that the silty clay deposits generally consist of a hard to very stiff brown weathered crust extending from 1.5 to 5.2m below the ground surface, followed by stiff grey silty clay at BH 1-21, BH 3-21, BH 4-21, BH 5-21, BH 6-21, and BH 1-22. Therefore, there are contradicting in our description of the encountered soils.

(City01): Comment 2.40 *Please provide documentation confirming bedrock elevation.*

Paterson's Previous Response: Based on available geological mapping, the bedrock in the subject area consists of Dolomite of the Oxford formation, with an overburden drift thickness of 10 to 25 m depth. Bedrock was not encountered within the maximum investigated depth of 6.4m. The proposed residential development is anticipated to consist of single and townhouse style residential homes, of slab-on-grade construction, and founded on shallow footings. Therefore, there is no requirement to determine the elevation of bedrock for the proposed residential development at the subject site, from a geotechnical perspective.

(City02): Outstanding: *Please confirm that all the proposed homes will be constructed as slab on grade.*

Response:
This needs to be confirmed with the client

(City01): Comment 2.41 *Please add DCPT results from 6.1 to 8.4 m to BH 5-21*.

Paterson's Previous Response: Refer to our response to comment 2.39 above.

(City02): Outstanding: *The Dynamic Cone Penetration Tests (DCPT) results suggest soft soils. This seems to contradict the description of the soil as hard to very stiff soils. Please review and advise*

Response: Refer to our response to comment 2.39 above.

(City01): Comment 2.43 *The soil annotations on Figure 3 appear to be floating.*

Paterson's Previous Response: Noted. The annotations for soil layers in Figure 3 have been modified in the above-mentioned revised report.

(City02): Outstanding: As established in the 'Fluvial Geomorphic and Erosion Hazard Assessment' the toe erosion allowance should be 5 metres. Page 23 of 46, Section 4.3.2.

Response:
This comment has been acknowledged. The toe erosion along the Wilson Cowan Drain has been revised to 5.0m. Please refer to the above-mentioned revised report.

(City01): Comment 2.46 *Some non-circular slip circles should be analyzed (considering the soil types).*

Paterson's Previous Response: The analysis of the stability of the slopes was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. According to standard practice for slope stability analysis, a simple circular failure surface method is applicable for a slope in a homogenous soil layer. On the other hand, a non-circular failure surface would be investigated in case of a heterogeneous multi-soil layered slope. Based on the encountered subsurface conditions along the north and west slopes at the subject site, it is not required to complete a non-circular slip circle analysis for the subject slopes, from a geotechnical perspective.

(City02): Outstanding: *Referencing Figure 3, page 52 of 114, three soil types are indicated to be included in the slip circle. Also note that grey silty clay soils are a significantly weaker soil and not considered homogenous. The City will need to see a couple of non-circular failure surface calculations.*

Response:
This comment has been acknowledged. Multiple non-circular failure surfaces have been added to Figure 3. Please refer to the above-mentioned revised report.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

July 18, 2024

Zubaida Al-Moselly, P.Eng.

Faisal I. Abou-Seido, P.Eng.

LICENSED PROFESSIONAL ENGINEER
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

PATERSON GROUP

Consulting Engineers
9 Auriga Drive
Ottawa, Ontario
K2E 7T9
Tel: (613) 226-7381

Geotechnical Engineering
Environmental Engineering
Hydrogeology
Materials Testing
Building Science
Rural Development Design
Retaining Wall Design
Noise and Vibration Studies

patersongroup.ca

October 17, 2023
PG5828-LET.01 Rev. 2

Uniform Developments
300-117 Centrepoint Drive
Ottawa, Ontario
K2G 5Y6

Attention: **Mr. Ryan MacDougall**

Subject: **Slope Stability Assessment**
Proposed River Park
4386 Rideau Valley Drive - Ottawa, Ontario

Dear Sir,

Paterson Group (Paterson) was commissioned by Uniform Developments to conduct a slope review for the proposed river park to be located across 4386 Rideau Valley Drive in the City of Ottawa, Ontario.

1.0 Field Observation

The field program for the proposed river park was completed on June 16, 2022. At that time, a total of two boreholes were advanced down to a maximum depth of 5.9 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5828-2 – Limit of Hazard Lands Plan attached to this letter.

Surface Conditions

The subject site is currently vacant and covered with grass and trees. It is bound to the east by Rideau River, to the west by Rideau Valley Drive followed by a future development, to the south by a single-family dwelling, and to the north by a similar vacant lot. The ground surface across the subject site is generally flat and gently sloping upwards towards the south and west from an approximate geodetic elevation of 80 m at the north to 88 m at the south. The site is approximately 1.5 to 2.0m lower than Rideau Valley Drive. The southern portion of the site is generally covered with mature trees.

The slope conditions were reviewed by Paterson on May 17, 2022. The existing slopes were generally observed to be covered with well rooted vegetation across the surface. The western slopes were observed to be approximately 2 to 3 m high and appeared to have a relatively steep profile of less than 1H:1V. On the other hand, the eastern slopes were observed to be 4 to 5m high and appeared to have a slope profile ranging between 2H:1V to 3H:1V.

The width of the Rideau River was noted to be between 26 m wide to the south and 80 m wide to the north along the site length. The majority of the riverbed appeared to be covered by an in-situ stiff to stiff brown silty clay. The majority of the riverbanks were observed to be affected by active erosion and were exposed directly to stream flow. Additional signs of erosion consisted of exposed tree roots.

Subsurface Conditions

Generally, the subsurface soil profile at the test hole locations consists of topsoil underlain by a deposit of very stiff to stiff brown silty clay underlain by glacial till. The brown silty clay was observed to be underlain by a stiff grey silty clay at BH 1-22. Glacial till was encountered below the clay deposit at all boreholes. The glacial till deposit was generally observed to consist of compact to dense brown silty sand with gravel, cobbles and boulders. Practical refusal to augering was encountered at an approximate depth of 5.9m and 2.7m at the locations of BH 1-22 and 2A-22, respectively. Practical refusal to DCPT was encountered at an approximate depth of 4.24m at BH 2-22. 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.

Based on available geological mapping, the bedrock in the subject area consists of Dolomite of the Oxford formation, with an overburden drift thickness of 10 to 25 m depth.

2.0 Slope Stability Assessment

The existing slope conditions were reviewed by Paterson to define a conceptual limit of hazard lands setback, which is to be respected for any permanent structures, such as gazebos. It should be noted that stone dust paths with minor grading adjustments and park benches are acceptable to be placed within the limit of hazard lands line from a geotechnical perspective. The proposed limit of hazard lands designation line consists of the following:

- ☐ a stable slope with a minimum factor of safety of 1.5 under static conditions and 1.1 under seismic loading
- ☐ a toe erosion allowance
- ☐ a 6 m access allowance and top of slope

Three slope cross sections were studied as the worst-case scenario. The cross-section locations are presented on Drawing PG5828-2 – Limit of Hazard Lands Plan attached to this report.

Stable Slope Setback

The analyses of the stability of the slopes were carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. Minimum factors of safety of 1.5 and 1.1 are generally recommended for static and seismic conditions, respectively, where the failure of the slope would endanger permanent structures.

The cross-sections were analysed using the existing slope geometry from the topographical site survey provided by the client and information collected during our site visit. The slope stability analysis was completed at the slope cross-sections under worst-case-scenario by assigning cohesive soil layers as being fully saturated.

Subsoil conditions at the cross-section locations were determined based on test holes coverage conducted within the subject site. The soil profile used in the slope stability analysis for cross section 1 was based on borehole BH 1-22 and that for cross sections 2 and 3 was based on BH 2-22 and BH 3-22. The soil profile considered in the slope stability analysis generally consists of stiff to very stiff silty clay underlain by glacial till. Within the vicinity of cross sections 2 and 3, the clay consists of a brown silty clay crust underlain by a stiff grey silty clay. For a conservative review of the groundwater conditions, the silty clay deposit was noted to be fully saturated for our analysis.

Table 1 – Effective Stress Soil Parameters (Static – Drained Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay	17	33	5
Grey Silty Clay	16	33	10
Glacial Till	20	36	5

Table 2– Total Stress Soil Parameters (Seismic - Undrained Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay	17	-	150
Grey Silty Clay	16	-	65
Glacial Till	20	36	5

Static Loading Analysis

The results are shown in Figures 1, 3, and 5. The results indicate a slope with a factor of safety of 1.16, 1.66, and 0.4 at Sections 1, 2, and 3, respectively. Based on these results, a stable slope setback varying between 7 and 9 m from the top of the slope are required for sections 1-1 and 3-3 to achieve a factor of safety of 1.5 for the limit of the hazard lands in the park area. Section 2-2 will not require a stable slope allowance.

Seismic Loading Analysis

An analysis considering seismic loading and the groundwater at ground surface was also completed. A horizontal acceleration of 0.16g was considered for all slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading. The results of the analyses including seismic loading are shown in Figures 2, 4, and 6. The results indicate a slope with a factor of safety greater than 1.1 at all sections. However, it should be noted that the stable slope setback associated with our static loading analysis governs the required stable slope setback required for static conditions.

Toe Erosion and Access Allowances

Based on the soil profiles encountered at the borehole locations and the soil encountered throughout the river, a stiff grey silty clay is anticipated to be subject to erosion activity by the river flow. Based on the encountered soils and the observed active erosion, a toe erosion allowance of 5 m should be applied for the subject slope. Furthermore, a minimum 6 m access allowance should be considered.

Limit of Hazard Lands

Based on the above, a setback taken from the top of the current slope has been provided as based on the above-noted observations and analysis. Reference should be made to Drawing PG5828-2 – Limit of Hazard Lands Plan for the proposed River Park at the subject site.

Drainage Requirements

It should be noted that the following should be considered for the proposed park:

- ❒ It is important to avoid directing uncontrolled water towards the slope (drainage, gutter, pool drainage, etc.)
- ❒ It is important to avoid overloading the top of the slope (backfill, fill, miscellaneous waste, grass cuttings, branches, leaves, snow, etc.)
- ❒ It is important to avoid excavating at the base of the slope.
- ❒ It is important to maintain a healthy native vegetation cover.
- ❒ Any future additions, such as aboveground swimming pools or accessory buildings, should entail reassessment of slope stability unless this has been pre-confirmed via supplementary slope stability analyses during the design stage.

3.0 Conclusions

The recommendations provided in this letter report are in accordance with Paterson's present understanding of the project. Should any conditions at the site be encountered which differ from our site observations, Paterson requests immediate notification to permit reassessment of the recommendations.

The present letter 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 her agents, is not authorized without review by Paterson Group Inc. for the applicability of our recommendations to the altered use of the report.

We trust this report meets your present requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
October 17, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

Maha Saleh, M.A.Sc., P.Eng.

David J. Gilbert, P.Eng

Attachments

☐ Soil Profile and Test Data Sheets
☐ Symbols
☐ Figures 1 to 6 - Sections for Slope Stability Analysis
☐ Drawing PG5828-2 – Limit of Hazard Lands Plan

Report Distribution

☐ Uniform Developments (e-mail copy)
☐ Paterson Group (1 copy)

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed River Park - 4386 Rideau Valley Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 16, 2022

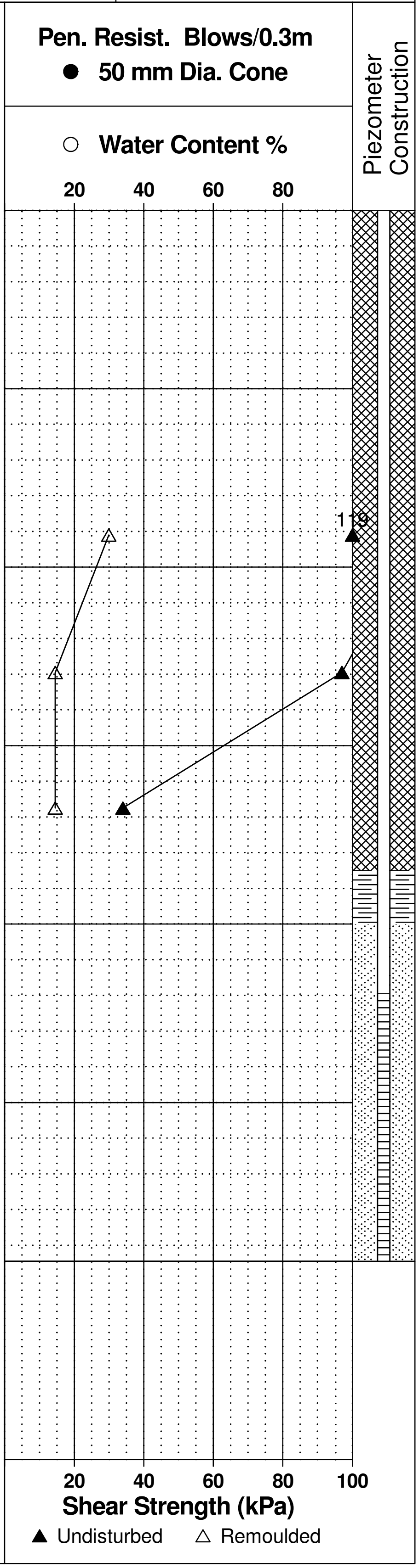
FILE NO. **PG5828**

HOLE NO. **BH 1-22**

SOIL DESCRIPTION	Depth (m)
Ground Surface	
TOPSOIL	0.25
Very stiff to stiff, brown **SILTY CLAY,** trace sand and gravel	0.69
Very stiff to firm, brown **SILTY CLAY** - grey by 3.0m depth	3.50
GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders, trace clay	5.89
End of Borehole Practical refusal to augering at 5.89m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	7
SS	3	100	P
SS	4		P
SS	5	25	P
SS	6	17	15
SS	7	8	10
SS	8	38	50+

DEPTH (m)	ELEV. (m)
0	83.47
1	82.47
2	81.47
3	80.47
4	79.47
5	78.47



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed River Park - 4386 Rideau Valley Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

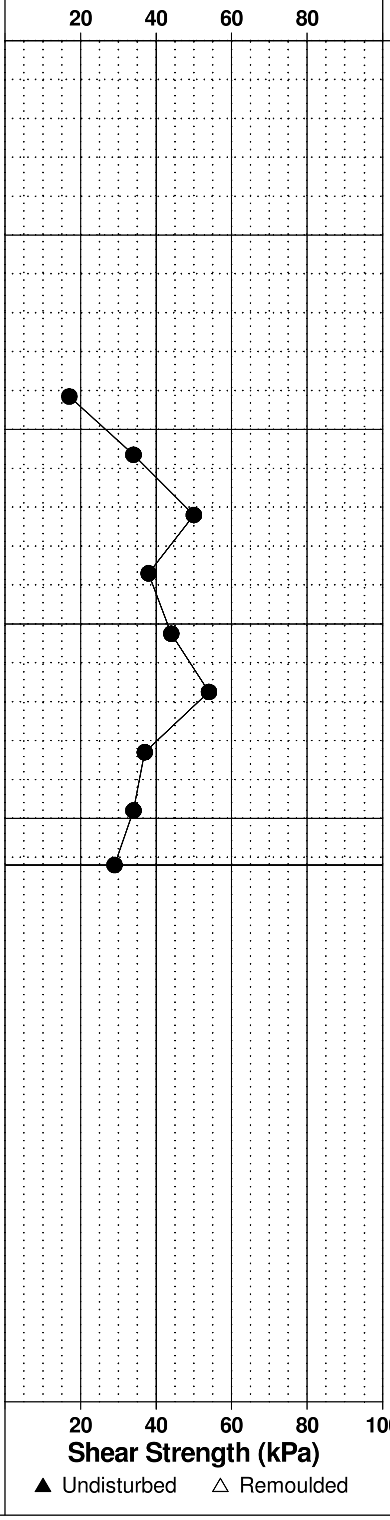
BORINGS BY Track-Mount Power Auger

DATE June 16, 2022

FILE NO. **PG5828**

HOLE NO. **BH 2-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
Ground Surface									
TOPSOIL 0.33						0	85.56		
Stiff to firm, brown **SILTY CLAY,** some sand		AU	1						
		SS	2	75	9	1	84.56		
1.45									
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders 1.65		SS	3	100	50+				
Dynamic Cone Penetration Test commenced at 1.65m depth.						2	83.56		
						3	82.56		
						4	81.56		
4.24									
End of Borehole Practical DCPT refusal at 4.24m depth (BH dry upon completion)									



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed River Park - 4386 Rideau Valley Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 16, 2022

FILE NO. PG5828

HOLE NO. BH 2A-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
Ground Surface						0	85.74		
OVERBURDEN						1	84.74		
1.52									
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders	^^^	SS	1	67	42	2	83.74		
2.67									
End of Borehole Practical DCPT refusal at 2.67m depth (BH dry upon completion)									

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the 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.
p	-	Push spoon sampling

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 = $(D30)^2 / (D10 \times D60)$
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

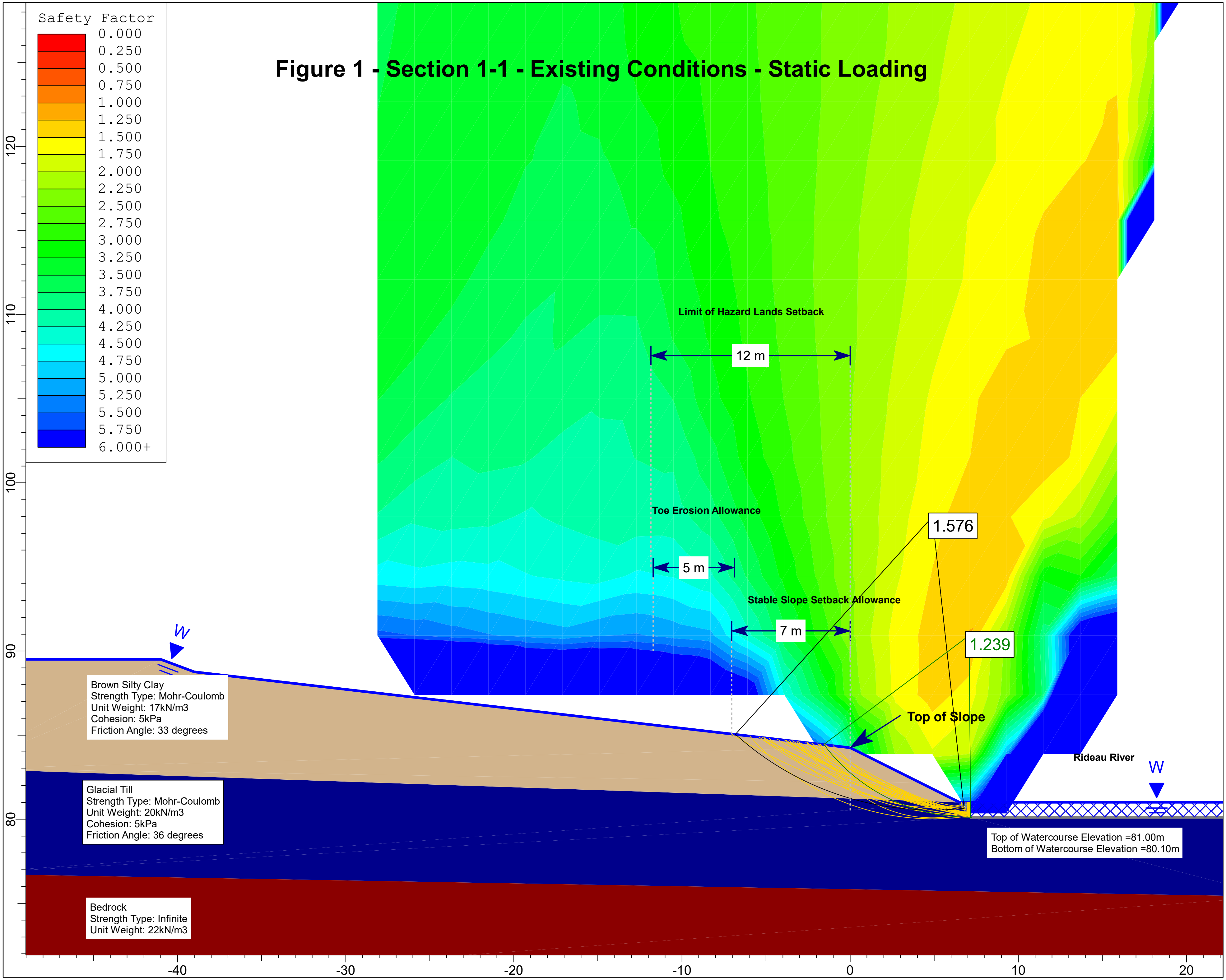
p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

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.

Figure 1 - Section 1-1 - Existing Conditions - Static Loading



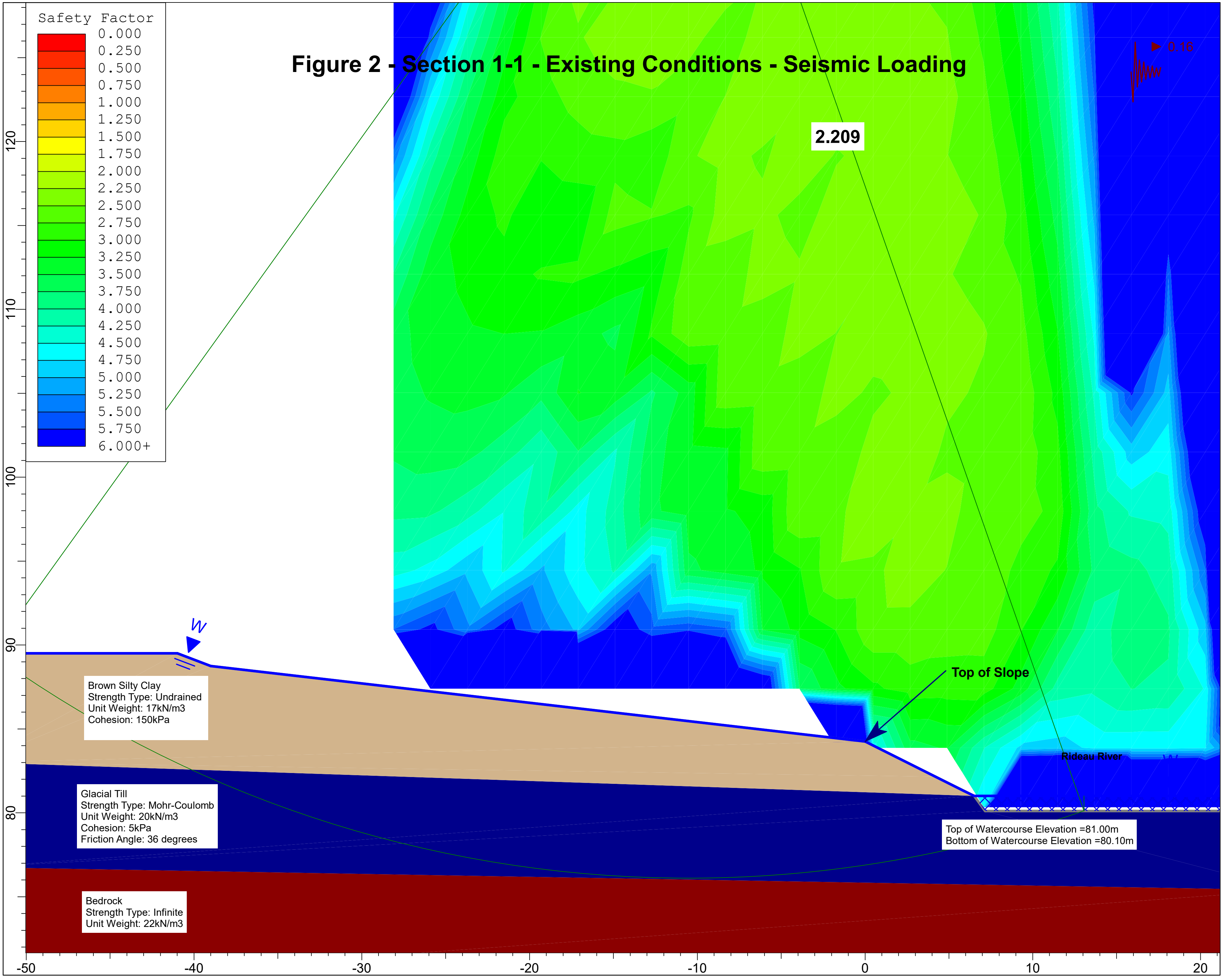


Figure 2 - Section 1-1 - Existing Conditions - Seismic Loading

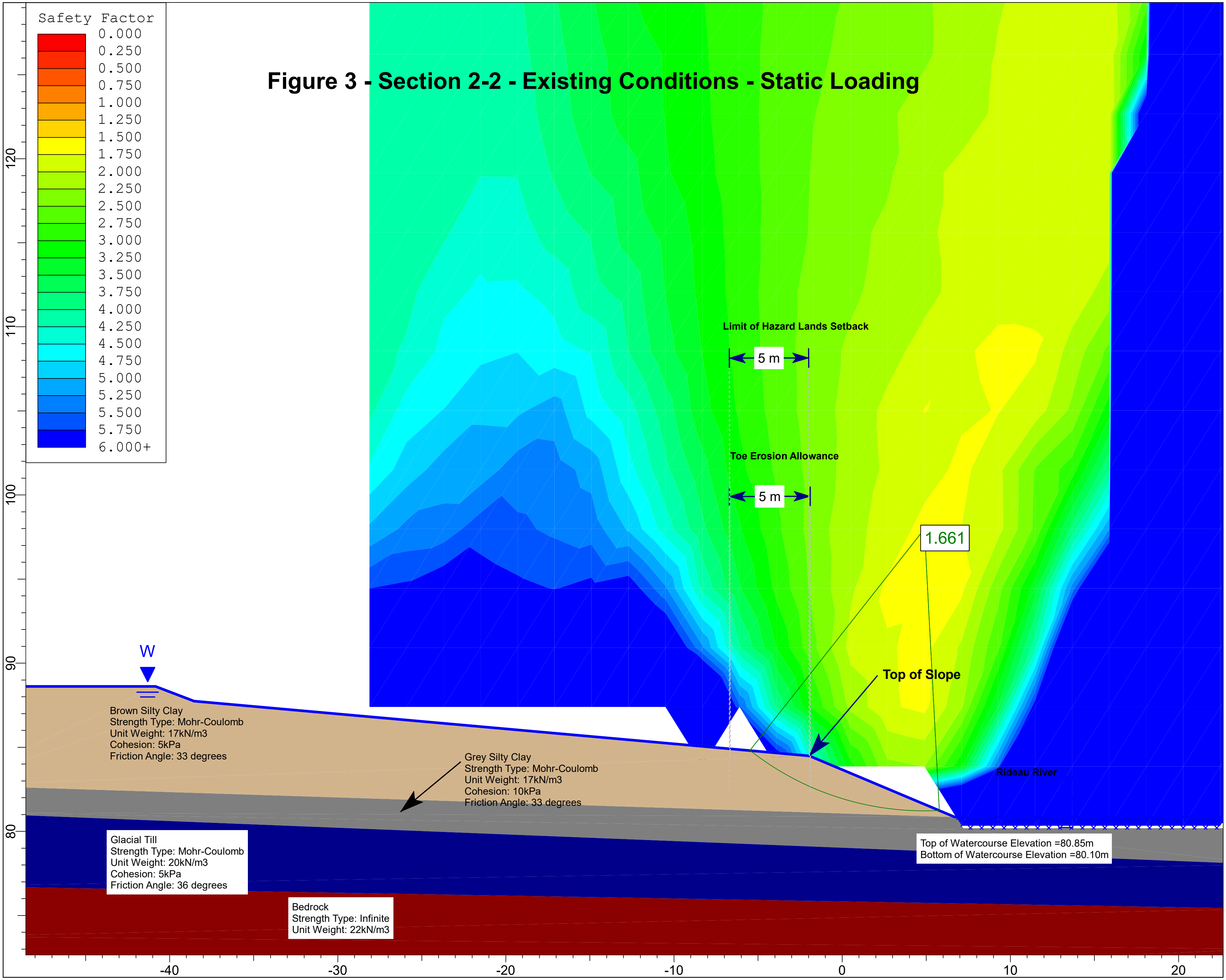


Figure 3 - Section 2-2 - Existing Conditions - Static Loading

Brown Silty Clay
Strength Type: Mohr-Coulomb
Unit Weight: 17kN/m3
Cohesion: 5kPa
Friction Angle: 33 degrees

Grey Silty Clay
Strength Type: Mohr-Coulomb
Unit Weight: 17kN/m3
Cohesion: 10kPa
Friction Angle: 33 degrees

Glacial Till
Strength Type: Mohr-Coulomb
Unit Weight: 20kN/m3
Cohesion: 5kPa
Friction Angle: 36 degrees

Bedrock
Strength Type: Infinite
Unit Weight: 22kN/m3

Top of Watercourse Elevation =80.85m
Bottom of Watercourse Elevation =80.10m

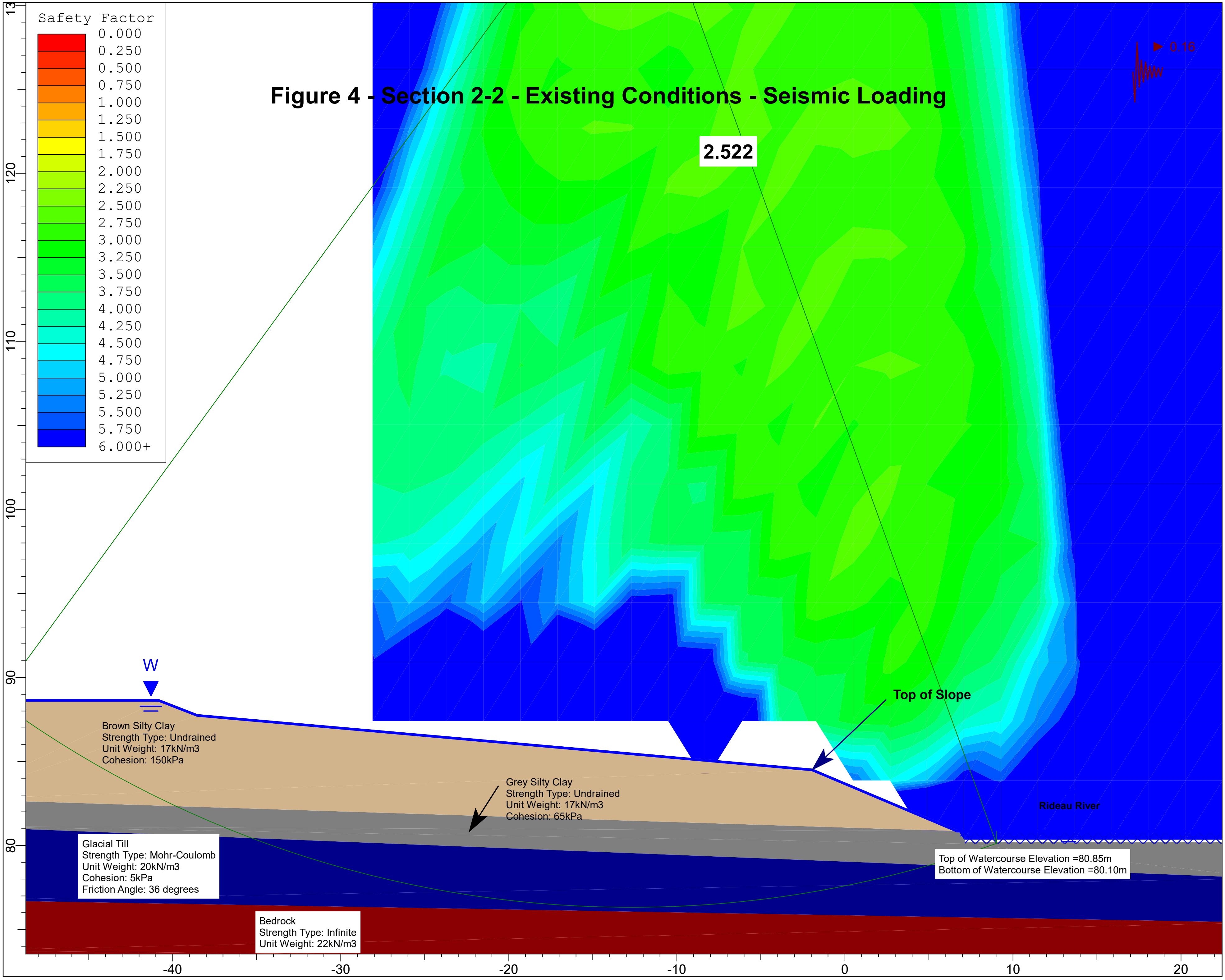


Figure 4 - Section 2-2 - Existing Conditions - Seismic Loading

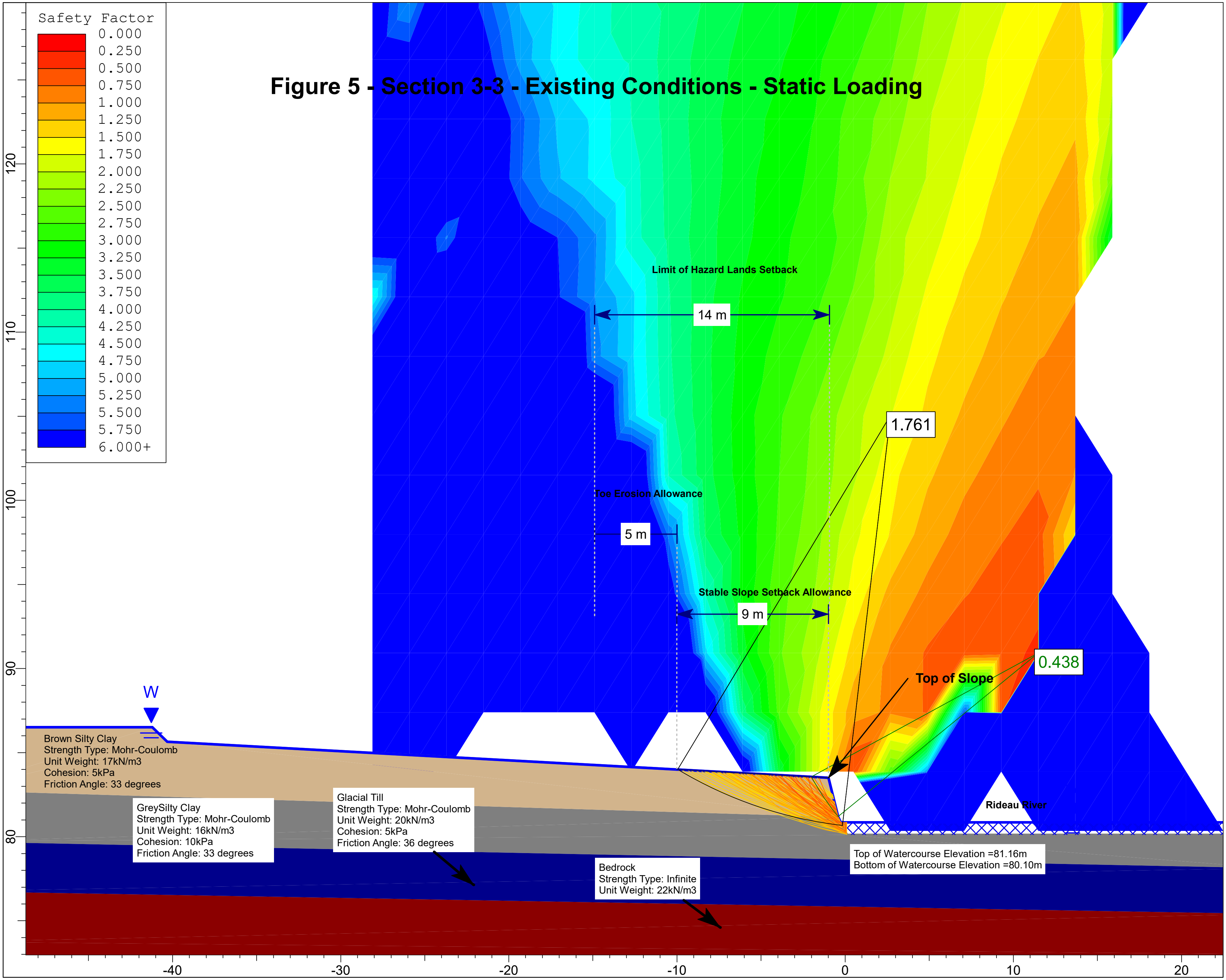


Figure 5 - Section 3-3 - Existing Conditions - Static Loading

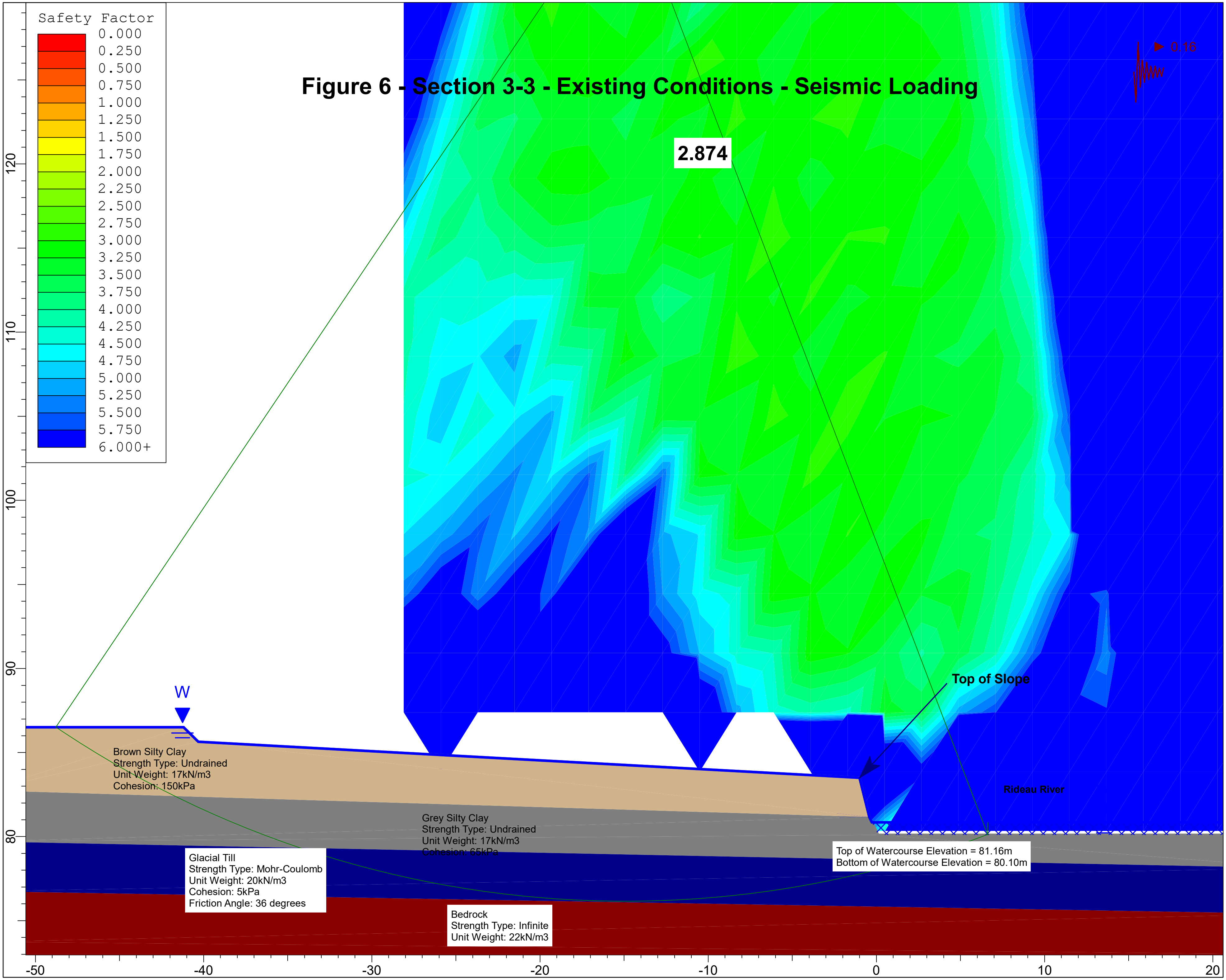
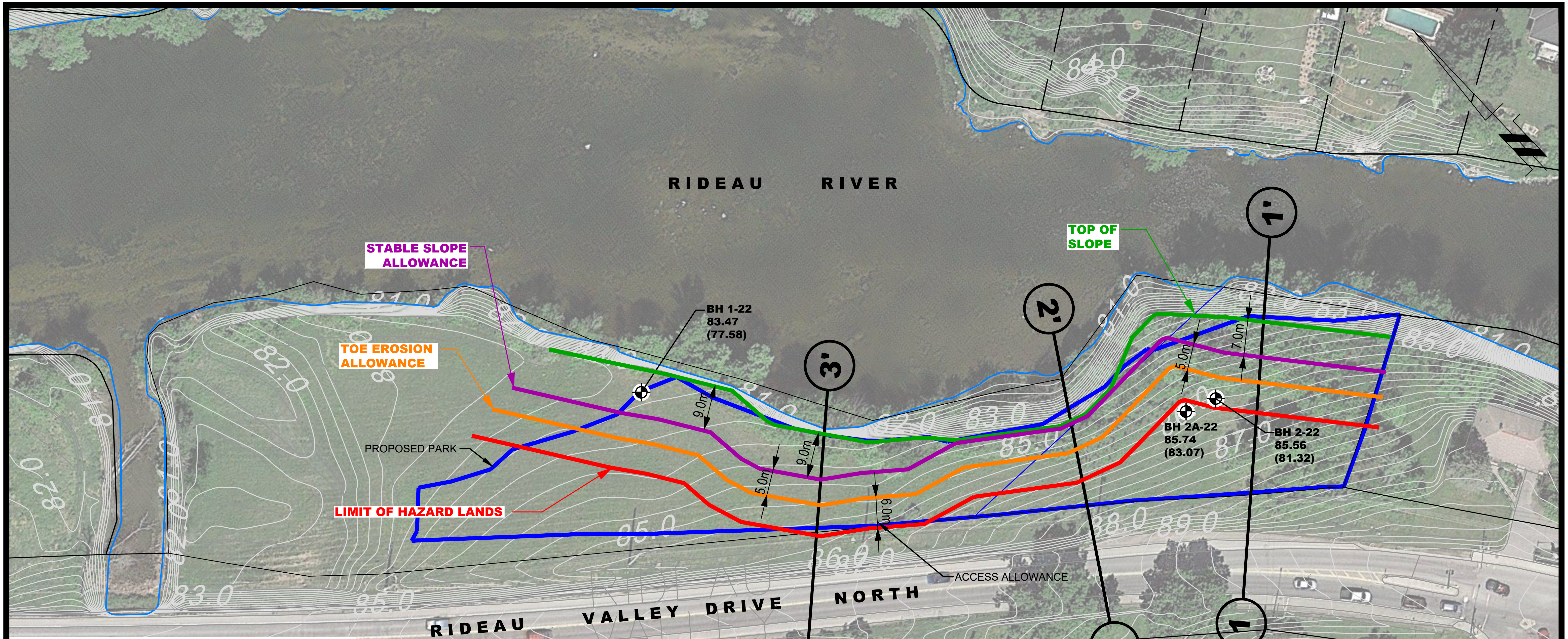


Figure 6 - Section 3-3 - Existing Conditions - Seismic Loading



LEGEND:

⊕	BOREHOLE LOCATION
(1)——(1')	SLOPE STABILITY CROSS-SECTION
85.74	GROUND SURFACE ELEVATION (m)
(83.07)	PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:750

0 10 20 30 50m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

UNIFORM DEVELOPMENTS

SLOPE STABILITY ANALYSIS
PROPOSED PARK
RIDEAU VALLEY DRIVE

MANOTICK, ONTARIO

Title:

LIMIT OF HAZARD LANDS PLAN

Scale: 1:750
Date: 06/2022
Drawn by: NFRV
Report No.: PG5828-LET.01
Checked by: MS
Dwg. No.: PG5828-2
Approved by: DJG
Revision No.:

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