PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

4386 Rideau Valley Drive
Ottawa, Ontario

Prepared for Uniform Developments

Report PG5828-1 Revision 2 dated October 14, 2022

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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.

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.

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.

Sample Storage

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

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 Ruisseau Creek along its center and bordered to the west by a tributary channel. 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 and BH 6-21.

Glacial till was encountered below the clay deposit at BH 2-21 and BH 9-21. 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 and 8.8 m at BH 3-21 and BH 5-21, 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.

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. 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.

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 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 could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

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.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

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.

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

Based on the undrained shear strength testing results, it is recommended that a permissible grade raise restriction of **2.0 m** be implemented for the subject site. 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.

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. 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.

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.

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 industrial building. 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.

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.

It is expected that deep service trenches in excess of 3 m will 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. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations 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 a tributary creek to Ruisseau Mud Creek and the main channel of Ruisseau Mud Creek, respectively. The existing slope conditions were reviewed by Paterson field personnel as part of the geotechnical investigation on May 19, 2021. Five (5) slope cross-sections were studied as the worst case scenarios. 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 watercourse 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 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 such that bank material did not appear to be exposed directly to stream flow. No active erosion or sloughing of the banks were observed at the time of our visit.

The creek was generally observed to consist of a tributary to the Ruisseau 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 the watercourse to the north of the subject site is generally heavily vegetated with brush and some trees. Ruisseau 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 Ruisseau 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 Ruisseau 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 the 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-sections were inferred based on nearby boreholes. 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.

Static Loading Analysis

The results are shown in Figures 2, 4, 6 and 8 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 and 9 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. The western tributary creek did not appear to be affected by active erosion at the time of our site visit. Based on the anticipated soils, a toe erosion allowance of 5 m and 1 m should be applied from the watercourse edge for the main channel and western tributary, respectively.

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 centre 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.

7.0 Recommendations

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

- Review detailed grading plan(s) from a geotechnical perspective.
- Observation of all bearing surfaces prior to the placement of concrete.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to 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, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than 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
October 14, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

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

David J. Gilbert, 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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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

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

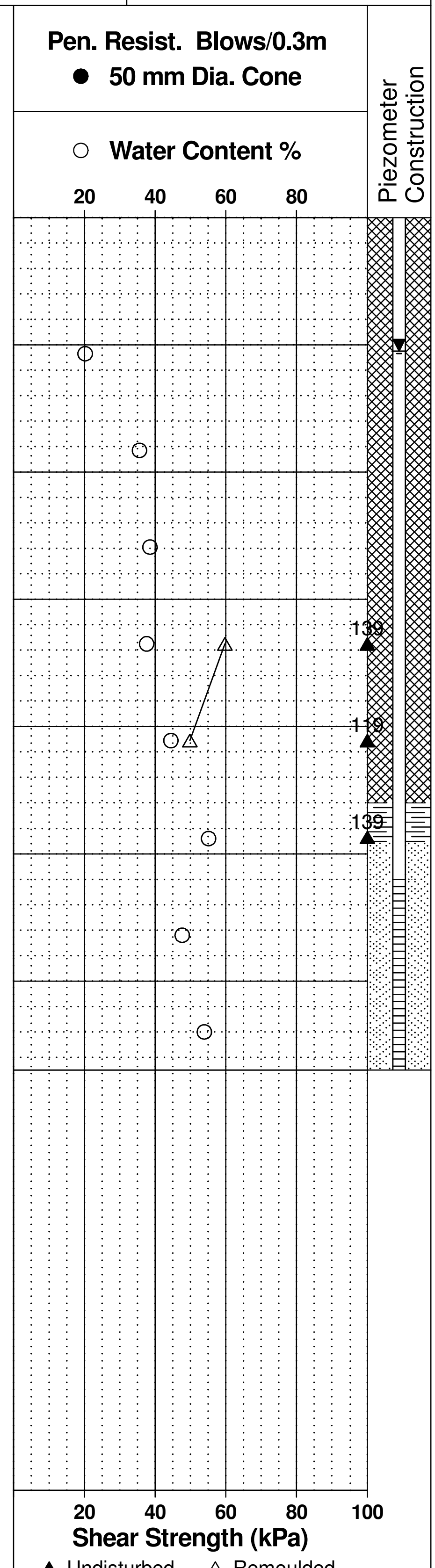
FILE NO. PG5828

HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	88.26
TOPSOIL 0.30	AU	1				
	SS	2	100	13	1	87.26
	SS	3	100	6	2	86.26
	SS	4	100	8		
Hard to very stiff, brown **SILTY CLAY,** trace sand	SS	5	100	7	3	85.26
	SS	6	100	5	4	84.26
	SS	7	83	P	5	83.26
- grey by 5.2m depth	SS	8	83	P		
6.70	SS	9	83	P	6	82.26

End of Borehole

(GWL @ 1.05m - May 26, 2021)



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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 19, 2021

FILE NO. PG5828

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.55
TOPSOIL (0.30)		AU	1				
Hard to very stiff, brown **SILTY CLAY,** some to trace sand		SS	2	100	11	1	88.55
		SS	3	58	8	2	87.55
(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	3	86.55
		SS	6	62	27	4	85.55
		SS	7	60	21	5	84.55
		SS	8	58	15	6	83.55
(6.70)		SS	9	50	10		

End of Borehole

(BH dry - May 26, 2021)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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DATUM Geodetic

FILE NO. PG5828

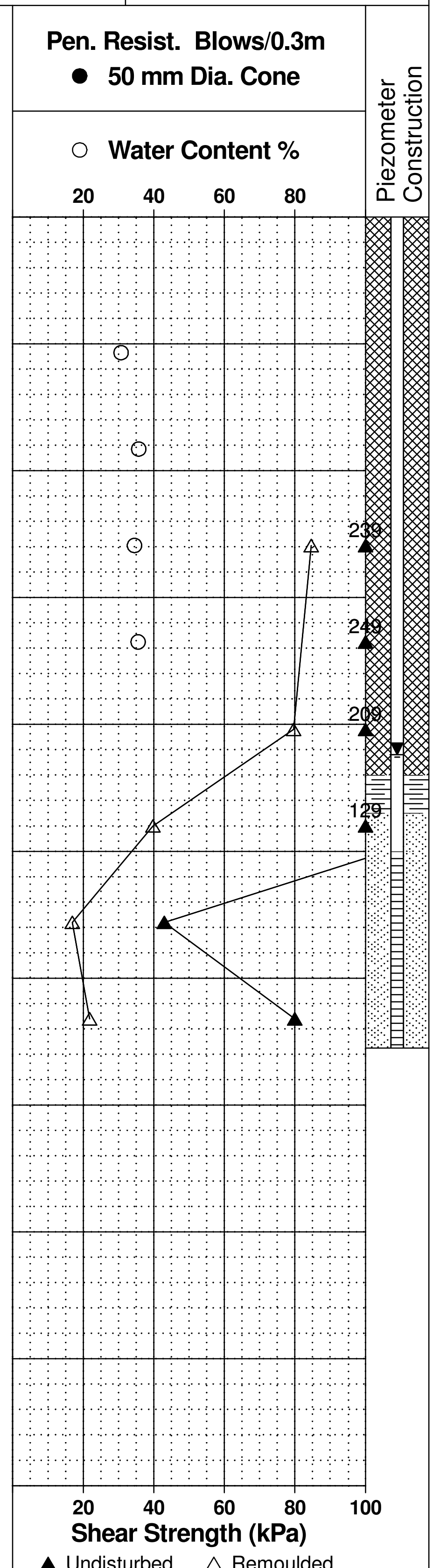
REMARKS

HOLE NO. BH 3-21

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

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

FILE NO. PG5828

REMARKS

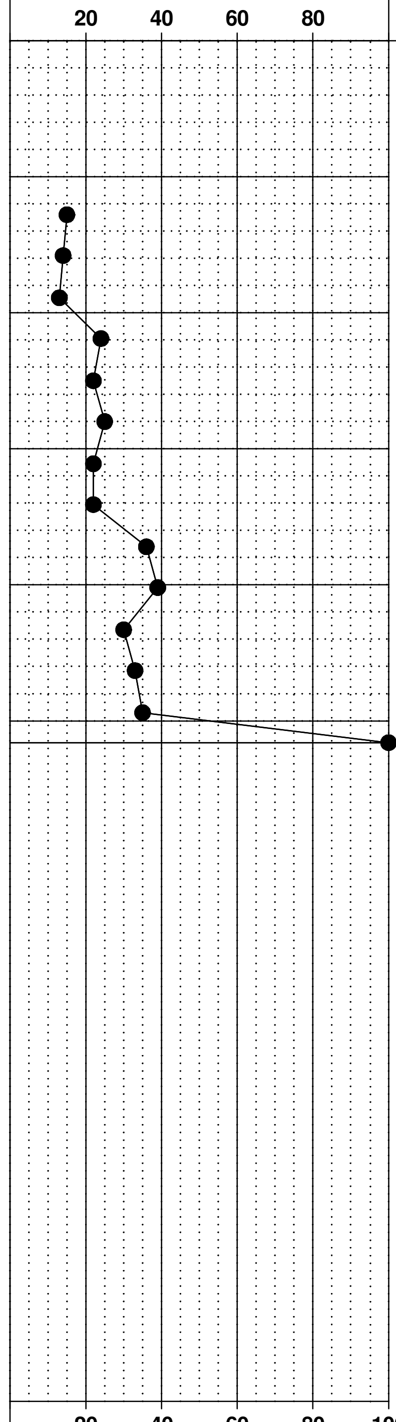
HOLE NO. BH 3-21

BORINGS BY Track-Mount Power Auger **DATE** May 20, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						10	77.89
						11	76.89
						12	75.89
						13	74.89
						14	73.89
15.16						15	72.89
End of Borehole Practical refusal to DCPT at 15.16m depth (GWL @ 4.24m - May 26, 2021)							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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SOIL PROFILE AND TEST DATA

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Proposed Residential Development
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DATUM Geodetic

FILE NO. **PG5828**

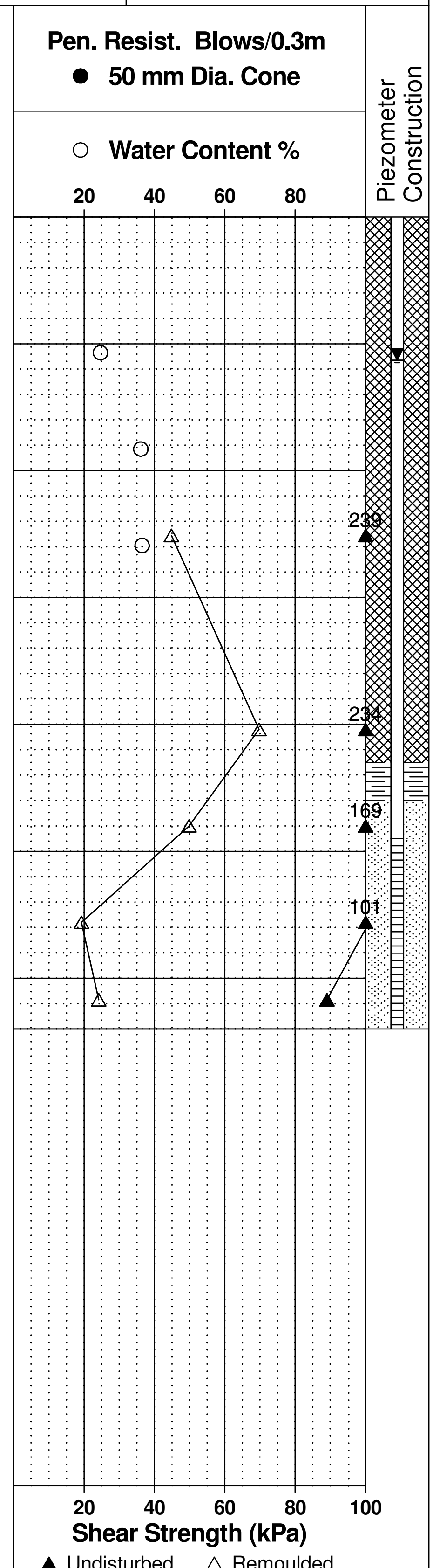
REMARKS

HOLE NO. **BH 4-21**

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL (to 0.36)		AU	1			0	88.11
Hard to very stiff, brown **SILTY CLAY,** some silty sand - sand content decreasing with depth (0.36 to 5.18)		SS	2	83	8	1	87.11
		SS	3	83	5	2	86.11
						3	85.11
		SS	4	100	6	4	84.11
						5	83.11
Stiff, grey **SILTY CLAY** (5.18 to 6.40)						6	82.11
End of Borehole							
(GWL @ 1.13m - May 26, 2021)							



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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

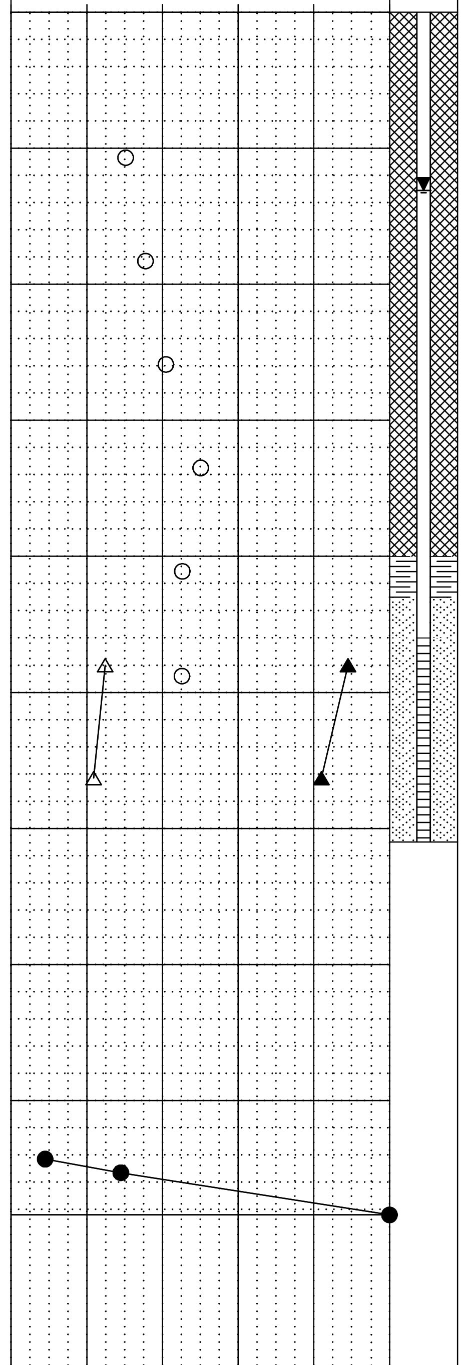
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	85.36
TOPSOIL 0.30							
Compact, brown **SILTY SAND** 0.60		AU	1				
Hard to very stiff, brown **SILTY CLAY**		SS	2	50	10	1	84.36
		SS	3	58	11	2	83.36
		SS	4	83	9	3	82.36
		SS	5	100	7	4	81.36
- stiff and grey by 4.3m depth		SS	6	100	5	5	80.36
6.10		SS	7	100	P	6	79.36
Dynamic Cone Penetration Test commenced at 6.10m depth. Cone pushed to 8.43m depth						7	78.36
8.84						8	77.36
End of Borehole Practical DCPT refusal at 8.84m depth. (GWL @ 1.31m - May 26, 2021)							

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

○ Water Content %

20 40 60 80

Piezometer Construction



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

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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 19, 2021

FILE NO. PG5828

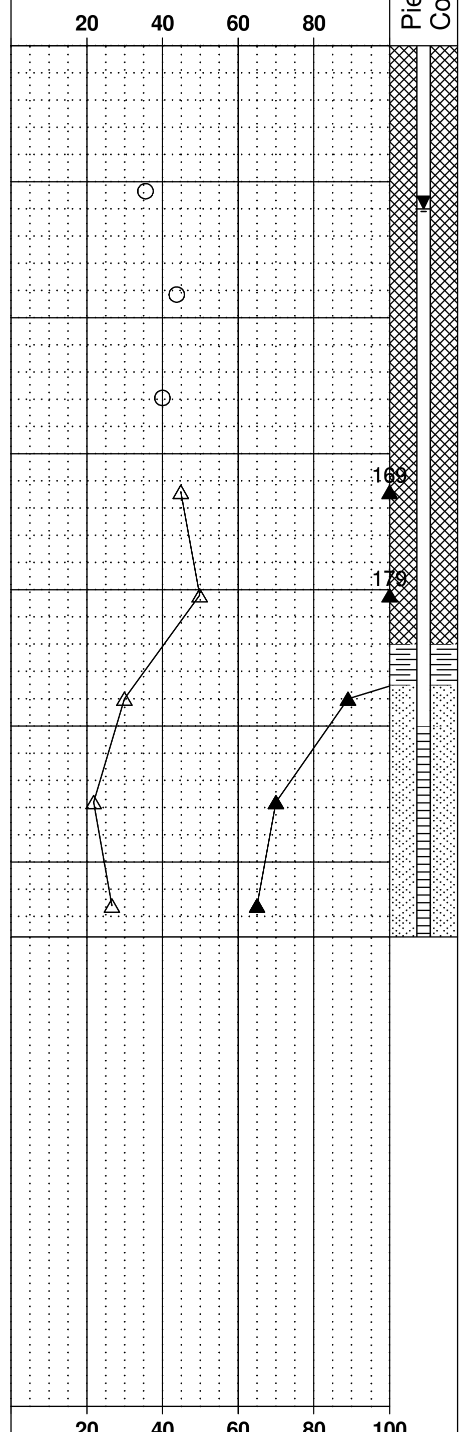
HOLE NO. BH 6-21

SOIL DESCRIPTION	STRATA PLOT	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		SS	4	83	5		
- sand content decreasing with depth						3	82.35
						4	81.35
- grey by 4.6m depth						5	80.35
6.55						6	79.35
End of Borehole							
(GWL @ 1.20m - May 26, 2021)							

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. PG5828

HOLE NO. BH 7-21

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	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		SS	2	75	7	1	86.56
- sand content decreasing with depth		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)

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

○ Water Content %

20 40 60 80

129

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
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REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. PG5828

HOLE NO. BH 8-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20				
FILL: Brown silty sand, some gravel, trace topsoil	1.07	AU	1		
		SS	2	62	30
GLACIAL TILL: Dense to compact, brown silty sand with gravel, cobbles and boulders	6.70	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

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

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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

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REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

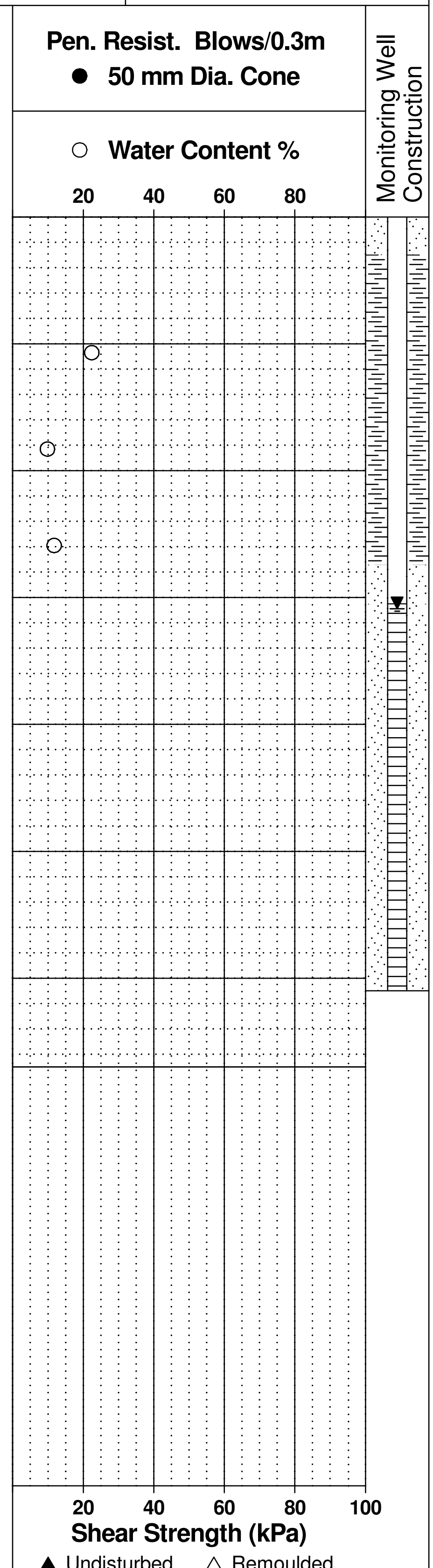
FILE NO. PG5828

HOLE NO. BH 9-21

SOIL DESCRIPTION	Bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	90.52
TOPSOIL	0.30						
Stiff, brown **SILTY CLAY,** some to trace sand	1.52	AU	1				
		SS	2	83	12	1	89.52
						2	88.52
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	6.70	SS	3	75	23	3	87.52
						4	86.52
						5	85.52
						6	84.52
		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.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

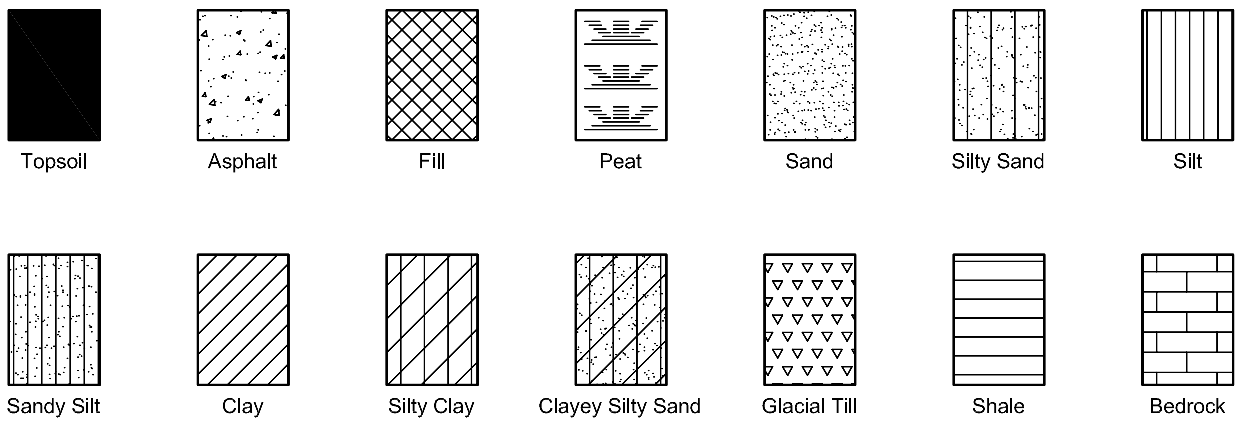
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

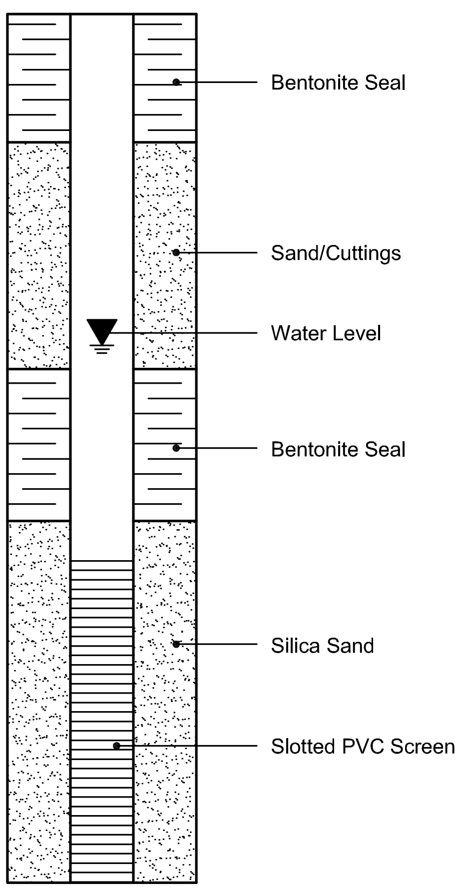
SYMBOLS AND TERMS (continued)

STRATA PLOT

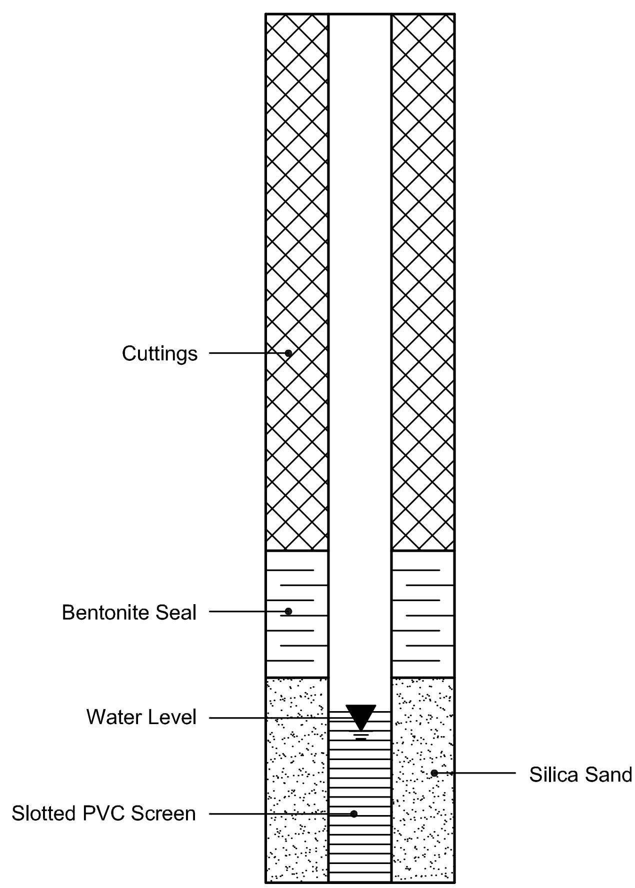


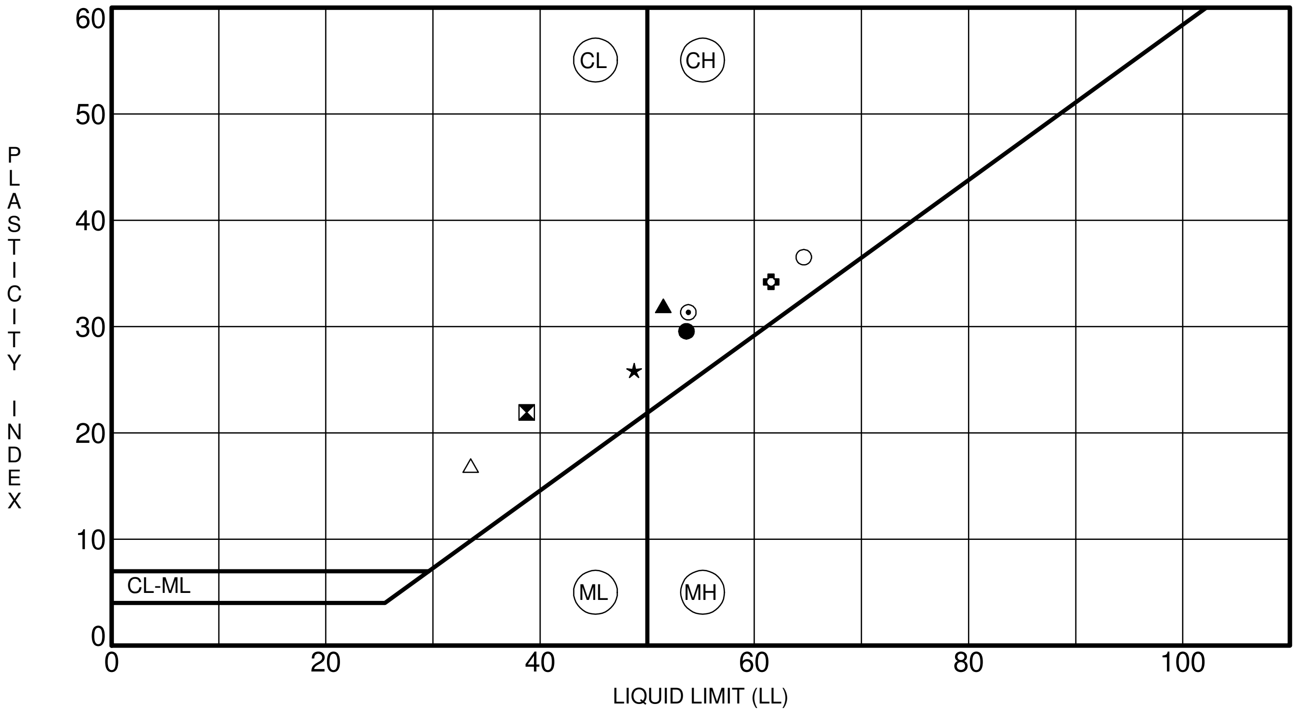
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	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**

FILE NO. **PG5828**

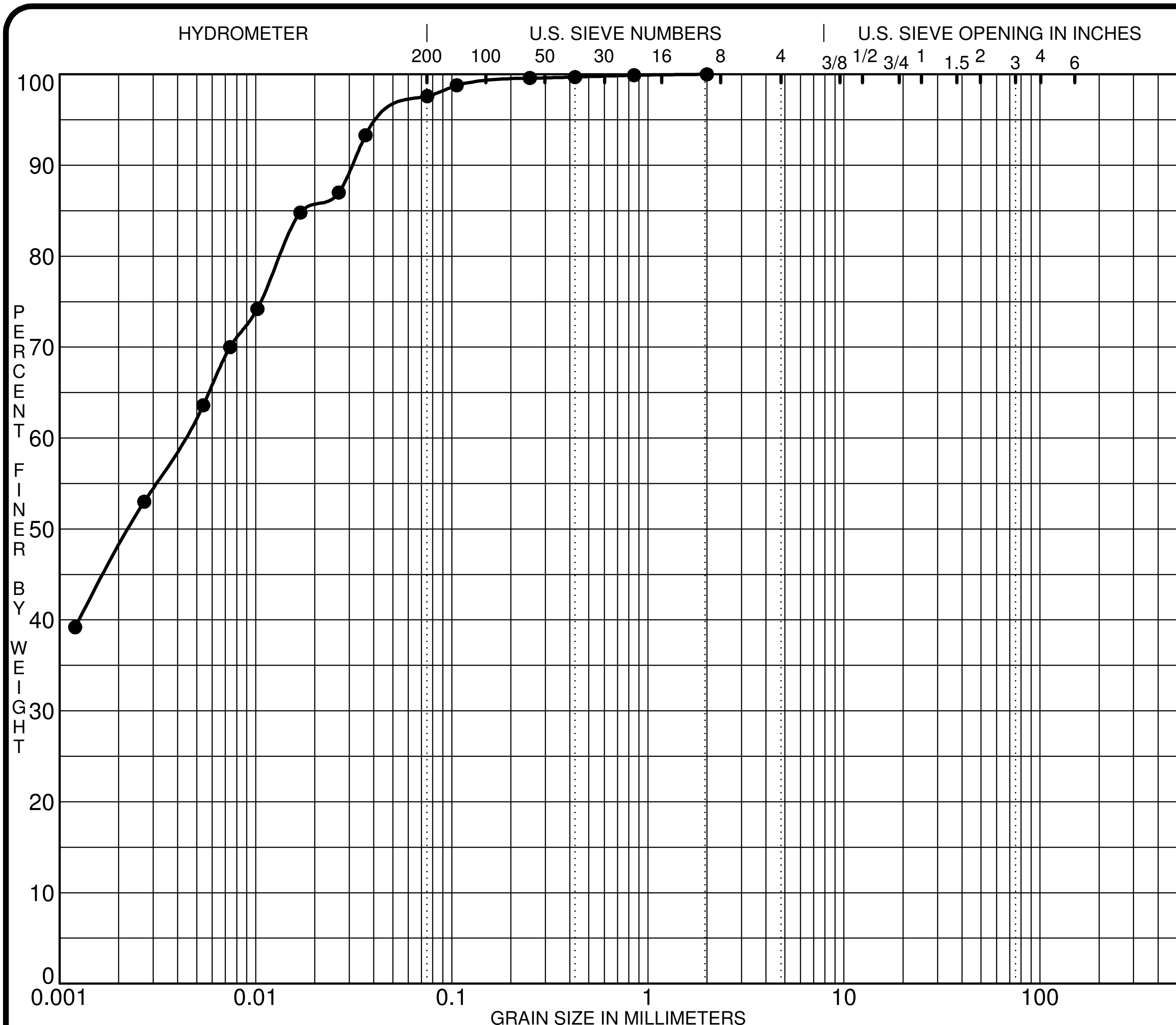
PROJECT **Geotechnical Investigation - Proposed Residential Development**

DATE **20 May 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	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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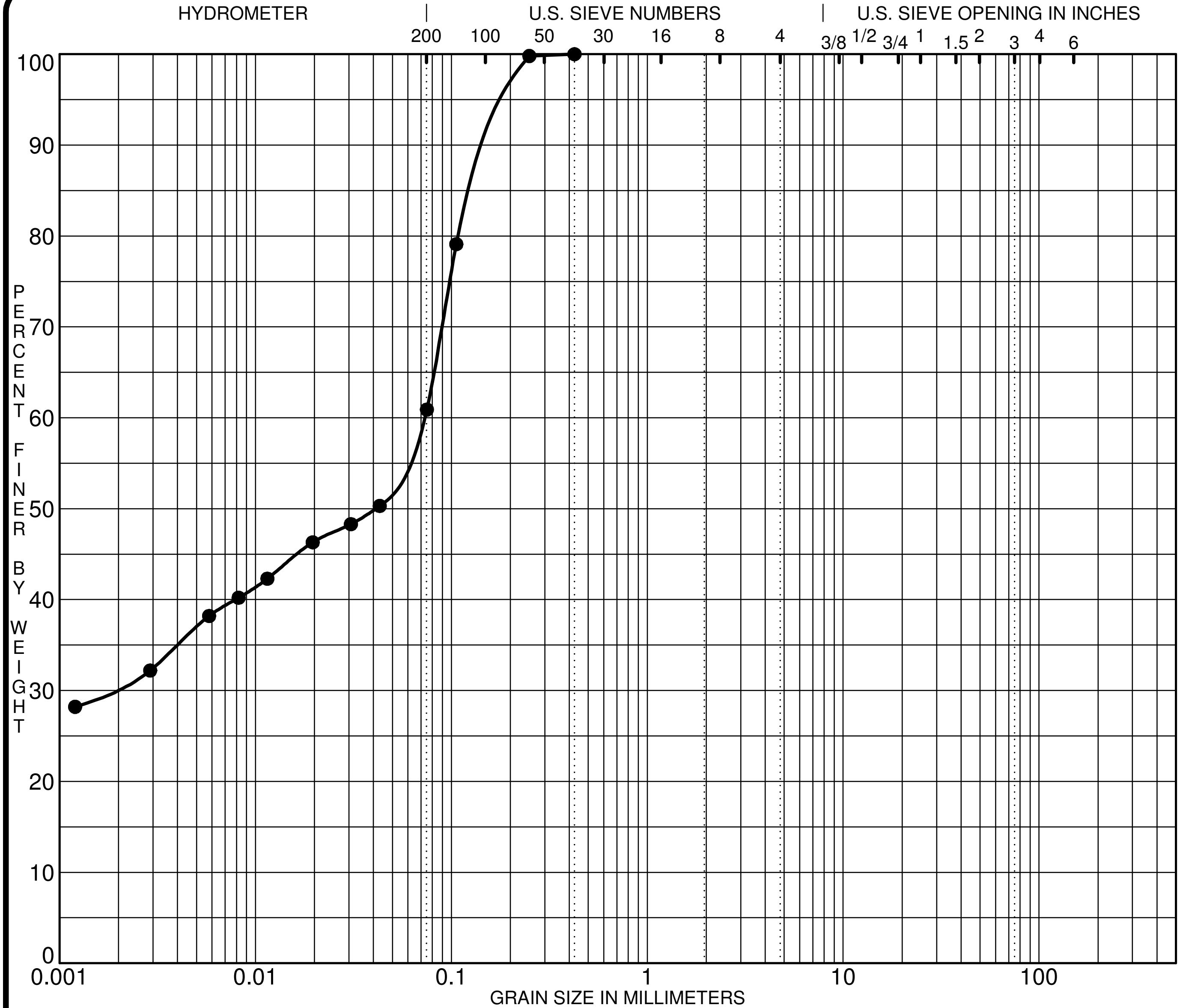
GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	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**	
⊠										
▲										
★										

CLIENT **Uniform Urban Developments** FILE NO. **PG5828**

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patersongroup **Consulting Engineers**

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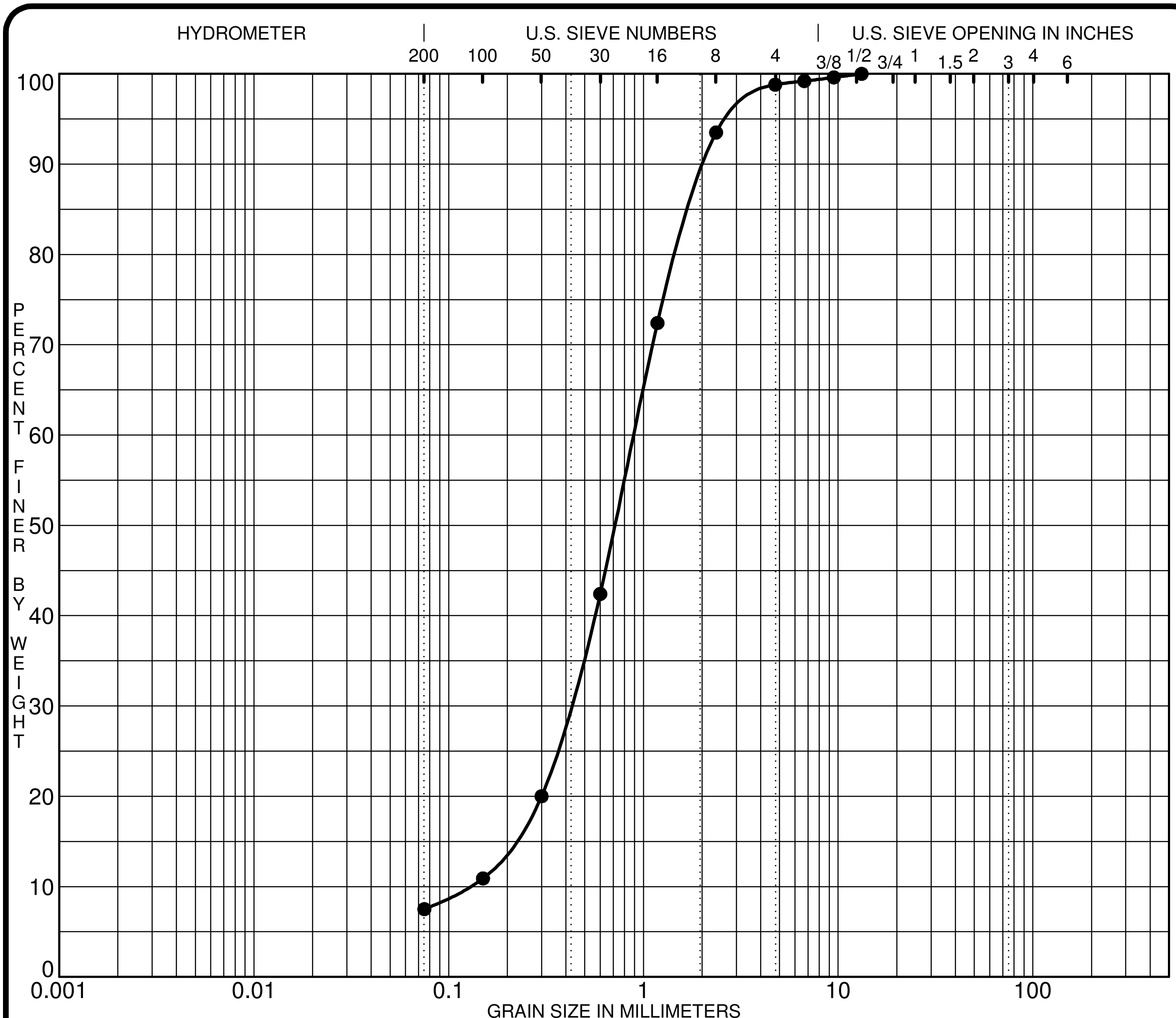
GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

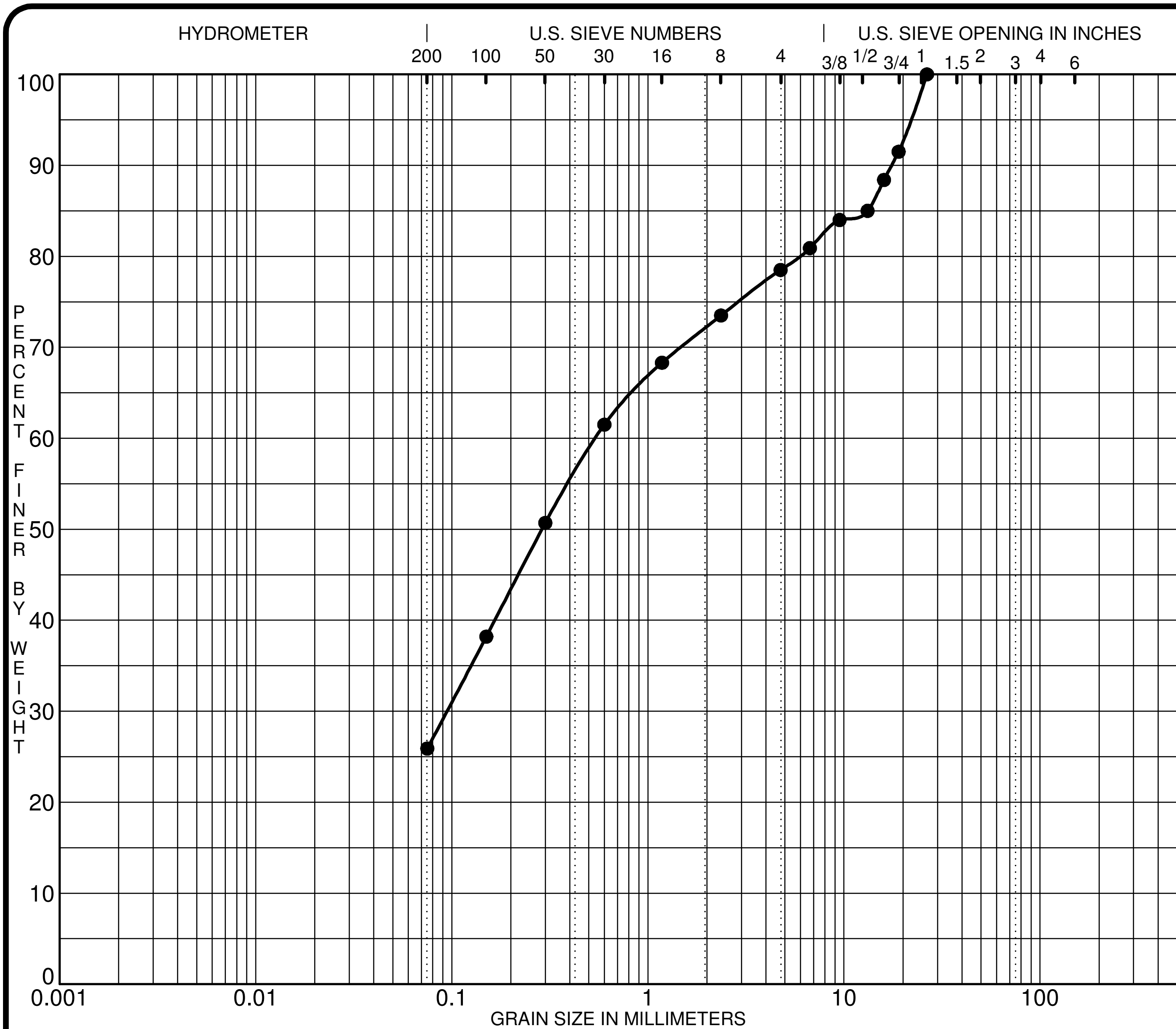
	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	BH 6-21	SS 4	CH - Inorganic clays of high plasticity					1.50	7.1
⊠									
▲									
★									

	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	
⊠										
▲										
★										

CLIENT **Uniform Urban Developments**
FILE NO. **PG5828**
PROJECT **Geotechnical Investigation - Proposed Residential Development**
DATE **19 May 21**

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	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**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

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 9 – SLOPE STABILITY ANALYSIS CROSS SECTIONS

PHOTOGRAPHS FROM SITE VISIT – MAY 19, 2021

DRAWING PG5828-1 – TEST HOLE LOCATION PLAN

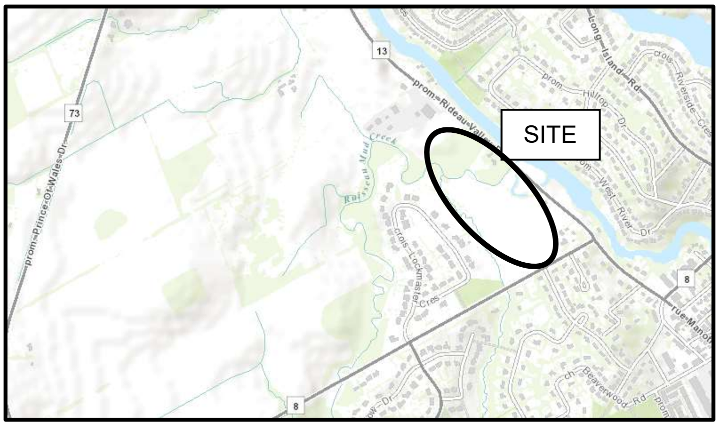


FIGURE 1

KEY PLAN

Figure 2 - Section A - Existing Conditions - Static Loading

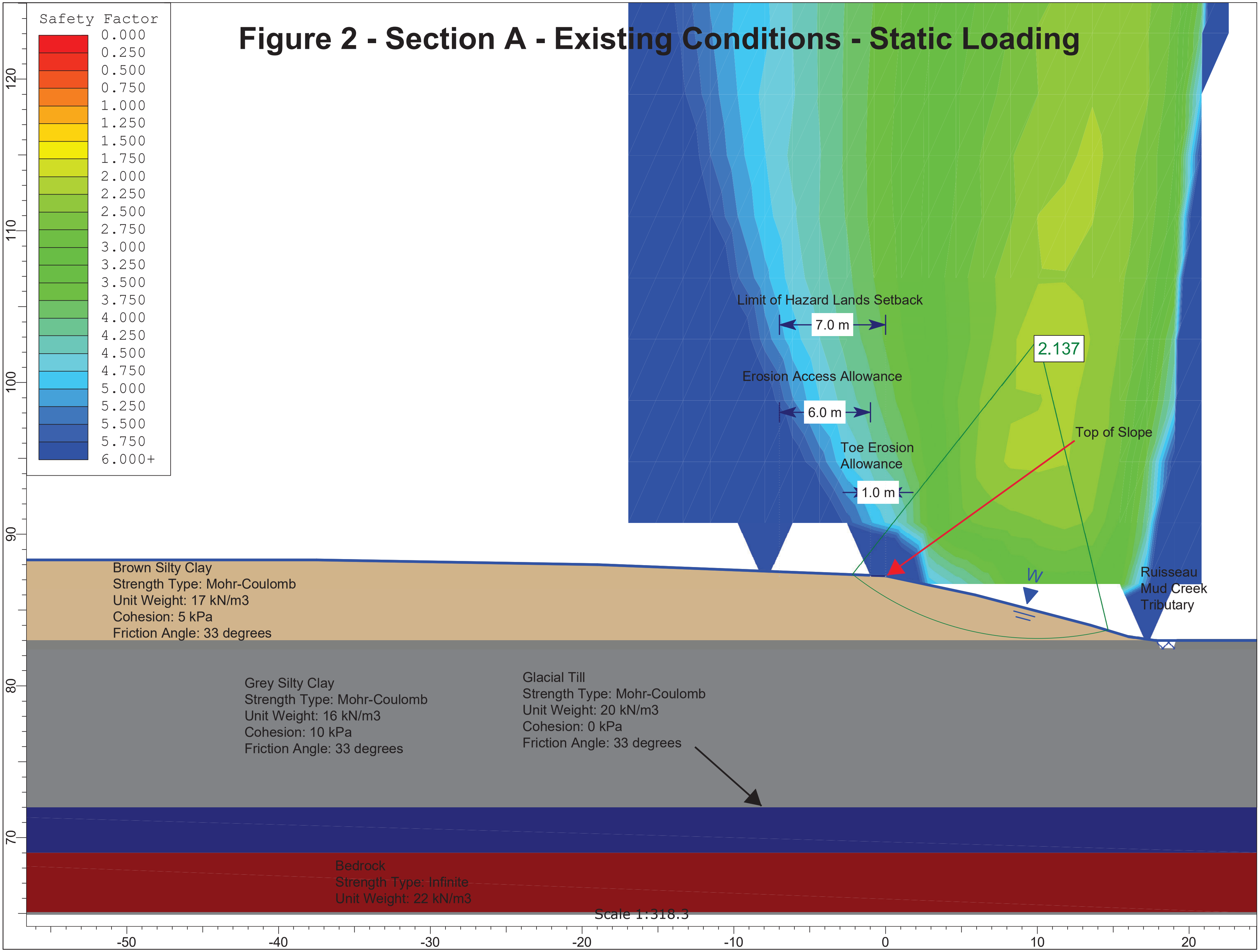


Figure 3 - Section A - Existing Conditions - Seismic Loading

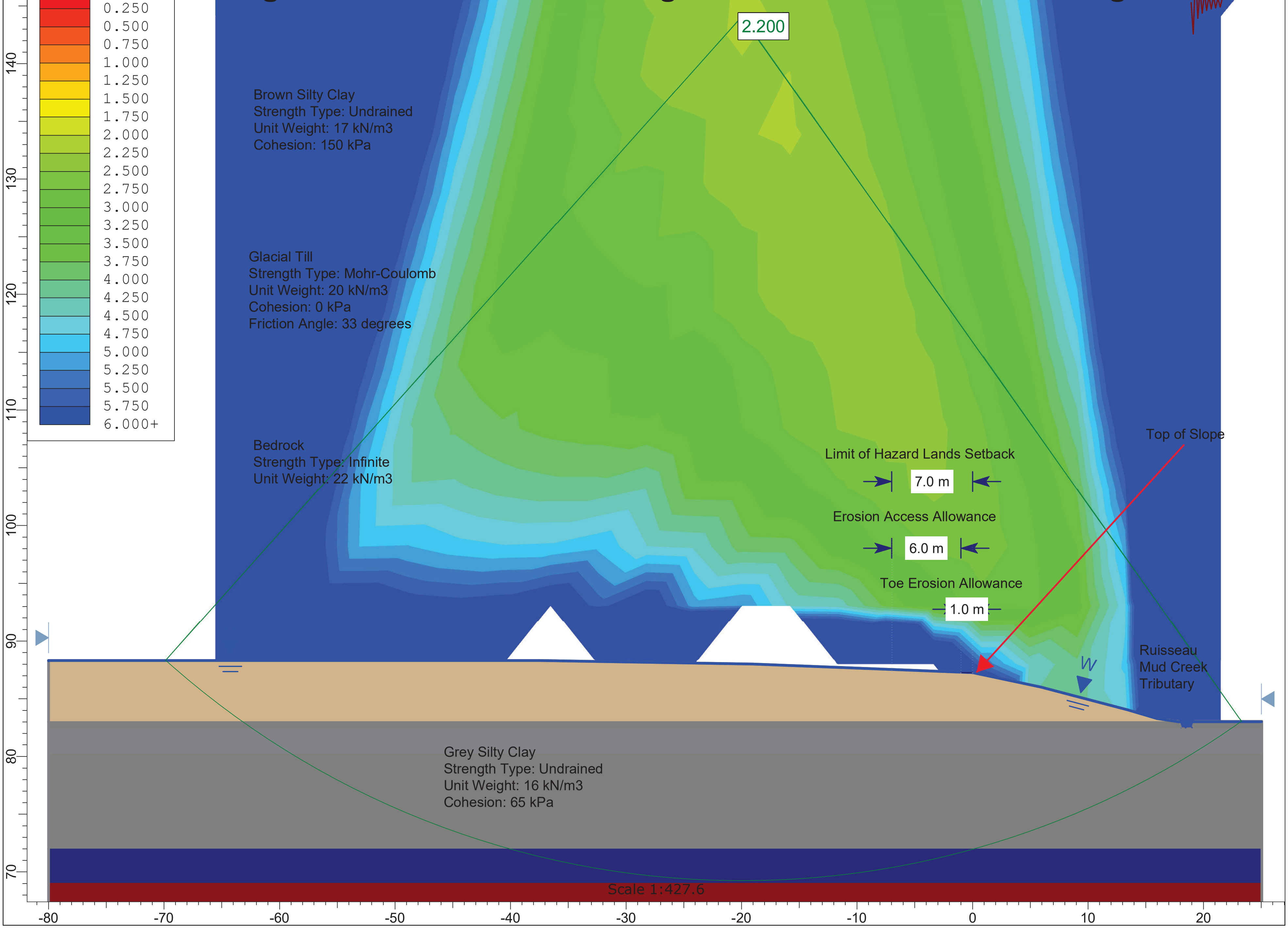


Figure 4 - Section B - Existing Conditions - Static Loading

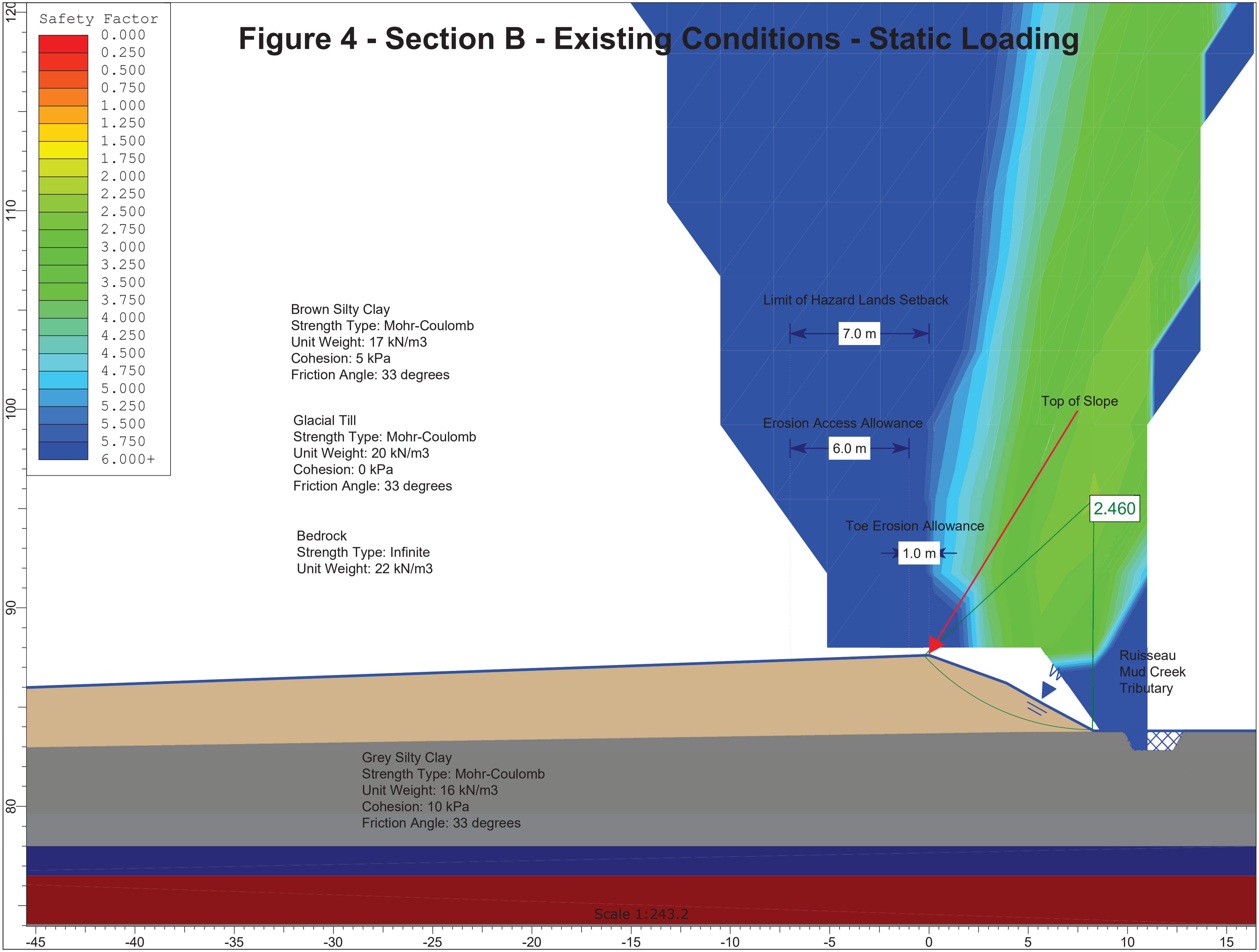


Figure 5 - Section B - Existing Conditions - Seismic Loading

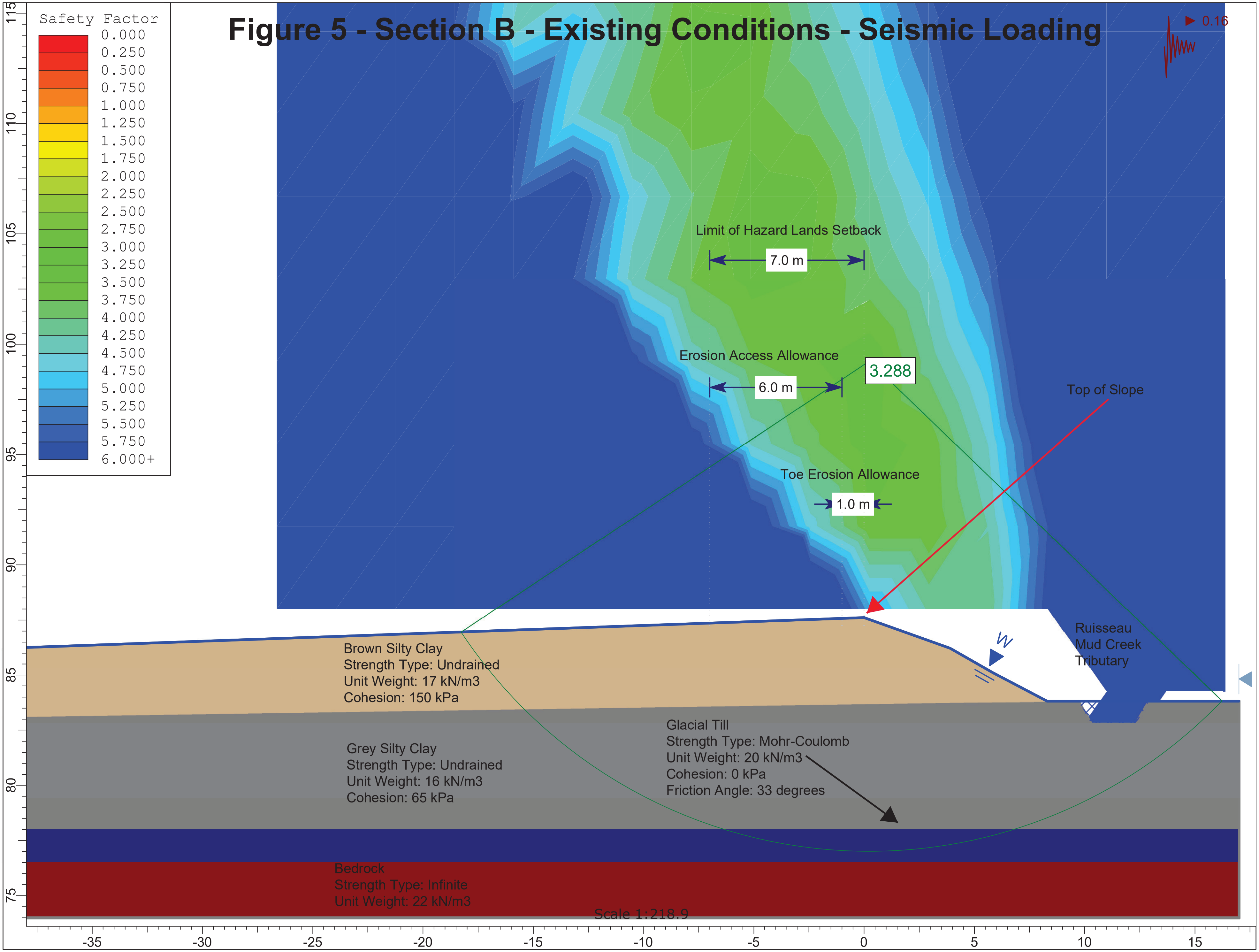


Figure 6 - Section C - Existing Conditions - Static Loading

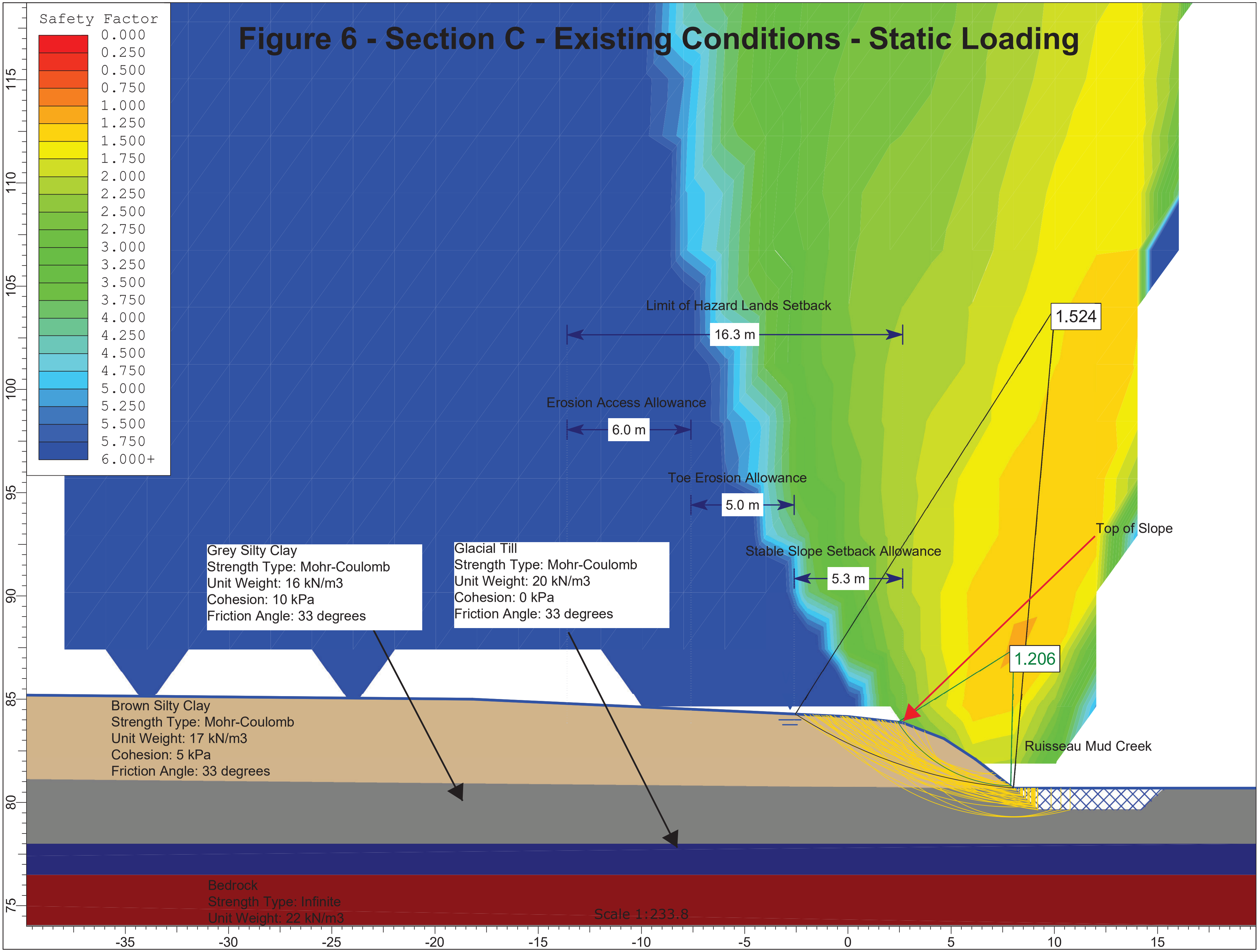


Figure 7 - Section C - Existing Conditions - Seismic Loading

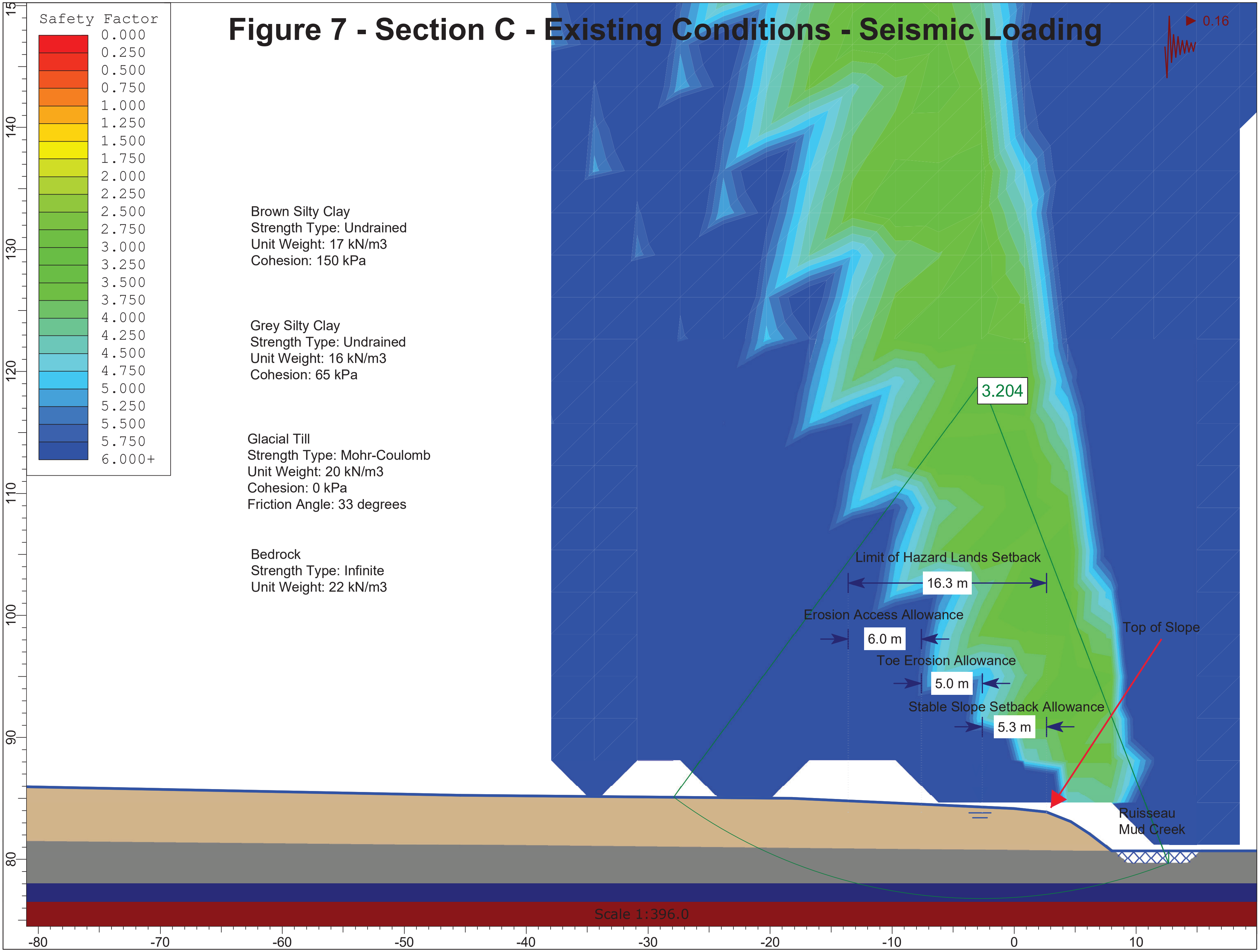


Figure 8 - Section D - Existing Conditions - Static Loading

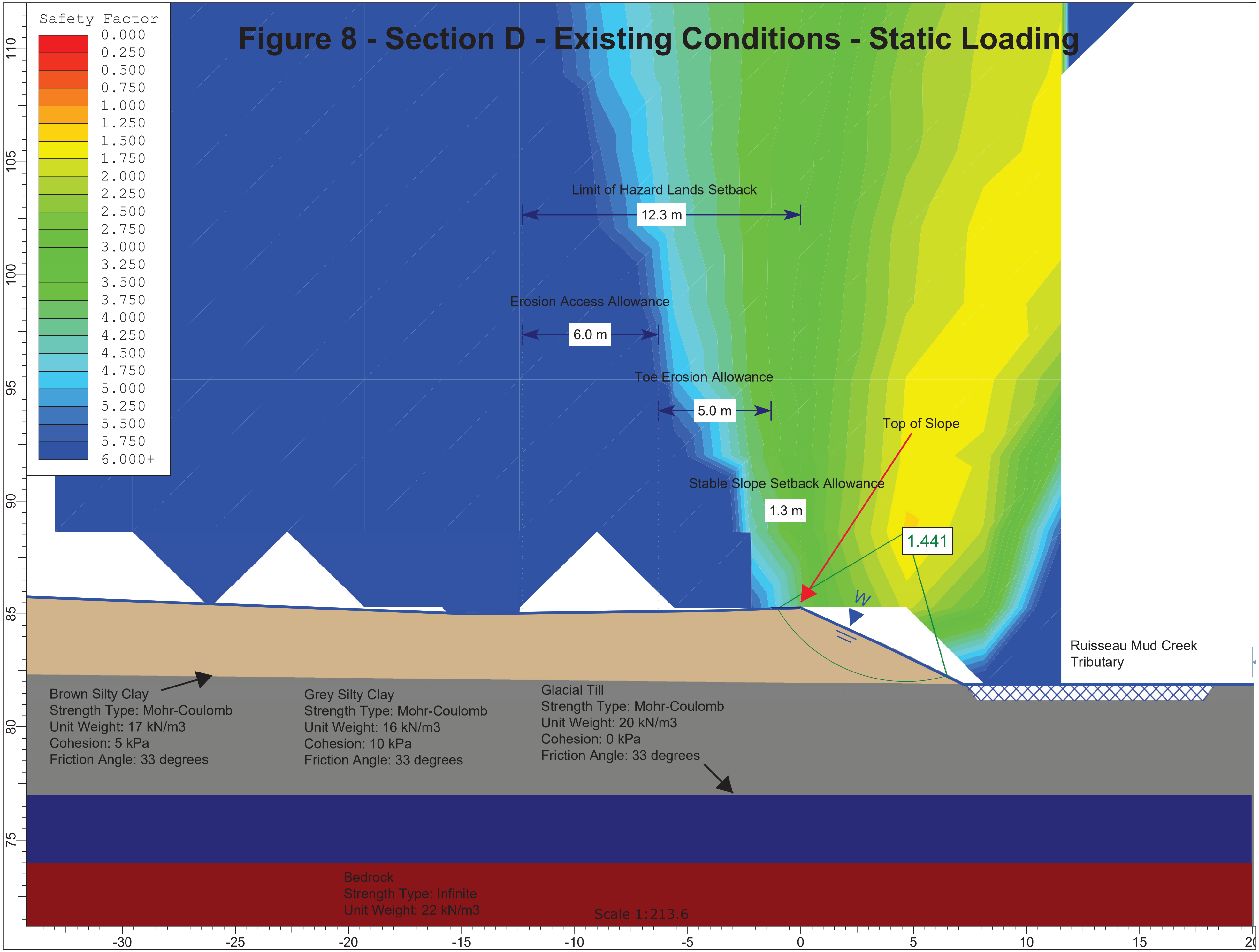


Figure 9 - Section D - Existing Conditions - Seismic Loading

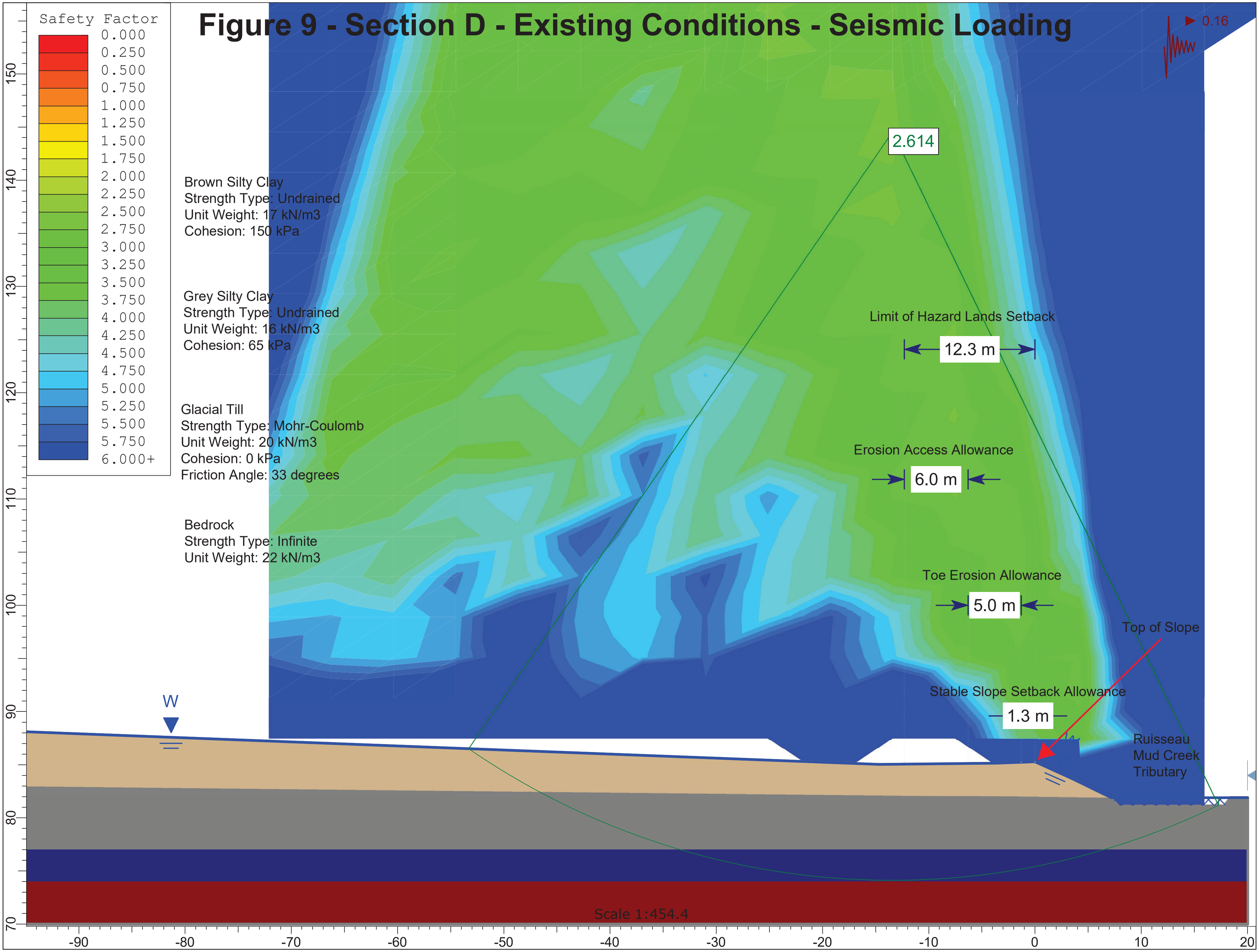


Photo 1: Area located at the bottom of the slope along the south-west portion of the subject site. Area is well vegetated and sloped gradually towards the valley floor.

Photo 2: Area along creek tributary 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 creek tributary appeared to be flowing very slowly and/or ponding.

Photo 3: Area along creek tributary 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 creek tributary 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 creek tributary appeared to be flowing very slowly and/or ponding.

Photo 5: Area along creek tributary 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 creek tributary appeared to be flowing very slowly and/or ponding.

Photo 6: Area along creek tributary 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 creek tributary and west portion of the subject site. Area appeared to be well vegetated with a slightly steeper bank along the tributary 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 creek tributary and west portion of the subject site. Area appeared to be well vegetated with a slightly steeper bank along the tributary 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 creek tributary and north-west portion of the subject site. Area appeared to be well vegetated with a gentle flow throughout the tributary at the time of site visit. Gradual slope observed from subject site to the valley floor.

Photo 10: Area of intersection of tributary along west portion of subject site and Ruisseau Mud Creek. Area of Ruisseau 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 Ruisseau 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 Ruisseau 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 Ruisseau 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.

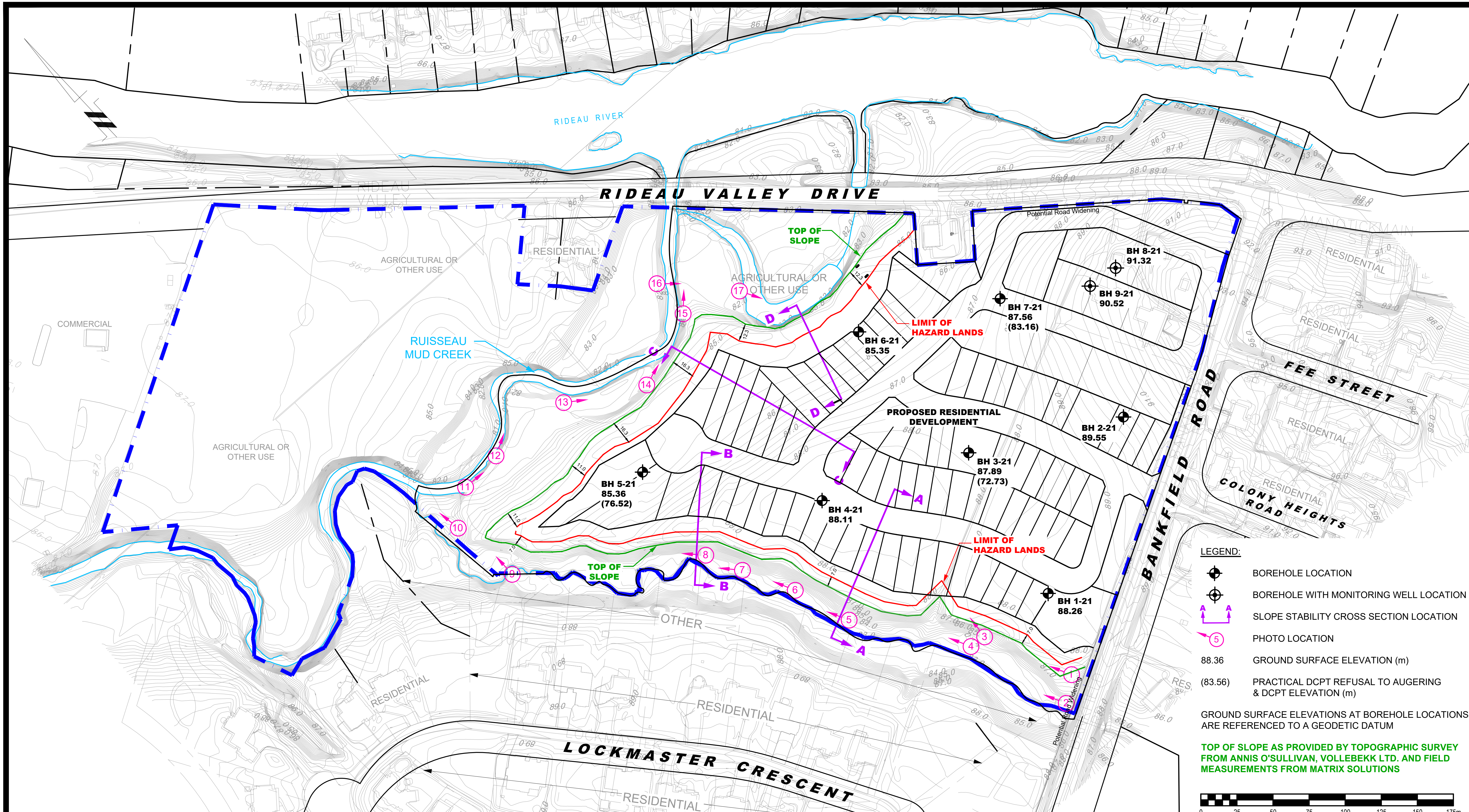
Photo 14: Area of Ruisseau Mud Creek along north-west portion of subject site. Area of valley floor appeared to have well rooted vegetation with relatively steep banks along creek. Some active erosion and fallen trees observed along photographed portion of creek.

Photo 15: Area of Ruisseau 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 Ruisseau 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 Ruisseau 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 Ruisseau 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.



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PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
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

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: MS	Dwg No.: PG5828-1
Approved by: DJG	Revision No.: 2