Geotechnical Engineering

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Materials Testing

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Geotechnical Investigation

Proposed Residential Development
760 River Road
Ottawa, Ontario

Prepared For

Claridge Homes

December 10, 2019

Report PG4728-1
Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed residential development to be located at 760 River Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The investigation objectives were to:

❏ determine the subsurface soil and groundwater conditions by means of boreholes.

❏ provide geotechnical recommendations for 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. This report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Project

It is understood that the proposed residential development will consist of single family residential dwellings and landscaped areas. It is expected that the development will be municipally serviced with local paved roadways.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was conducted on November 1, 2018. At that time, a total of five (5) boreholes were drilled and sampled to a maximum depth of 5.9 m below existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site taking into consideration site features. The locations of the boreholes are shown on Drawing PG4728-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from auger flights or a 50 mm diameter split-spoon sample. The soil samples were classified on site, placed in sealed 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. A dynamic cone penetration test (DCPT) was completed at several boreholes.

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 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

The overburden thickness was also evaluated during the investigation by completing a dynamic cone penetration test (DCPT) at borehole BH 5. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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.

Undrained shear strength tests were completed in cohesive soils with a shear vane apparatus.

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.

Groundwater

Flexible polyethylene standpipes were installed in all boreholes to permit monitoring and sampling of the groundwater subsequent to the completion of the geotechnical drilling program.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one 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 in the field by Paterson personnel in a manner to provide general coverage of the proposed development taking into consideration existing site features. The borehole locations and ground surface elevations at the borehole locations were surveyed by Annis O'Sullivan Vollebekk Ltd and are referenced to a geodetic datum. The locations and ground surface elevations of the boreholes are presented on Drawing PG4728-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the our field investigation were examined in our laboratory. A total of five (5) Atterberg limit tests were completed on selected silty clay samples. Grain Size distribution (hydrometer) testing was also completed on one (1) soil sample and one (1) soil sample was submitted for shrinkage testing. The results are presented in Subsection 4.2 of our current report and in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are discussed in Subsection 6.7 and shown in appendix 1.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is vacant and in the initial grading stages for the proposed residential development. Access roads, deforested areas and fill piles have been observed throughout the site. A sales centre for the future residential development and associated gravel parking lot was noted to be under construction within the southeast corner of the subject site. Prior to any development, the site was predominantly grass and tree covered.

Based on historical aerial photographs, a residential dwelling was identified within the central portion of the subject and was demolished in 2016.

The ground surface is relatively flat within the eastern portion of the subject site and at a slightly lower elevation than River Road and slopes down towards the Rideau River.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of topsoil or fill material comprised of brown silty clay with varying amounts of sand, gravel and organics. The abovenoted layers are underlain by a hard to stiff brown silty clay crust followed by a very stiff to firm grey silty clay deposit. Practical refusal to DCPT was observed at a depth of 13.56 m at BH 5. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Atterberg and Shrinkage Testing Results

The results of Atterberg Limits tests conducted within the silty clay are presented below in Table 1 - Summary of Attergerg Limits' Results and on the Atterberg Limits' Results sheet in Appendix 1. The tested silty clay samples had measured liquid limits of 41 to 53% and plasticity indices ranging from 23 to 34%, which classifies them as inorganic clays of low to high plasticity (CL to CH) in accordance with the Unified Soil Classification System.

The results of the shrinkage testing of BH 5 - SS 2 resulted in a shrinkage limit of 17% with a shrinkage ratio of 1.91.

Table 1 - Summary of Atterberg Limits' Results				
Sample	**Liquid Limit %**	**Plastic Limit %**	**Plasticity Index %**	**Classification**
BH 1 - SS 2	42	17	24	CL
BH 2 - SS 2	41	18	23	CL
BH 3 - SS 2	41	17	23	CL
BH 4 - SS 2	43	17	26	CL
BH 5 - SS 2	53	19	34	CH

Grain Size Distribution and Hydrometer Testing

Two (2) samples were submitted for grain size distribution and hydrometer testing. The results are summarized in Table 2 and presented on the Grain Size Distribution sheets in Appendix 1.

Table 2 - Summary of Grain Size Distribution and Hydrometer Tests				
Sample	**Gravel (%)**	**Sand (%)**	**Fines Content**	
			Silt (%)	**Clay (%)**
BH 1 - SS 2	0	27.8	37.7	34.5
BH 3 - SS 2	0	16.1	40.9	43.0

Bedrock

Based on available geological mapping, bedrock in the northern half of the subject site consists of interbedded sandstone and dolomite of the March formation with an overburden drift thickness of 10 to 25 m depth. Bedrock in the southern half of the subject site consists of dolomite of the Oxford Formation with an overburden drift thickness of 10 to 15 m depth.

4.3 Groundwater

The groundwater levels were measured in the borehole locations on November 13, 2018, and are presented in the Soil Profile and Test Data sheets in Appendix 1. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole, which can lead to higher than typical groundwater levels. The long-term groundwater level can also be estimated based on moisture levels and colouring of the recovered soil samples. Based on these observations at the borehole locations, the long-term groundwater level is expected at a 4 to 5 m depth.

Table 3 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	88.09	0.48	87.61	November 13, 2018
BH 2	87.63	Damaged	n/a	November 13, 2018
BH 3	87.55	1.30	86.25	November 13, 2018
BH 4	87.65	0.90	86.75	November 13, 2018
BH 5	87.01	0.94	86.07	November 13, 2018

Note:
-The ground surface elevations at the borehole locations were provided by Annis O’Sullivan Vollebekk Ltd.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is expected that the proposed buildings will be founded over conventional shallow footings placed on an undisturbed, hard to very stiff silty clay or an engineered fill bearing surface.

Due to the presence of the silty clay deposit, the subject site will be subjected to a permissible grade raise.

A construction setback defined as the Limit of Hazard Lands has been defined along the existing slope which runs approximately north-south along the western portion of the site. This is presented on Drawing PG4728-1 - Test Hole Location Plan and is discussed further in Section 6.8.

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

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 construction debris should be entirely removed from within the proposed 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 used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

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

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed on an undisturbed, hard to very stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

Footings placed on engineered fill bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa.**

Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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

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

Permissible Grade Raise Recommendations

A permissible grade raise restriction has been determined for the subject site based on the undrained shear strength values completed within the silty clay deposit. Based on the testing results, a permissible grade raise restriction of **3 m** above existing ground surface is recommended for the subject site.

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. A higher seismic site class, such as Class C, may be applicable for the subject site. However, the higher seismic site class would have to be confirmed by a site-specific seismic shear wave velocity test. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Wall

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

Two distinct conditions, static and seismic, should be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

5.6 Basement Slab

With the removal of all topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone for a basement slab. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of the SPMDD.

5.7 Pavement Structure

Car only parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 4 and 5.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 5 - Recommended Pavement Structure - Local Roadways

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to 98% of the SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping 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 its load carrying capacity.

Due to the impervious nature 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 the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Perimeter Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 150 mm in diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer. Ã

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 Miradrain G100N or 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 recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

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

6.3 Excavation Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

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, subbedding 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

Groundwater Control for Building Construction

Due to existing groundwater level and inferred depths of the proposed footings, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

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, 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.

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.

6.6 Winter Construction

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

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

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

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 non aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Restrictions

The proposed development is located in an area of medium sensitive silty clay deposits for tree planting. Tree planting for this subject development should be limited to low water demand trees. The minimum permissible distance from the foundation will depend on the nature of the tree, the depth of the clay crust and the final grade raise in relation to the permissible grade raise. A minimum permissible distance of 4.5 m from the foundation wall is recommended for a tree planting.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

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 neighbouring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Installation of Decks or Additions

If consideration is given to construction of a deck or addition, a geotechnical consultant should be retained by the homeowner to review the site conditions. Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

6.9 Slope Stability Assessment

The slope conditions were reviewed by Paterson field personnel on November 26, 2019 as part of the slope stability assessment. Three slope cross-sections were studied as the worst case scenarios. The cross section locations are presented on Drawing PG4728-1 - Test Hole Location Plan attached to the current report.

The existing slope extending down to the Rideau River generally has a height of 2 to 4 m with an incline of approximately 3H:1V. This slope is generally vegetated with trees.

The ravine located in the southeastern and southern portion of the site has a height of approximately 2 to 3 m near River Road, increasing to heights of up to 8 m as it approaches the Rideau River to the west. The slopes in the vicinity of the ravine are generally vegetated with small brush and trees. No watercourse is present within the ravine.

A slope stability analysis was carried out to determine the required construction setback from the top of the bank based on a factor of safety of 1.5. Erosional and access allowances were also considered in the determination of limits of hazard lands and are discussed in the following sections. The proposed limit of hazard lands and top of bank are shown on Drawing PG4728-1 - Test Hole Location Plan attached to the current report.

Slope Stability Assessment

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.

A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

The cross-sections were analyzed based on the existing conditions observed during our site visit and review of the available topographic mapping. The slope stability analysis was completed at each slope cross-section under worst-case-scenario by assigning cohesive soils under fully saturated groundwater conditions. Subsoil conditions at the cross-sections were inferred based on nearby boreholes and general knowledge of the area's geology.

Table 6 - Effective Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay Crust	17	33	7
Grey Silty Clay	16	33	10

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the open boreholes completed at the time of our geotechnical investigation and based on our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 7 below.

Table 7 - Total Stress Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
Brown Silty Clay Crust	17	-	80
Grey Silty Clay	16	-	70 to 30

Static Loading Analysis

The results for the slope stability analyses under static conditions at Sections A, B and C are shown on Figures 2, 4, and 6 attached to the present report. The factor of safety was found to be greater than 1.5 at the three (3) cross-section analyzed. Therefore, when considering static conditions, no stable slope allowance is required from the top of the slope in order to achieve a factor of safety of 1.5 for the limit of the hazard lands.

Seismic Loading Analysis

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g 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 slope stability analyses under seismic conditions are shown on Figures 3, 5, and 7 in Appendix 2. The results indicate that the factors of safety are greater than 1.1 under seismic conditions. Based on these results, the slopes are considered to be stable under seismic loading. Therefore, when considering seismic loading, no stable slope allowance is required from the top of the slope to achieve a factor of safety of 1.1 for the limit of the hazard lands.

Geotechnical Setback - Limit of Hazard Lands

The toe erosion allowance for the slope along the Rideau River (Sections A and B) are based on the cohesive nature of the soils, the observed current erosional activities and the width and location of the current watercourses. Signs of erosion were noted in areas where the existing watercourses have meandered in close proximity to the toe of the slope. It is considered that a toe erosion allowance of 5 m in addition to an erosion access allowance of 6 m is appropriate for the slope along the Rideau River, which should be applied from the top of slope.

The limit of hazard lands, which include these allowances, are indicated on Drawing PG4728-1 - Test Hole Location Plan attached to the present report.

For the ravine in the southeastern and southern portions of the site (Section C), given that only periodic flow is present after significant storm events and that no signs of active erosion were observed, an erosion access allowance is not considered to be required for these slopes. Further, given that there is no stable slope allowance along the ravine, hazard lands are not present in the vicinity of the ravine.

The existing vegetation on the slope face should not be removed as it contributes to the stability of the slope and reduces erosion. If the existing vegetation needs to be removed, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed be placed across the exposed slope face. The use of an erosion control blanket, may be necessary to minimize rill-type erosion until the vegetation takes root.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects be performed by the geotechnical consultant.

- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and granular fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling.
- ❏ Field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request 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 Claridge Homes 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.

Scott Dennis

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Dec. 10, 2019
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Report Distribution:

☐ Claridge Homes (3 copies)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMIT TESTING RESULTS

GRAIN SIZE DISTRIBUTION RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 760 River Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

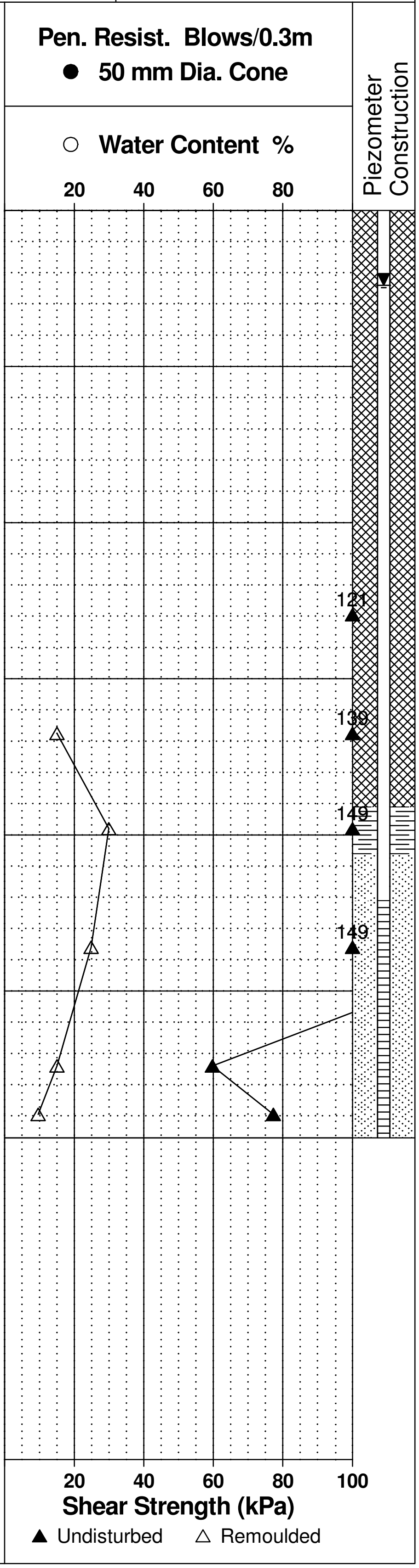
DATE November 1, 2018

FILE NO. PG4728

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty clay with sand and gravel 0.60		AU	1		
Very stiff to stiff, brown **SILTY CLAY**		SS	2	62	6
		SS	3	96	5
		SS	4	96	
		SS	5	50	
- grey by 5.5m depth 5.94					
End of Borehole (GWL @ 0.48m - Nov. 13, 2018)					

DEPTH (m)	ELEV. (m)
0	88.09
1	87.09
2	86.09
3	85.09
4	84.09
5	83.09



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 760 River Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

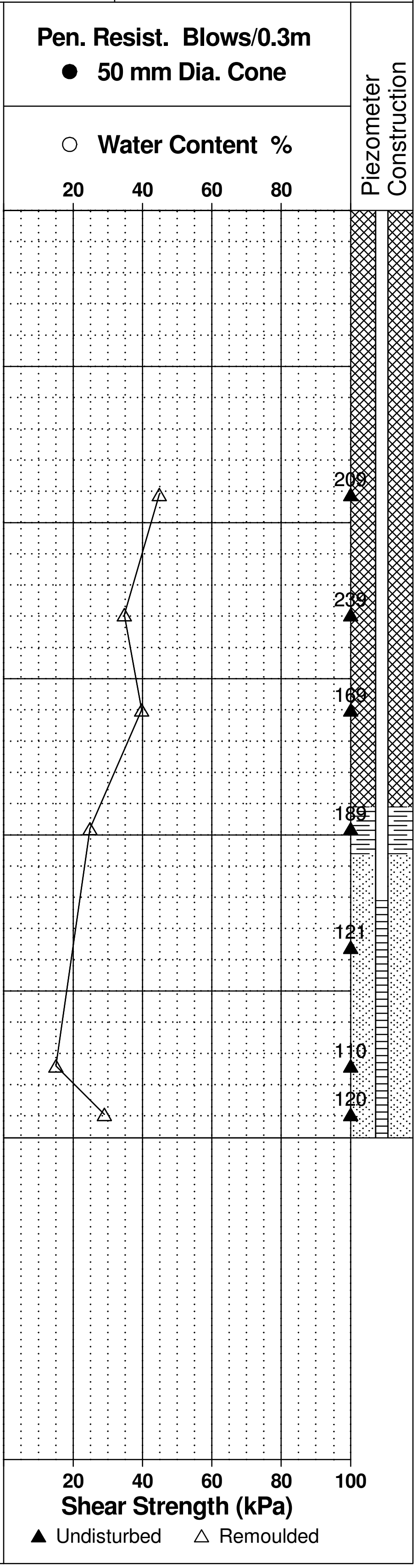
BORINGS BY CME 55 Power Auger

DATE November 1, 2018

FILE NO. PG4728

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.63
FILL: Brown silty clay, trace gravel		AU	1				
0.60							
Hard to very stiff, brown **SILTY CLAY**		SS	2	96	4	1	86.63
		SS	3	96		2	85.63
		SS	4	96		3	84.63
						4	83.63
						5	82.63
- grey by 5.9m depth							
5.94							
End of Borehole							
(Piezometer damaged - Nov. 13, 2018)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 760 River Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. **PG4728**

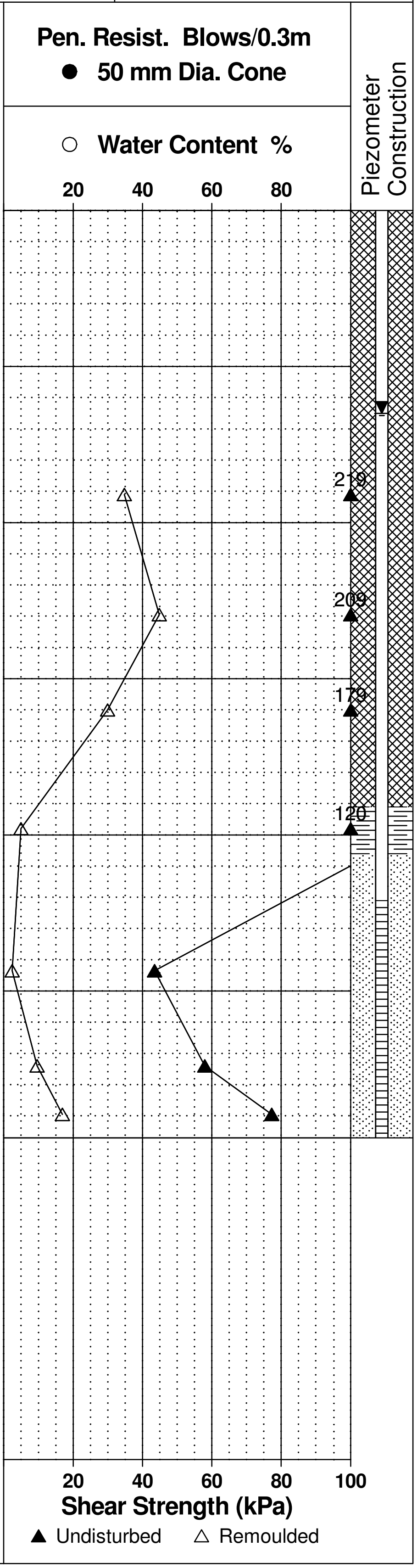
REMARKS

HOLE NO. **BH 3**

BORINGS BY CME 55 Power Auger

DATE November 1, 2018

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	87.55
FILL: Brown silty clay with organics	AU	1				
0.60						
	SS	2	96	4	1	86.55
	SS	3	96		2	85.55
Hard to very stiff, brown **SILTY CLAY**	SS	4				
					3	84.55
					4	83.55
- firm to stiff and grey by 4.6m depth	SS	5	88		5	82.55
5.94						
End of Borehole						
(GWL @ 1.30m - Nov. 13, 2018)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 760 River Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4728

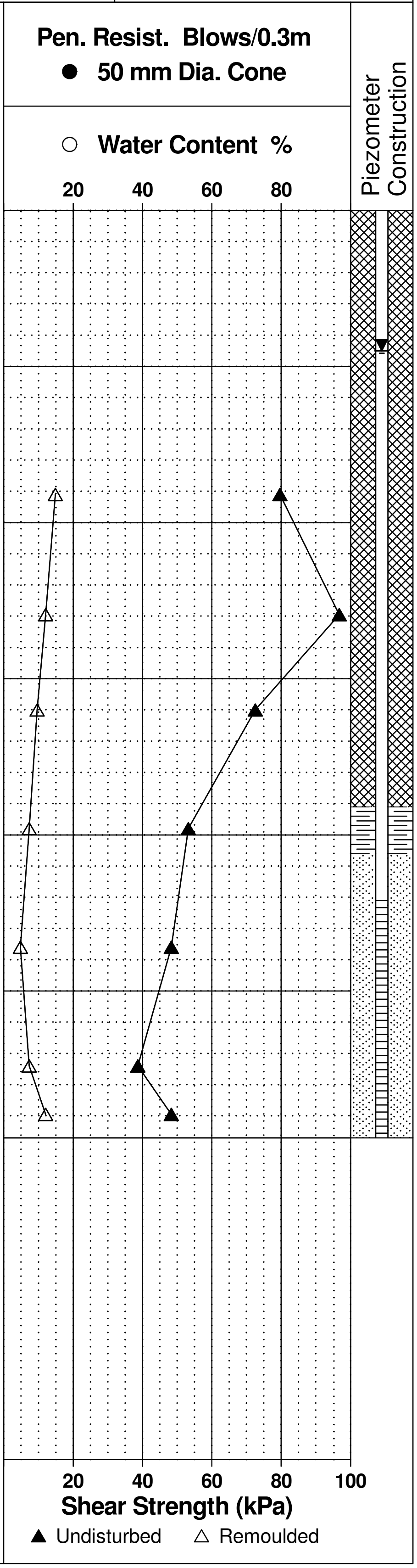
REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger

DATE November 1, 2018

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	87.65
FILL: Brown silty clay, trace organics	AU	1				
0.60						
	SS	2	88	4	1	86.65
Stiff to firm, brown **SILTY CLAY**	SS	3	79		2	85.65
	SS	4	96			
- grey by 3.0m depth					3	84.65
					4	83.65
					5	82.65
5.94						
End of Borehole (GWL @ 0.90m - Nov. 13, 2018)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 760 River Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

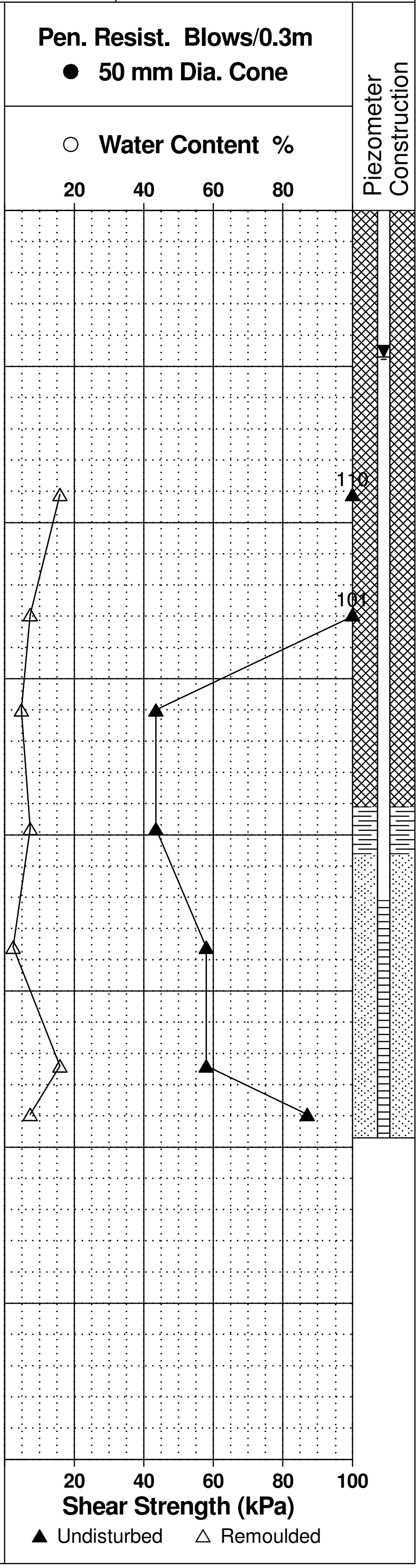
BORINGS BY CME 55 Power Auger

DATE November 1, 2018

FILE NO. PG4728

HOLE NO. BH 5

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	87.01
TOPSOIL, trace gravel 0.30	AU	1				
Very stiff to stiff, brown **SILTY CLAY**	SS	2	71	4	1	86.01
	SS	3	67		2	85.01
	SS	4	96			
- firm to stiff, and grey by 3.0m depth					3	84.01
					4	83.01
					5	82.01
5.94						
Dynamic Cone Penetration Test commenced at 5.94m depth. Cone pushed to 11.6m depth.					6	81.01
					7	80.01
					8	79.01



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 760 River Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4728

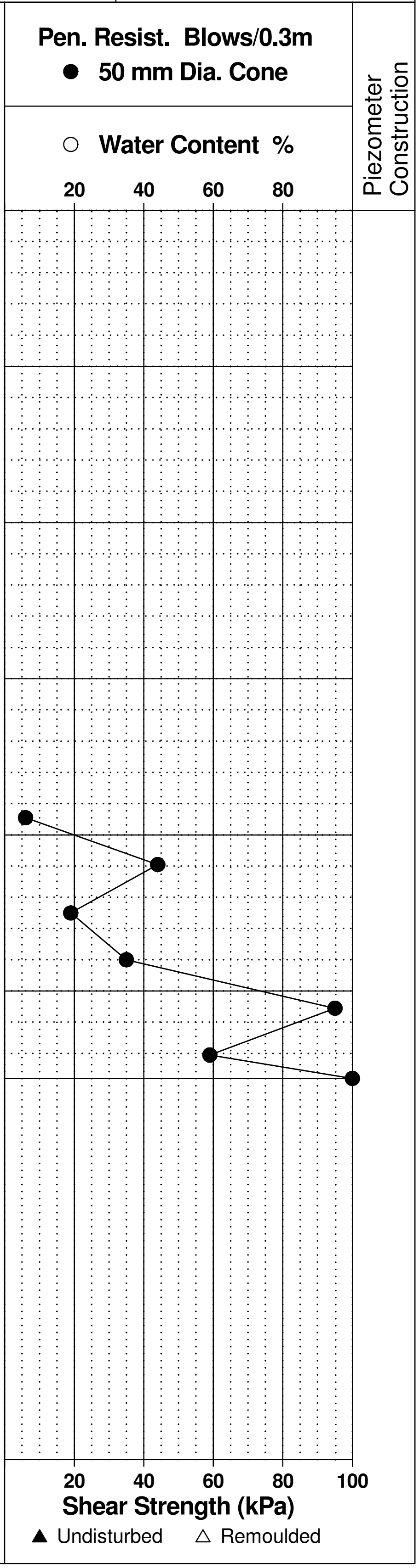
REMARKS

HOLE NO. BH 5

BORINGS BY CME 55 Power Auger

DATE November 1, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						8	79.01
						9	78.01
						10	77.01
						11	76.01
						12	75.01
						13	74.01
13.56 End of Borehole Practical DCPT refusal at 13.56m depth (GWL @ 0.94m - Nov. 13, 2018)							



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 = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

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

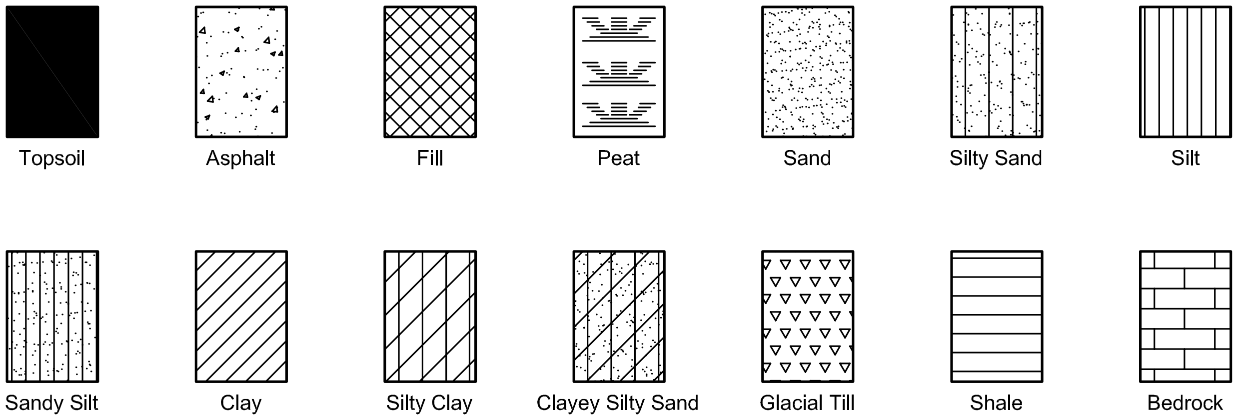
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

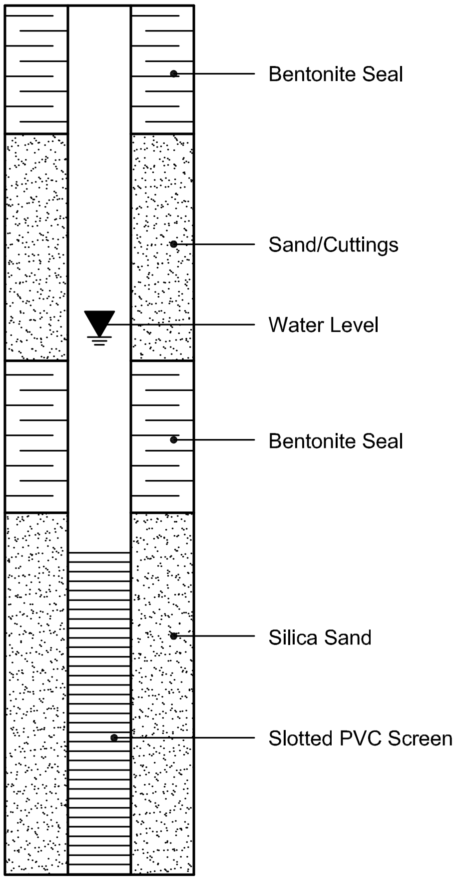
SYMBOLS AND TERMS (continued)

STRATA PLOT

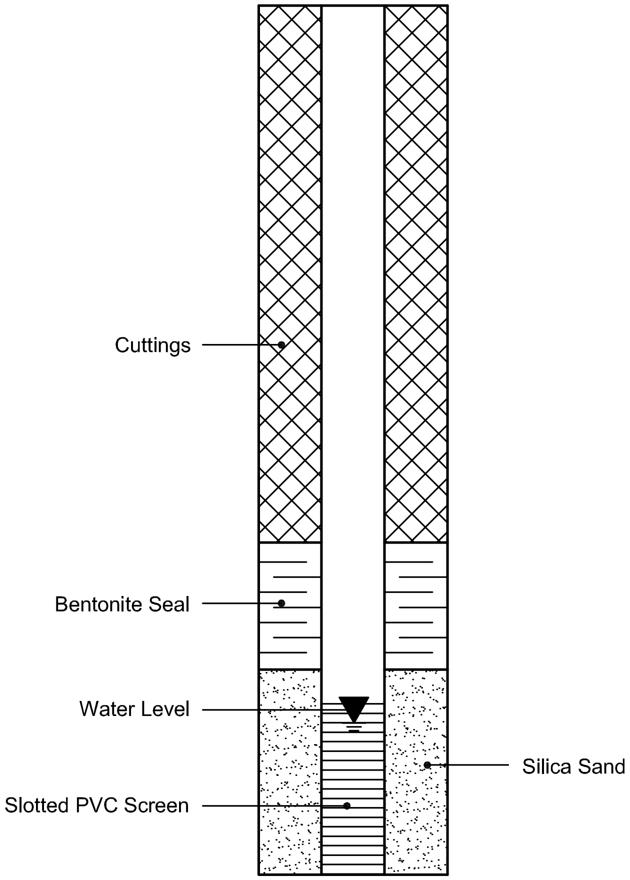


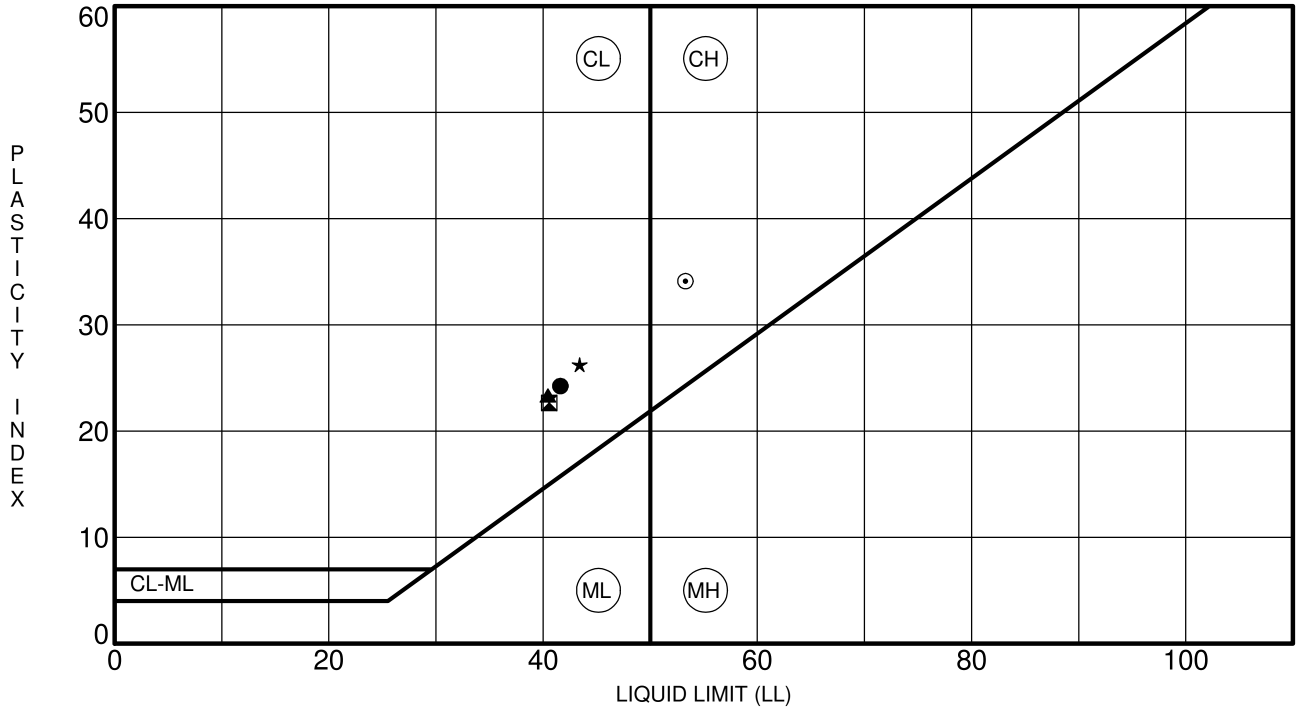
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1**	**SS 2**	**42**	**17**	**24**		**CL - Inorganic clays of low plasticity**
⧗	**BH 2**	**SS 2**	**41**	**18**	**23**		**CL - Inorganic clays of low plasticity**
▲	**BH 3**	**SS 2**	**40**	**17**	**23**		**CL - Inorganic clays of low plasticity**
★	**BH 4**	**SS 2**	**43**	**17**	**26**		**CL - Inorganic clays of low plasticity**
⊙	**BH 5**	**SS 2**	**53**	**19**	**34**		**CH - Inorganic clays of high plasticity**

CLIENT **Claridge Homes**

PROJECT **Geotechnical Investigation - Prop. Residential Development - 760 River Road**

FILE NO. **PG4728**

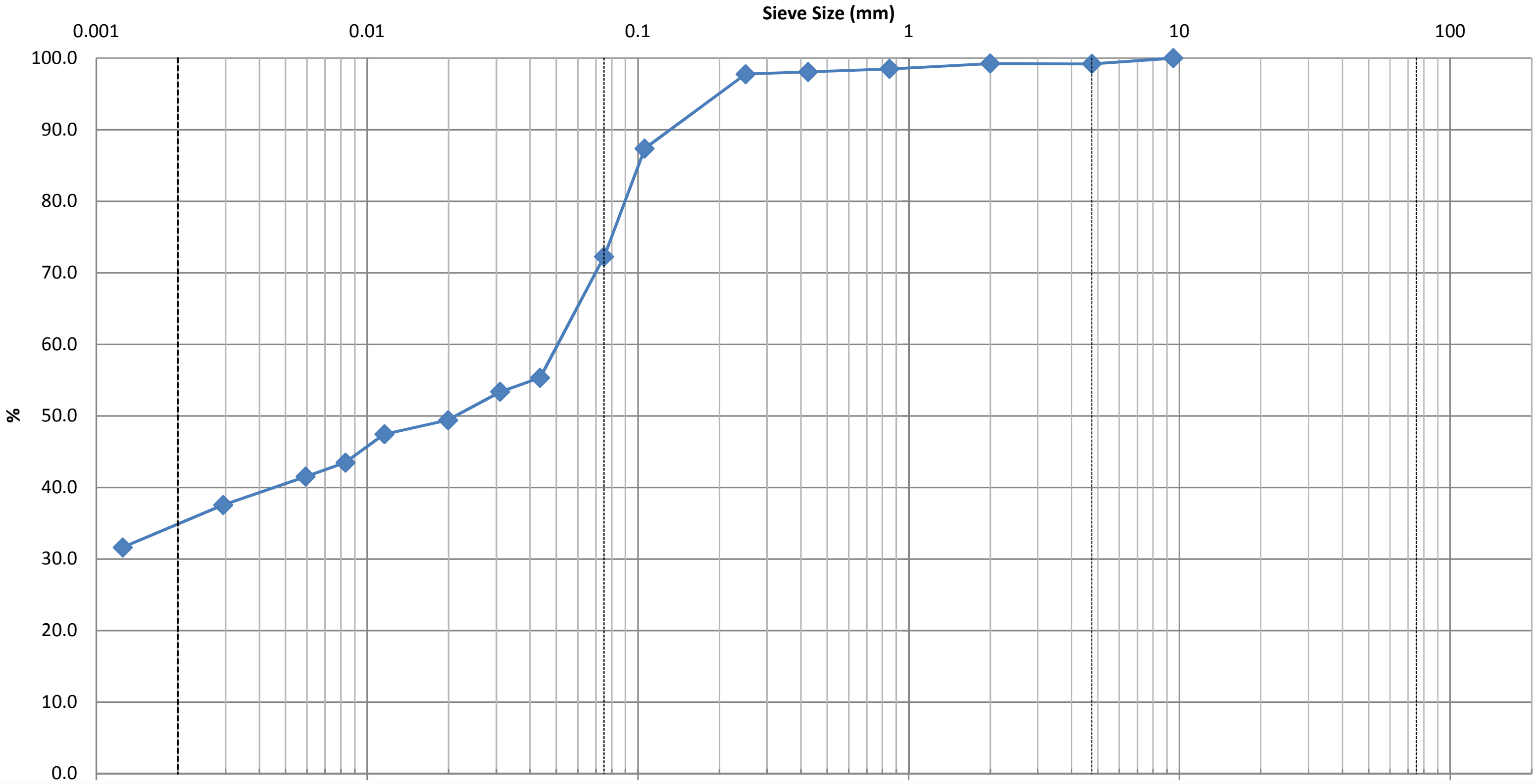
DATE **1 Nov 18**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

CLIENT:	Claridge Homes	DEPTH:	2' 6" - 4' 6"	FILE NO:	PG4728
CONTRACT NO.:		BH OR TP No.:	BH1	LAB NO:	06249
PROJECT:	760 River Road			DATE RECEIVED:	15-Nov-18
				DATE TESTED:	19-Nov-18
DATE SAMPLED:	15-Nov-18			DATE REPORTED:	22-Nov-18
SAMPLED BY:	N. Zulinski			TESTED BY:	D. Bertrand



	Clay	Silt	Sand			Gravel		Cobble
			Fine	Medium	Coarse	Fine	Coarse	

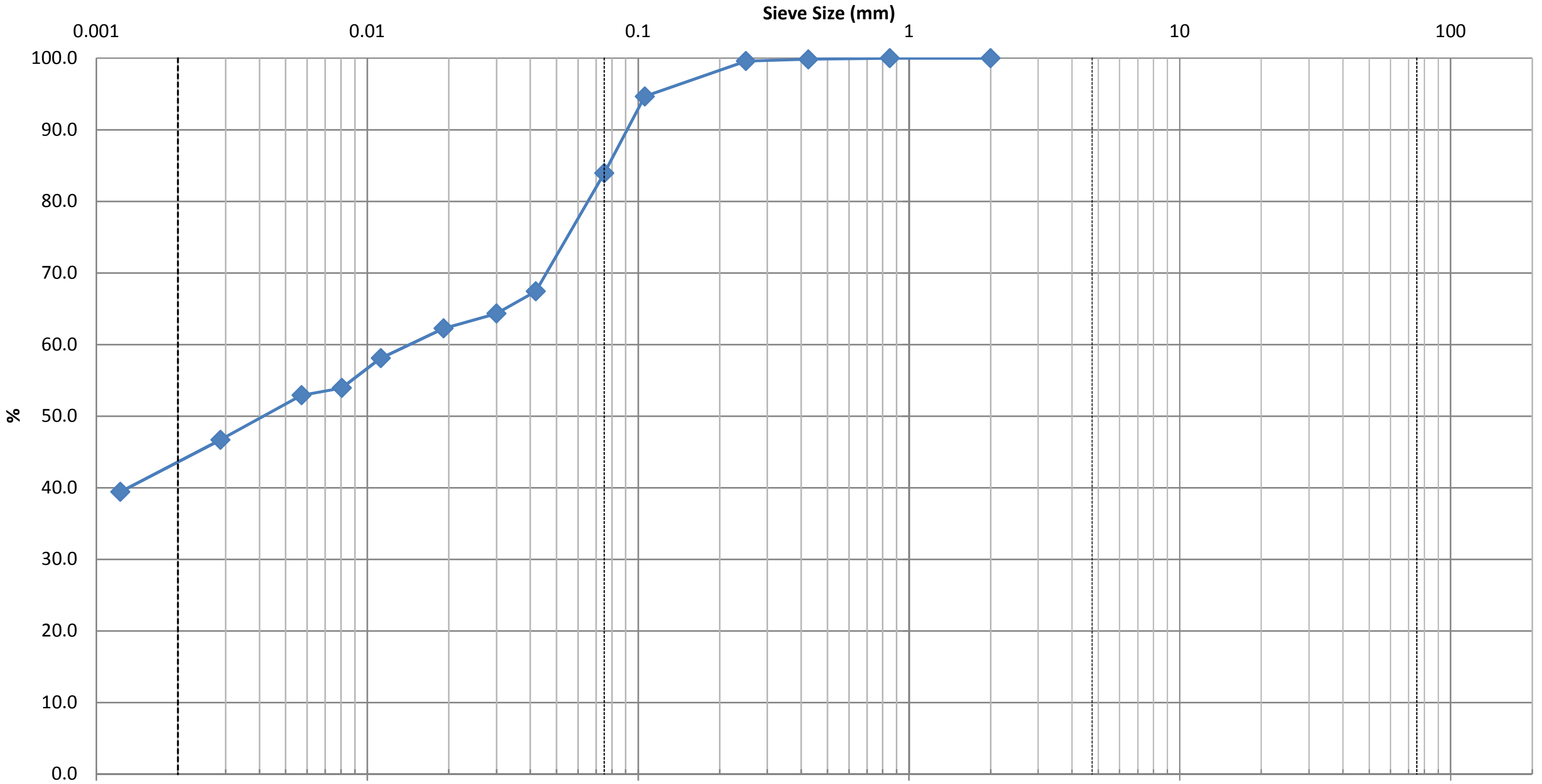
Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		19.6					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				0.0	27.8	37.7	34.5

Comments	

patersongroup
consulting engineers

CLIENT:	Claridge Homes	DEPTH:	2' 6" - 4' 6"	FILE NO:	PG4728
CONTRACT NO.:		BH OR TP No.:	BH3	LAB NO:	06250
PROJECT:	760 River Road			DATE RECEIVED:	15-Nov-18
				DATE TESTED:	19-Nov-18
DATE SAMPLED:	15-Nov-18			DATE REPORTED:	22-Nov-18
SAMPLED BY:	N. Zulinski			TESTED BY:	D. Bertrand



	Clay	Silt	Sand			Gravel		Cobble
			Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					23.9					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)
					0.0	16.1		40.9		43.0

Comments	

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 25232

Report Date: 09-Nov-2018

Order Date: 5-Nov-2018

Project Description: PG4728

	Client ID:	BH5-SS3	-	-	-
	Sample Date:	11/05/2018 09:00	-	-	-
	Sample ID:	1845147-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.29	-	-	-
Resistivity	0.10 Ohm.m	49.2	-	-	-
Anions					
Chloride	5 ug/g dry	54	-	-	-
Sulphate	5 ug/g dry	18	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 7 - SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG4728-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

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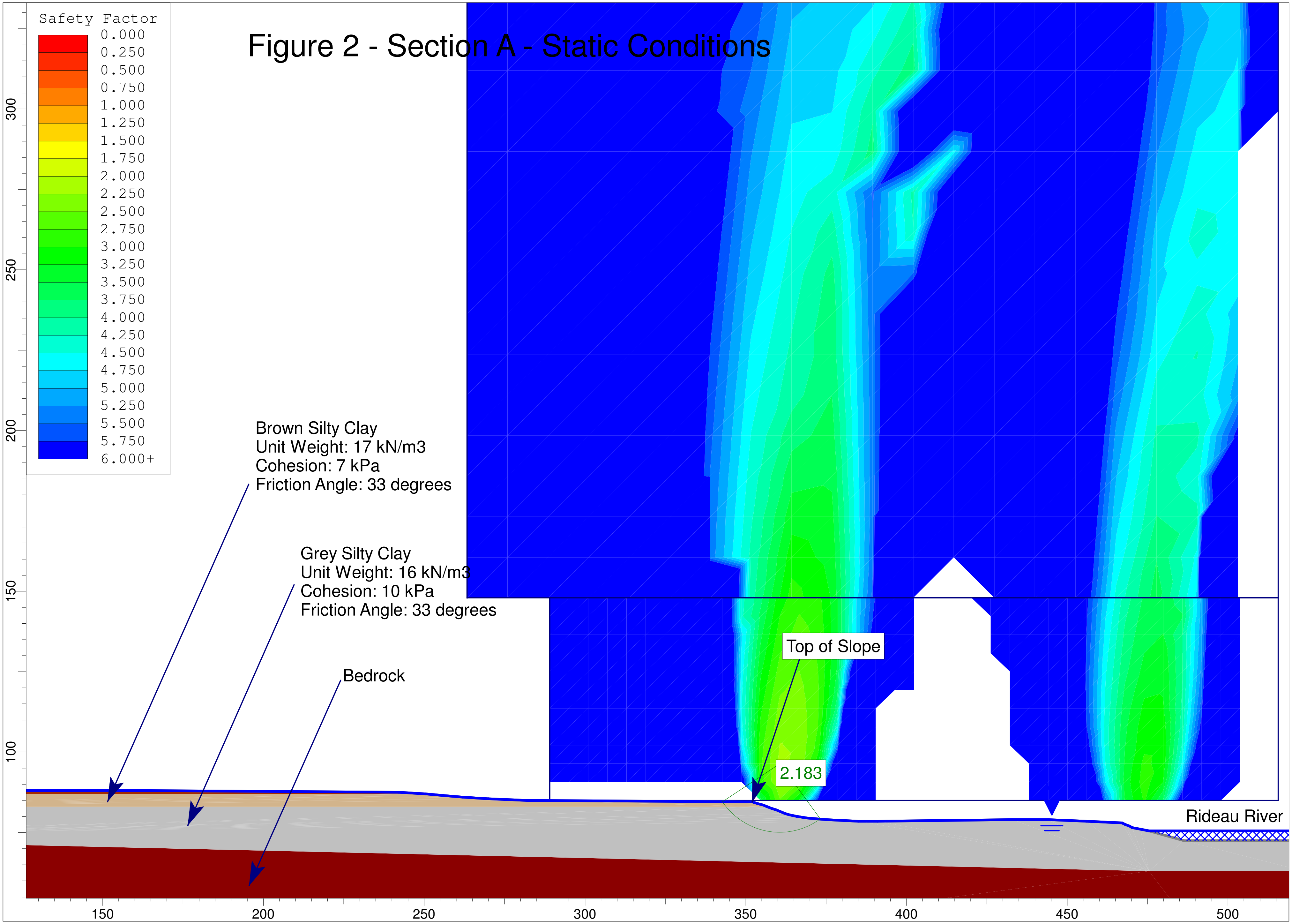
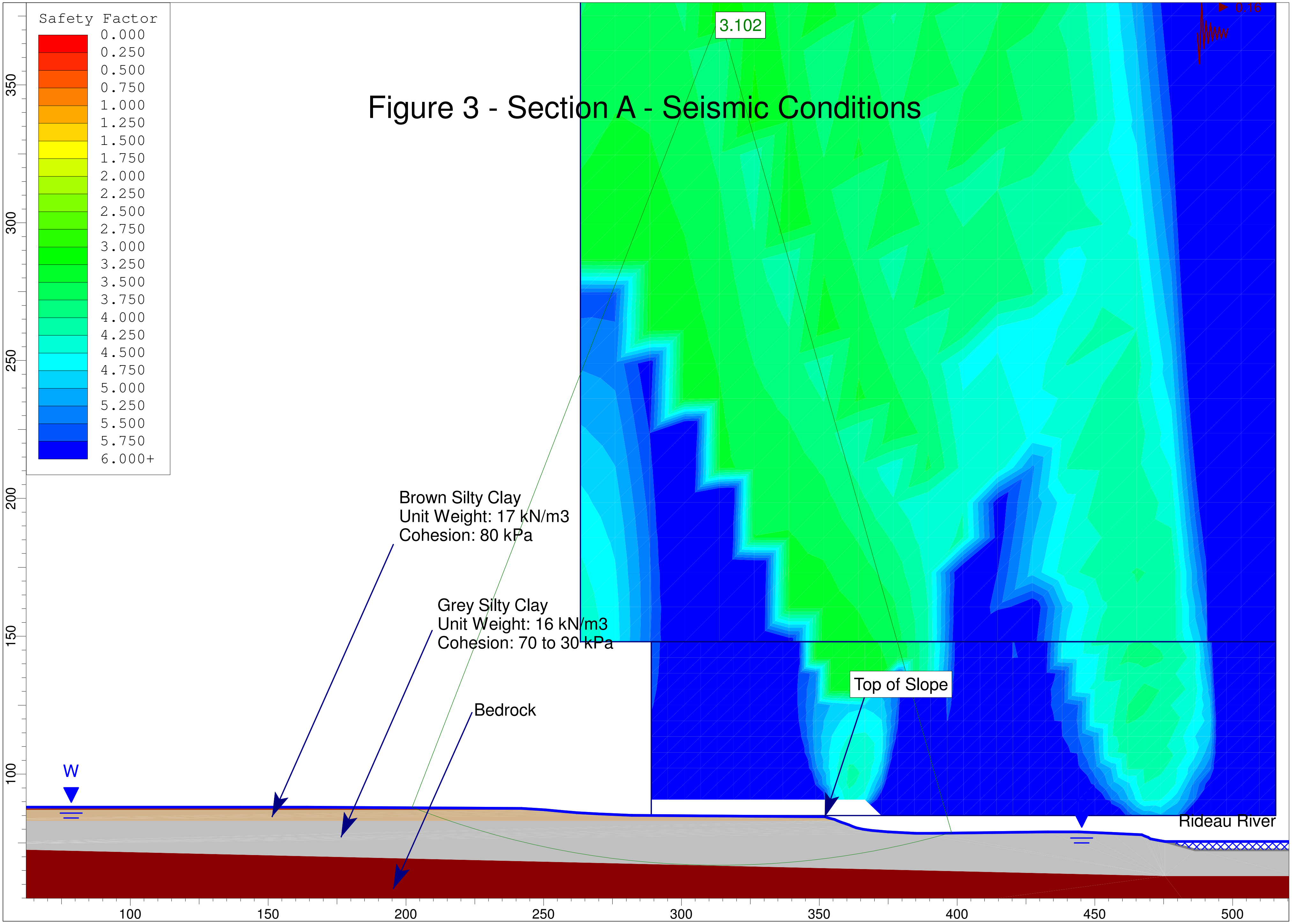


Figure 2 - Section A - Static Conditions

Figure 3 - Section A - Seismic Conditions



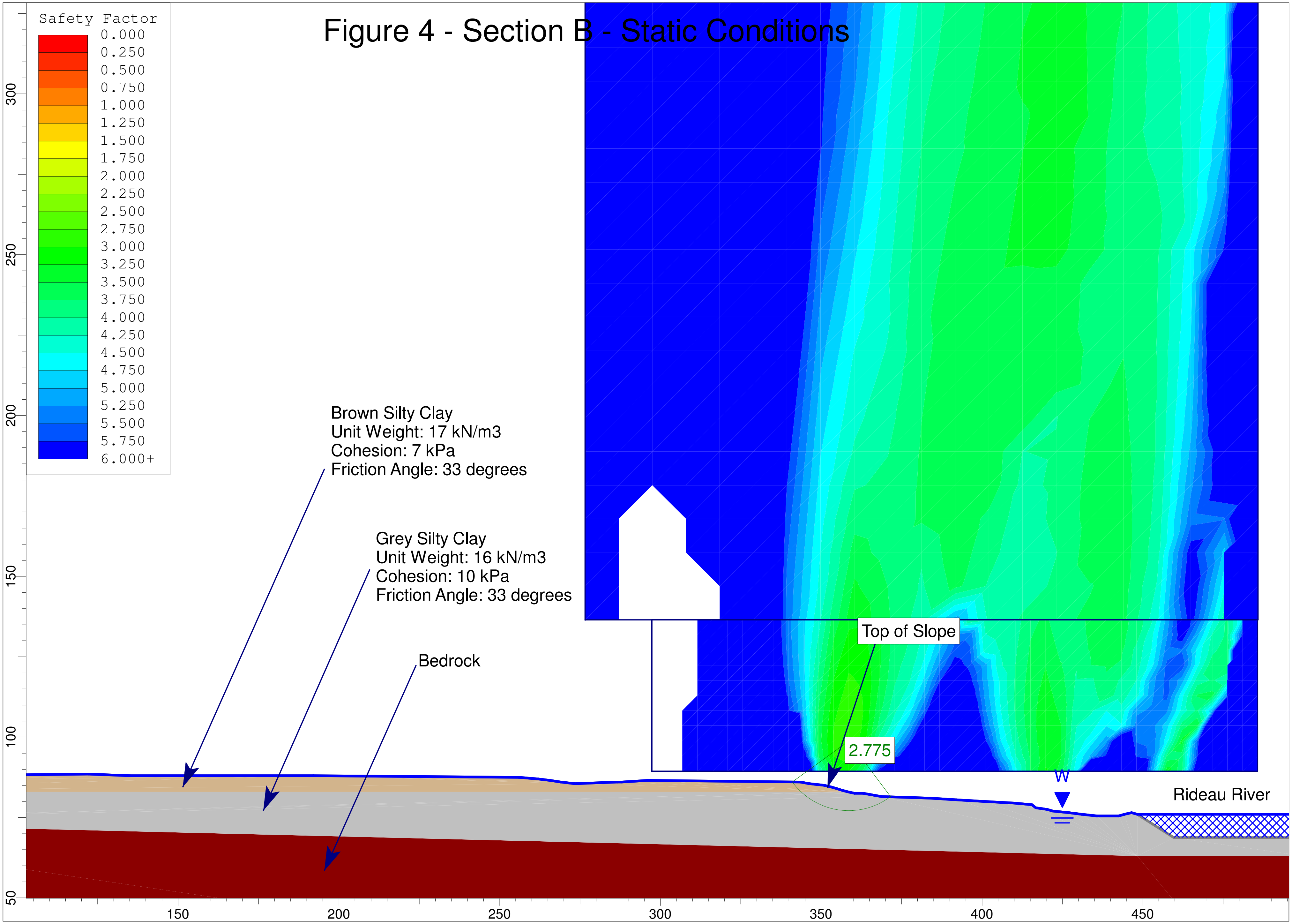


Figure 4 - Section B - Static Conditions

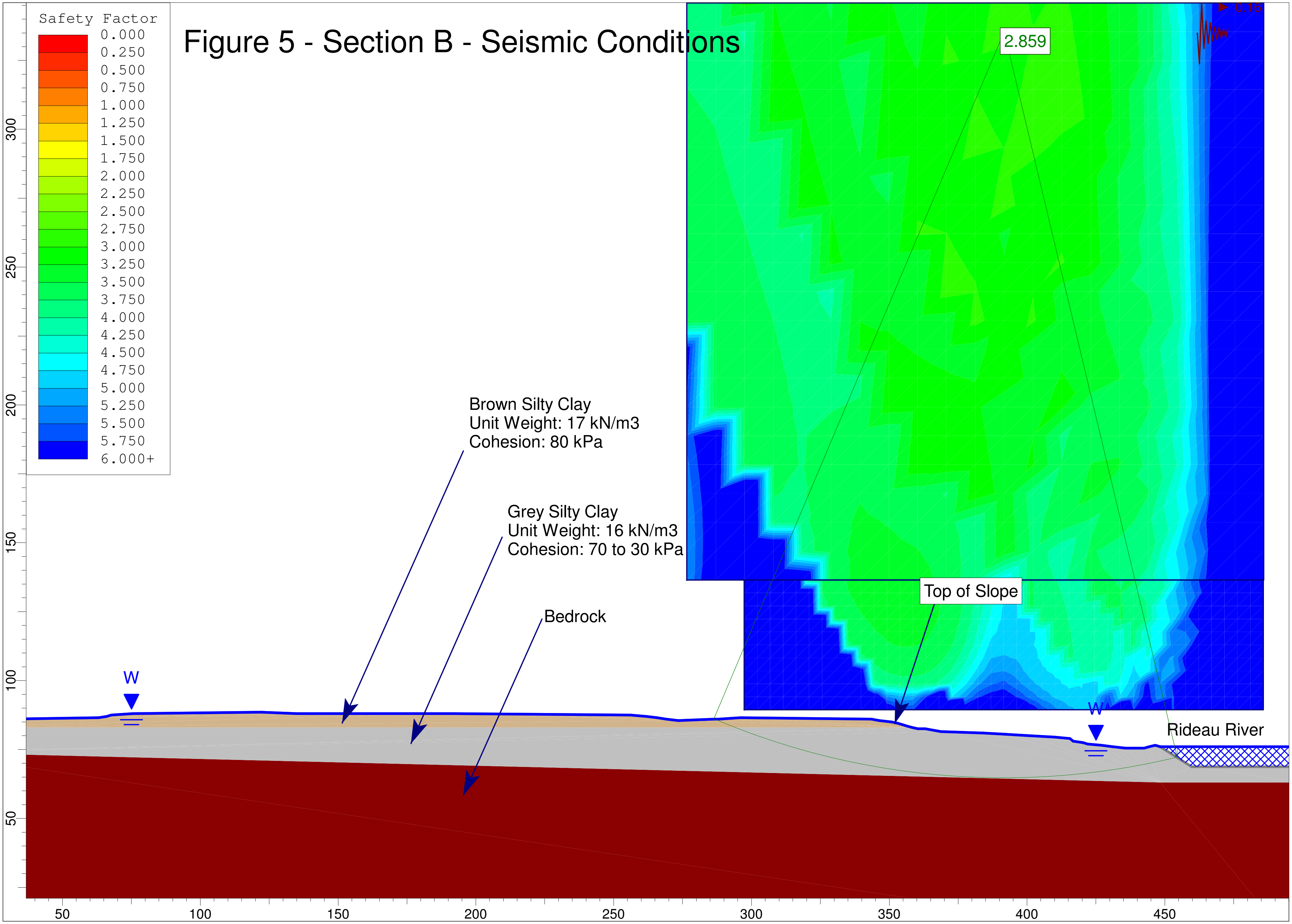
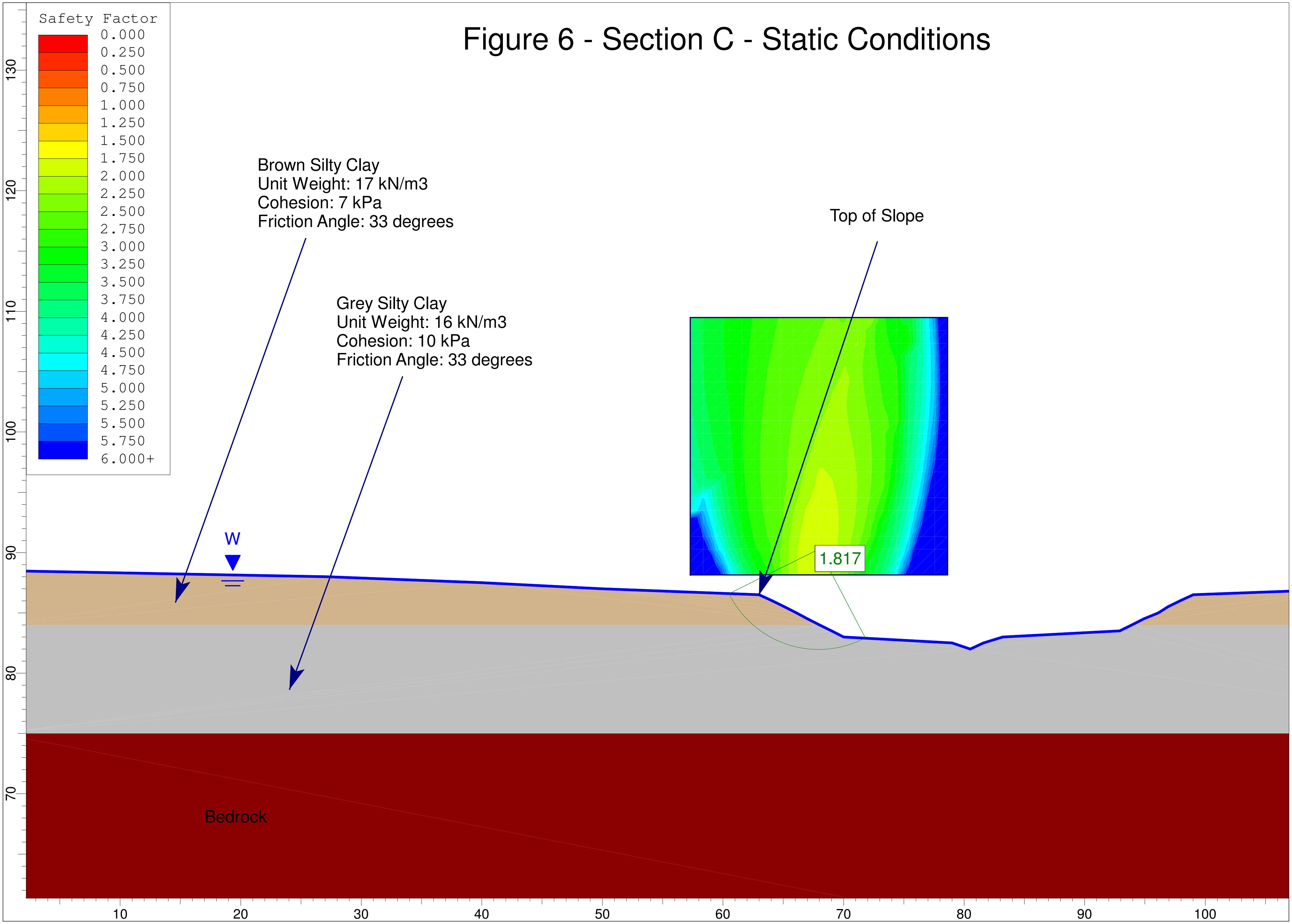


Figure 5 - Section B - Seismic Conditions

Figure 6 - Section C - Static Conditions



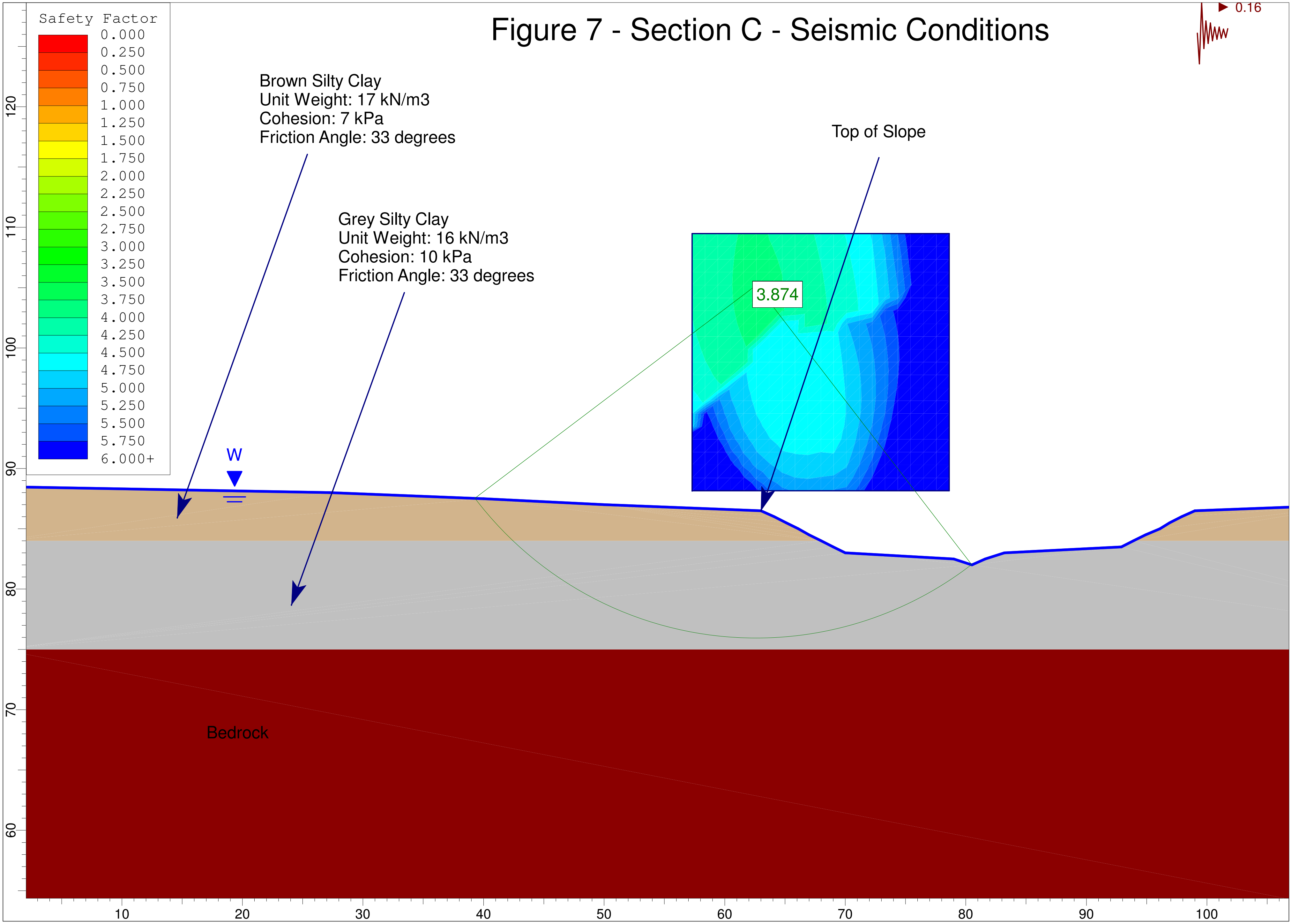
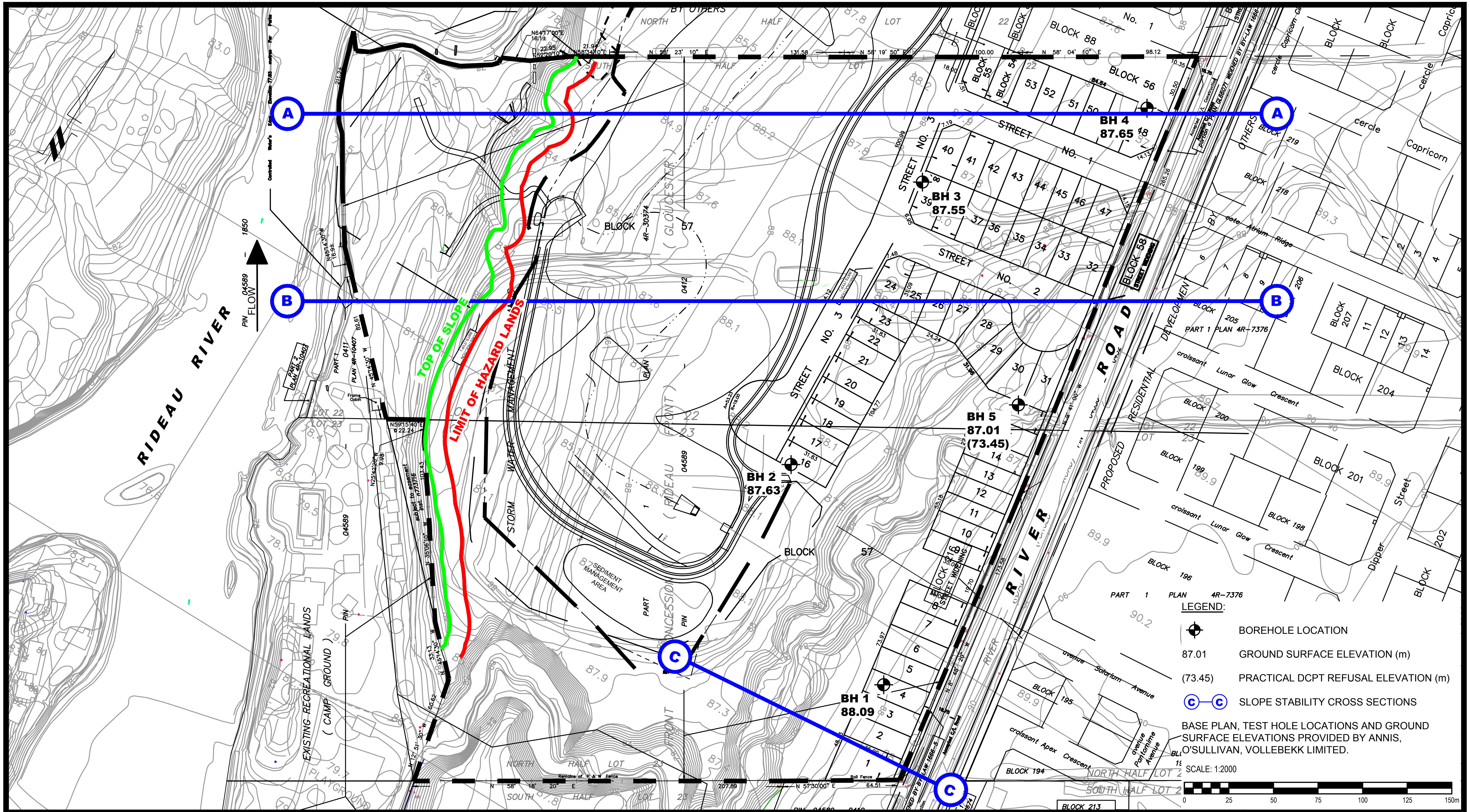


Figure 7 - Section C - Seismic Conditions



patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED NEW BASE PLAN, LIMIT OF HAZARD LANDS, TOP OF SLOPE AND SLOPE CROSS SECTIONS	13/11/2019	SD

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - 760 RIVER ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:2000	Date: 11/2018
Drawn by: MPG	Report No.: PG4728-1
Checked by: NZ	Dwg. No.: PG4728-1
Approved by: DJG	Revision No.: 1

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