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Geotechnical Investigation

Proposed Residential Development
Riverside South - Phase 12
750 River Road
Ottawa, Ontario

Prepared For

Riverside South Development Corporation

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Report: PG3320-3

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Riverside South Development Corporation to conduct a geotechnical investigation for Phase 12 of the Riverside South residential development to be located at 750 River Road in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

- ❏ determine the subsurface soil and groundwater conditions by means of test holes.
- ❏ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

2.0 Proposed Development

It is anticipated that the current phase of the proposed development will consist of single and townhouse style residential dwellings with basement or slab-on-grade construction, attached garages, associated driveways, local roadways and landscaped areas. It is further anticipated that the site will be serviced by future municipal services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was conducted on October 26, 2020 and consisted of 4 test pits advanced to a maximum depth of 3.3 m below the existing ground surface and using the existing test hole information completed by others. The test hole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the proposed residential development taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG3320-1 - Test Hole Location Plan included in Appendix 2.

All test holes were advanced using a rubber tire backhoe. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of excavating to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the sidewalls of the test pits. All soil samples were visually inspected and classified on site. The soil samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the soil samples were recovered from the test pits are shown as G on the Soil Profile and Test Data Sheets in Appendix 1.

Undrained shear strength testing was conducted at regular intervals in cohesive soils and completed using a field vane apparatus.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Where present, the depth at which groundwater was encountered at the completion of excavation was noted in the field.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the current phase of the residential development taking into consideration existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson. The ground surface elevations at the borehole locations were referenced to a geodetic datum. The test hole locations are presented on Drawing PG3320-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Gradation and Atterberg Limits testing were also completed on select samples obtained from the geotechnical investigations. The results of this testing are provided in Section 4.2.

3.4 Analytical Testing

Soil samples was submitted for analytical testing by others to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The majority of the subject site consists of former agricultural lands and has been cleared of trees. A ravine which runs east to west from River road to the Rideau River is located along the northern limit of the property. The central portion of the subject site is currently occupied by an existing stormwater management pond with outlet structures located along the western limits of the subject site.

Based on available aerial photos, 2 residential dwellings were identified within the central portion of the site as recently as 1976 and were no longer present in 1999. An access road which ran in an east to west direction through the eastern half of the subject site connected the dwellings to River Road.

The site is bordered to the north by residential properties and densely treed areas, to the east by River Road, to the south by the stormwater management pond and former agricultural lands, and to the west by the Rideau River. Existing ground surface across the site generally slopes downward from east to west, towards the Rideau River, with geodetic elevation 88 to 76 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of a topsoil layer underlain by a firm to very stiff brown silty clay crust. The silty clay was generally observed to consist of a firm to very stiff, brown silty clay crust becoming a firm to stiff, grey silty clay at depths ranging from 3.1 to 6.1 m below the existing ground surface.

A 2.2 m thick fill layer was encountered underlying the topsoil in test hole TP 2-20. The fill was generally observed to consists of a grey silty clay with gravel and cobbles.

A 0.9 m thick layer of silty sand with trace amounts of clay and organic matter was encountered at the existing ground surface in borehole BH 16-14.

A glacial till deposit was observed underlying the silty clay at approximate depth of 7.5 to 8.4 m within boreholes BH 16-14, BH 13-7 and BH 13-8. The glacial till deposit was generally observed to consist of a very stiff, grey clayey silt to silty clay with sand, gravel, cobbles and boulders.

Practical refusal to augering was encountered in borehole BH 13-8 at a depth of 8.8 m below the existing ground surface.

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.

Based on available geological mapping, the bedrock at the subject site consists of interbedded sandstone and dolomite of the March formation with an overburden thickness of 10 to 25 m.

Laboratory Testing

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay/clayey silt 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. The test samples classify as inorganic clays of low plasticity (CL) and inorganic clays of high plasticity (CH) in accordance with the Unified Soil Classification System.

Table 1 - Atterberg Limits Results

Sample	Sample	LL (%)	PL (%)	PI (%)	w (%)	Classification
TP 1-20	G2	38	21	17	32.2	CL
TP 2-20	G2	48	21	26	28.7	CL
TP 3-20	G2	55	24	30	32.5	CH
TP 4-20	G3	52	26	26	30.8	CH
BH 14-2	TP5	56	24	32	48	CH
BH 13-7	SS3	38	14	24	32	CL
BH 13-7	SS6	42	15	27	38	CL

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

Grain size distribution analysis was also completed on selected soil samples. The results of the grain size analysis are summarized in Table 2 and presented on the Grain Size Distribution Results sheets in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis					
Test Hole	**Sample**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
TP 2-20	G2	6.4	16	41.2	36.4
TP 4-20	G3	4.7	5.8	42.4	47.1
BH 13-7	SS10	5	40	46	9

4.3 Groundwater

Open hole groundwater levels were recorded during excavation of the test holes for the current geotechnical investigation. Groundwater level readings were also measured at the piezometer locations by others, subsequent to the completion of the geotechnical investigations. The observed groundwater levels are summarized in Table 3.

Table 3 - Summary of Groundwater Level Readings				
Test Hole Number	**Ground Surface Elevation (m)**	**Groundwater Depth (m)**	**Groundwater Elevation (m)**	**Date**
TP 1-20	88.17	3.18	84.99	October 28, 2020
TP 2-20	87.94	Dry	-	October 28, 2020
TP 3-20	84.38	3.30	81.08	October 28, 2020
TP 4-20	84.10	2.90	81.20	October 28, 2020
BH 14-1	87.92	0.72	87.20	December 19, 2014
Note: Ground surface elevations at test hole locations were surveyed by Paterson and are referenced to a geodetic datum.				

It should be noted that surface water can become perched within a backfilled borehole, which can lead to higher than normal groundwater level readings. The long-term groundwater level can also be estimated based on the recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is anticipated at a depth of approximately 3 to 5 m below ground surface. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed residential development. It is expected that the proposed residential dwellings will be founded on conventional spread footings placed on an undisturbed, firm 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.

The proposed development is bordered to the west by the east valley corridor wall of the Rideau River and to the north by an existing ravine. A geotechnical limit of hazard lands line, from the top of slope, was established based on the City of Ottawa guidelines for slope stability assessment. The limit of hazard lands line is indicated in Drawing PG3320-1 - Test Hole Location Plan presented in Appendix 2.

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 footprints. Under paved areas, existing construction remnants, such as foundation walls should be excavated to a minimum depth of 1 m below final grade.

Fill Placement

Fill used for grading beneath the buildings should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or B Type II. Alternatively, consideration to placing a workable brown silty clay fill or glacial till, free of oversized boulders can be considered provided the material is placed and compacted using suitable compaction equipment and placed under dry and above freezing temperatures. These materials should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to reduce 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 their respective SPMDD. Separation will be required to remove all stones greater than 300 mm in their longest dimension prior to reusing the glacial till. 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 4 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, stiff silty clay, an undisturbed, dense glacial till bearing surface or engineered fill prepared as detailed in Subsection 5.2 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 **250 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 one 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 the concrete for the 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 silty clay 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 of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise Restrictions

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 **2.5 m** above existing ground surface is recommended for the subject site.

5.4 Design for Earthquakes

The proposed site can be taken as seismic site response Class C as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A) for foundations considered at this site. The soils underlying the site are not susceptible to liquefaction.

5.5 Slab-on-Grade / Basement Slab Construction

With the removal of topsoil and fill, containing organic matter or deleterious materials, within the footprint of the proposed buildings, the native soil surface or approved engineered fill pad will be considered an acceptable subgrade surface 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 A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone for slab on grade construction. If consideration is given to a basement level, the upper 200 mm of sub-floor fill should consist of a 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

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 = $(1.45\text{-}a_{max}/g)a_{max}$
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.

5.7 Pavement Structure

For design purposes, the pavement structures presented in the following tables could be used for the design of car only perking area and local roadways.

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 - Either 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	

The pavement thickness should be increased if bus traffic or other frequent heavy truck 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 granulars (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 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

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter perforated and 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.

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 a composite drainage system (such as Miradrain G100N or system Platon) connected to a perimeter drainage system is provided. Imported granular materials, such as clean sand or OPSS Granular B Type I material, should be used for this purpose.

6.2 Protection Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to reduce the trench width, where applicable, the excavation can be carried out within the confines of a fully braced steel trench box or other acceptable shoring systems.

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 subsoil at this site 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 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.

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 the impermeable nature of the subsurface soils, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

The subsurface soil conditions mostly 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. 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 installation of straw, propane heaters and tarpaulins or other suitable means. 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 to avoid the introduction of frozen materials, snow or ice 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 during construction. Also, 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.

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 slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Restrictions

Paterson completed a soils review of the site to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) for trees planted within a public right-of-way (ROW). Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Gradation testing was also completed on selected soil samples. The above-noted test results were completed on samples taken at depths between the anticipated underside of footing elevation and a 3.5 m depth below finished grade. The results of our testing are presented in Tables 1 and 2 in Subsection 4.2 and in Appendix 1.

A low to medium sensitivity clay soil was encountered between the anticipated design underside of footing elevations and 3.5 m below finished grade, based on our Atterberg limits test results which indicated the modified plasticity index does not exceed 40% of the silty clay deposit at the subject site.

The following tree planting setbacks are therefore recommended. Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to **4.5 m** for small (mature height up to 7.5 m) and medium size trees (mature tree 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 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.

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

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

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

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

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 October 16, 2020 as part of the slope stability assessment. Four slope cross-sections were studied as the worst case scenarios. The cross section locations are presented on Drawing PG3320-1 - Test Hole Location Plan attached to the current report.

The existing slope on the western end of the site extending down to the Rideau River generally has a height of 4 to 6 m with an incline ranging from approximately 4H:1V to 3H:1V. This slope is generally vegetated with trees. Two outlet structures for the existing SWMP have been built in

The ravine located in the northern portion of the site has a height of approximately 4 to 6 m near River Road, decreasing to an approximate heights of 1 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.

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 PG3320-1 - 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	5
Grey Silty Clay	16	33	10
Clayey Silt to Silty Clay Glacial Till	20	33	0

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 the geotechnical investigations completed by others 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	-	100
Grey Silty Clay	16	-	40
Clayey Silt to Silty Clay Glacial Till	20	33	0

Static Loading Analysis

The results for the slope stability analyses under static conditions at Sections A, B, C, and D are shown on Figures 2, 4, 6, and 8 attached to the present report. The factor of safety was found to be greater than 1.5 at Sections A and B. However, Section D requires a setback of 13.2 m from the top of slope to obtain a factor of safety greater than 1.5. The factor of safety at Section C was found to be less than 1.5, however, a negligible (less than 0.5 m) setback from the top of slope would be required to achieve a factor of safety of 1.5.

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, 7, and 9 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

Signs of erosion were noted along the slope face that confines the Rideau River located along th western boundary of the subject site. Some minor sloughing failures were noted in the lower portion of the slope, leaving some exposed tree roots.

Although signs of historic erosion were observed during our site review, the watercourse is dam controlled and within a confined channel which is able to support routine flows. Further, the slope face was observed to be in a stable condition and signs of active erosion were not observed.

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 Rideau River. Therefore, a toe erosion allowance of 7 m in addition to an erosion access allowance of 6 m, applied from the top of slope, is considered appropriate for the slope along the Rideau River.

For the ravine in the northern portion of the site (Section A), minor signs of erosion such as localized undercutting and exposed tree roots were observed. However, no signs of active erosion were noted. Further, given the generally stiff cohesive soils encountered within the depth of the ravine, a toe erosion allowance of 2 m in addition to an erosion of 6 m is considered appropriate for the ravine, as applied from the top of slope.

The Limit of Hazard Lands, which includes these allowances, are indicated on Drawing PG3320-1 - Test Hole Location Plan attached to the present report. 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

The following is recommended to be completed once the site plan and development are determined:

- ❑ Complete a supplemental investigation once design details are finalized.
- ❑ Review detailed grading plan(s) from a geotechnical perspective.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Review and inspection of all foundation drainage systems.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to ensure that the specified level of compaction has been achieved.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been completed in general accordance with Paterson's recommendations could be issued upon request, following the completion of a satisfactory material 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 its 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 thanRiverside South Development Corporation, 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.

Kevin A. Pickard, EIT

David J. Gilbert, P.Eng.

Report Distribution

- ❑ Riverside South Development Corporation (e-mail copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE RECORDS BY OTHERS

ATTERBERG LIMIT TESTING RESULTS

GRAIN SIZE DISTRIBUTION RESULTS

GRAIN SIZE DISTRIBUTION RESULTS BY OTHERS

ANALYTICAL TEST RESULTS BY OTHERS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Riverside South-Phase 12 - 708-720 River Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE October 28, 2020

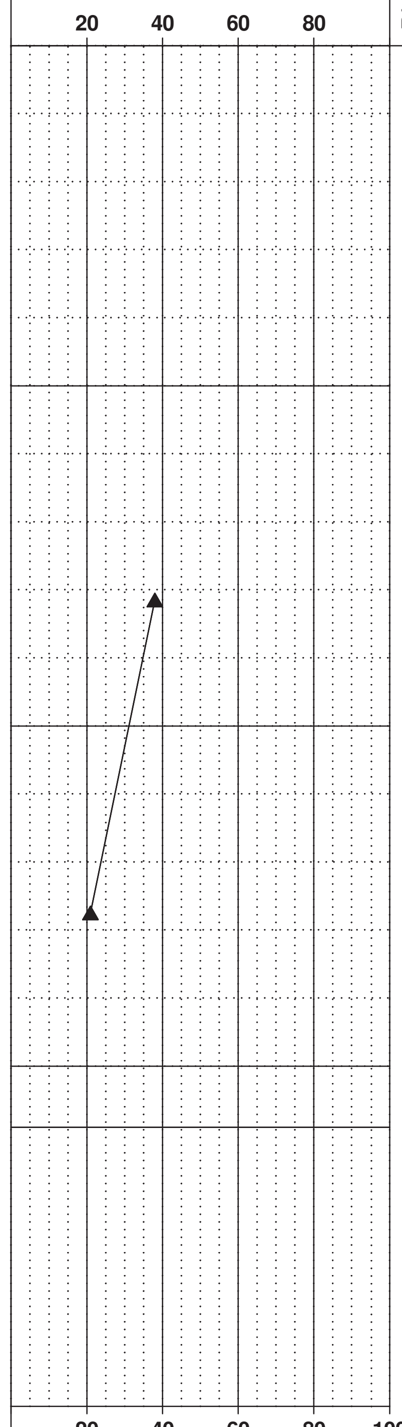
FILE NO. PG3320

HOLE NO. TP 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.17
TOPSOIL (0.30)							
Firm to soft, brown **SILTY CLAY**		G	1				
						1	87.17
		G	2				
						2	86.17
		G	3				
						3	85.17
(3.18) End of Test Pit (Groundwater infiltration at bottom of test pit)							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Riverside South-Phase 12 - 708-720 River Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

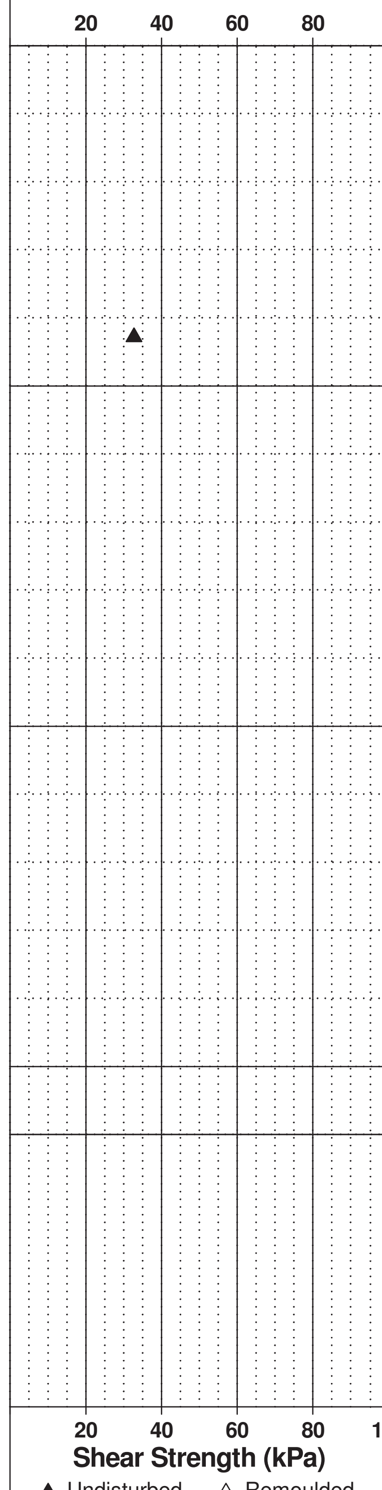
BORINGS BY Backhoe

DATE October 28, 2020

FILE NO. PG3320

HOLE NO. TP 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	87.94		
TOPSOIL 0.20									
FILL: Firm, grey silty clay		G	1						
		G	2						
1.20						1	86.94		
FILL: Grey silty clay, trace gravel, cobbles		G	3						
2.20						2	85.94		
Brown **SILTY CLAY**						3	84.94		
3.20									
End of Test Pit									
(TP dry upon completion)									



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Riverside South-Phase 12 - 708-720 River Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG3320

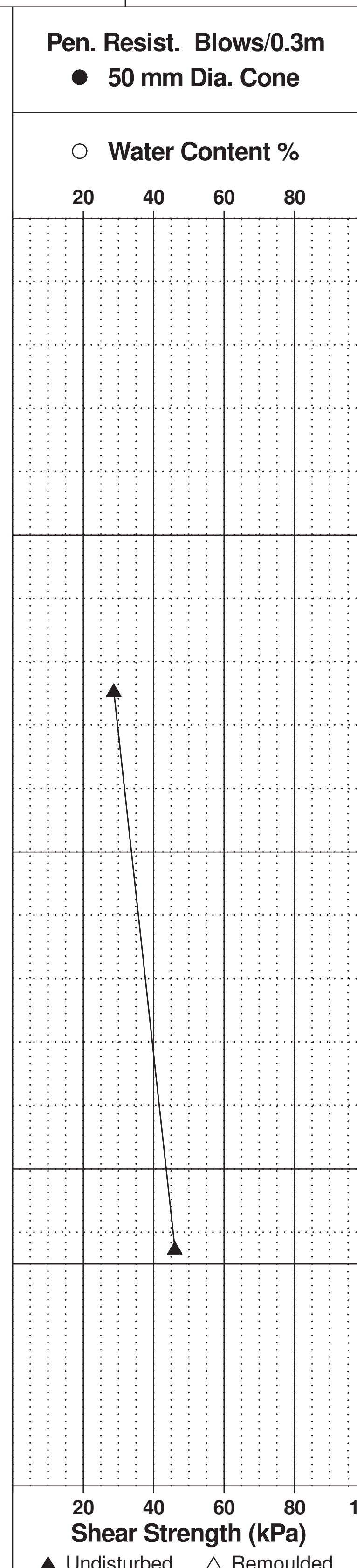
REMARKS

HOLE NO. TP 3-20

BORINGS BY Backhoe

DATE October 28, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	84.38
TOPSOIL		G	1				
0.30							
						1	83.38
		G	2				
Firm, brown **SILTY CLAY**							
						2	82.38
						3	81.38
3.30		G	3				
End of Test Pit (Groundwater infiltration at bottom of test pit)							



Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Riverside South-Phase 12 - 708-720 River Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE October 28, 2020

FILE NO. PG3320

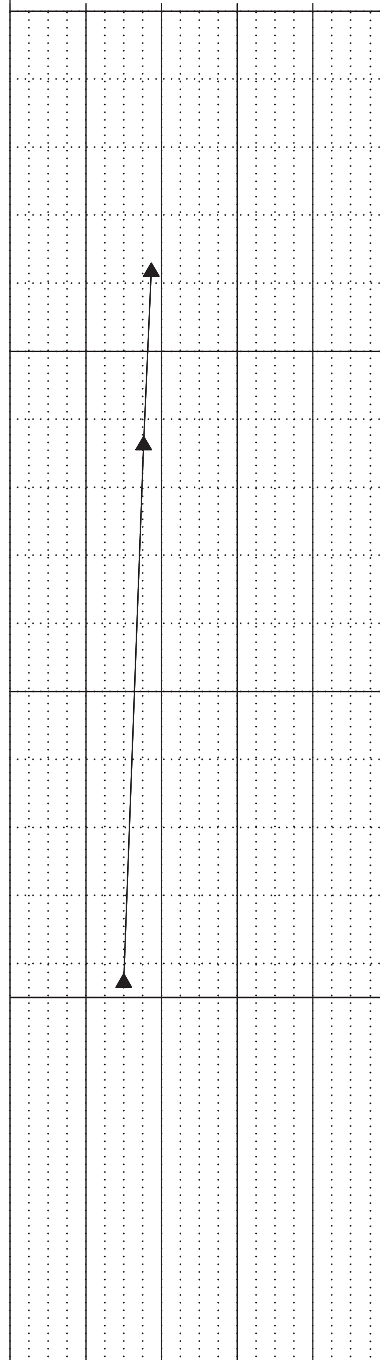
HOLE NO. TP 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	84.10
Firm, brown **SILTY CLAY**		G	1				
						1	83.10
		G	2				
						2	82.10
2.90		G	3				
End of Test Pit (Groundwater infiltration at bottom of test pit)							

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

○ Water Content %

20 40 60 80



20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Piezometer Construction

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

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

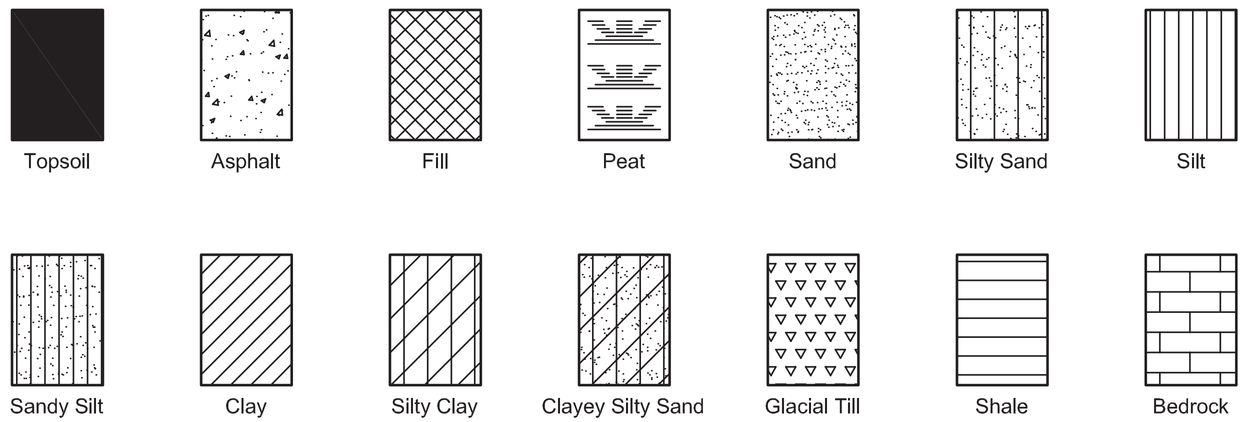
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

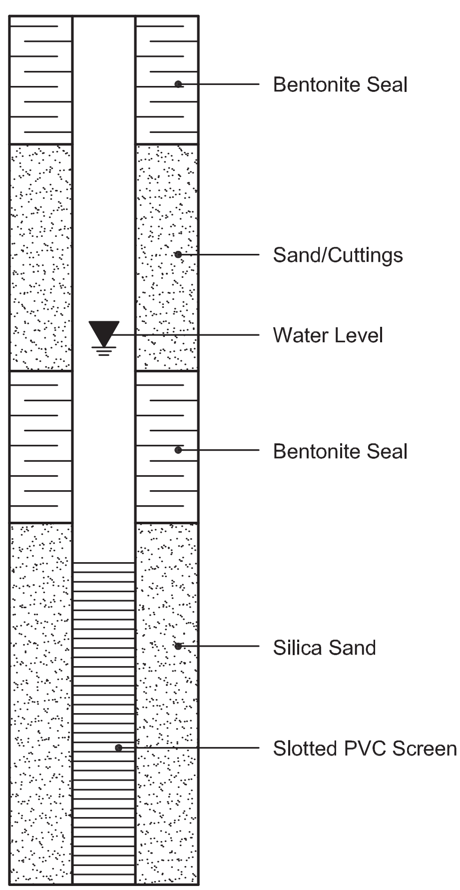
SYMBOLS AND TERMS (continued)

STRATA PLOT

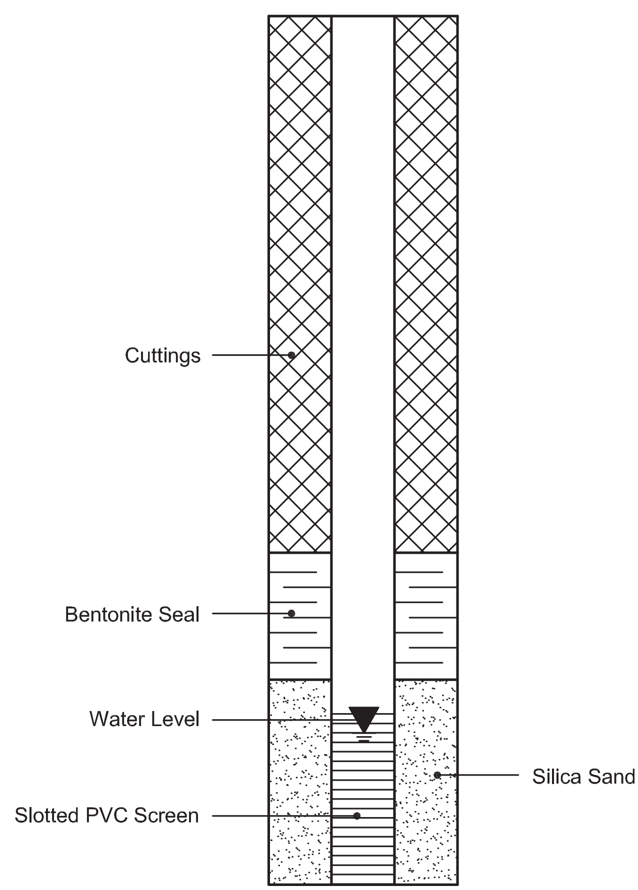


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Borehole: BH16-14

Project:	Subsurface Investigation	**Drilling method:**	CME 850 (Hollow Stem Auger)
Client:	City of Ottawa	**Date started/completed:**	14-Oct-2016
Location:	River Road	**Ground surface elevation:**	84.11 m AMSL
Number:	163401322	**Top of casing elevation:**	n/a
Field investigator:	B. Chenier	**Easting:**	367271.409
Contractor:	CCC	**Northing:**	5013688.4

SUBSURFACE PROFILE

Stratigraphic Description	Elevation (m AMSL) / Depth (m BGS)
Ground Surface	84.11
Loose, brown, SAND (SP), with silt, trace clay and organic matter	0.00
	83.25
Very stiff, grey, lean CLAY (CL), trace sand	0.86
- trace orange	
	80.30
Very stiff, grey, CLAY (CH)	3.81
- becomes wet	
- Su > 120 kPa	
- Su > 120 kPa	
	76.64
Very stiff, grey, clayey SILT TILL (ML), with medium gravel, trace sand	7.47
	76.49
End of Borehole	7.62

Depth scale: (ft) 5, 10, 15, 20, 25, 30, 35, 40, 45; (m) 2, 4, 6, 8, 10, 12, 14

SAMPLE DETAILS

Sample Number	Sample Type	Recovery	N Value
1	SS	13" 54%	2-3-3-3 (6)
2	SS	19" 79%	3-4-4-6 (8)
3	SS	23" 96%	4-5-7-10 (12)
4	SS	24" 100%	3-3-4-5 (7)
5	SS	24" 100%	2-2-3-4 (5)
6	SS	24" 100%	1-2-2-4 (4)
7	SS	24" 100%	1-1-2-3 (3)
8	SS	n/a	0-0-1-1 (1)
9	SS	n/a	0-1-1-10 (2)

INSTALLATION DETAILS

Diagram	Description
	Backfilled with bentonite

Notes:
m AMSL - metres above mean sea level
m BGS - metres below ground surface
SS - split-spoon sample
n/a - not available

RECORD OF BOREHOLE: 14-1

PROJECT: 1406631

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 12, 2014

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Boring Method: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	GROUND SURFACE	87.92					
	TOPSOIL - (ML) sandy SILT; dark brown	0.00					
	(ML) CLAYEY SILT, some sand; brown; cohesive, w>PL, very stiff	87.67 / 0.25					
1			1	SS	6		
	(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown, highly fissured (WEATHERED CRUST); cohesive, w~PL, very stiff to stiff	86.55 / 1.37	2	SS	4	CHEM	Native Backfill
2–3			3	SS	3		
3			4	SS	2		
4	(CI/CH) SILTY CLAY to CLAY; grey, with black mottling; cohesive, w>PL, soft to firm	83.81 / 4.11	5	SS	2		Bentonite Seal
5							Silica Sand
			6	TP	PH		Standpipe
6							Cave
	End of Borehole	81.22 / 6.70					
7							WL in Standpipe at Elev. 87.20 m on Dec. 19, 2014

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp —○W— Wl (20 40 60 80)

DEPTH SCALE 1 : 50

GOLDER

LOGGED: DWM

CHECKED: SAT

RECORD OF BOREHOLE: 14-2

PROJECT: 1406631

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 11, 2014

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ├──○W──┤ Wl (20 40 60 80)

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

ELEV. / DEPTH (m)	SOIL PROFILE DESCRIPTION
87.59	GROUND SURFACE
0.00	TOPSOIL - (ML) sandy SILT; dark brown
87.29 / 0.30	(ML) CLAYEY SILT, trace sand; brown; cohesive, w>PL, very stiff
86.07 / 1.52	(ML) SILT; grey brown; non-cohesive, very loose
85.31 / 2.28	(CI/CL-ML) SILTY CLAY/CLAYEY SILT; grey brown, with sand seams (WEATHERED CRUST); cohesive, w>PL, stiff
84.54 / 3.05	(CH/CI-ML) SILTY CLAY to CLAY, some silt seams; grey, with black mottling below 6.1 m depth; cohesive, w>PL, firm to stiff
79.97 / 7.62	End of Borehole

SAMPLE NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	4	
2	SS	2	
3	SS	2	
4	SS	PH	
5	TP	PH	C
6	SS	PH	

DEPTH SCALE 1 : 50

GOLDER

LOGGED: DWM

CHECKED: SAT

MIS-BHS 001 1406631.GPJ GAL-MIS.GDT 18-5-22 JM

Stantec

BOREHOLE RECORD

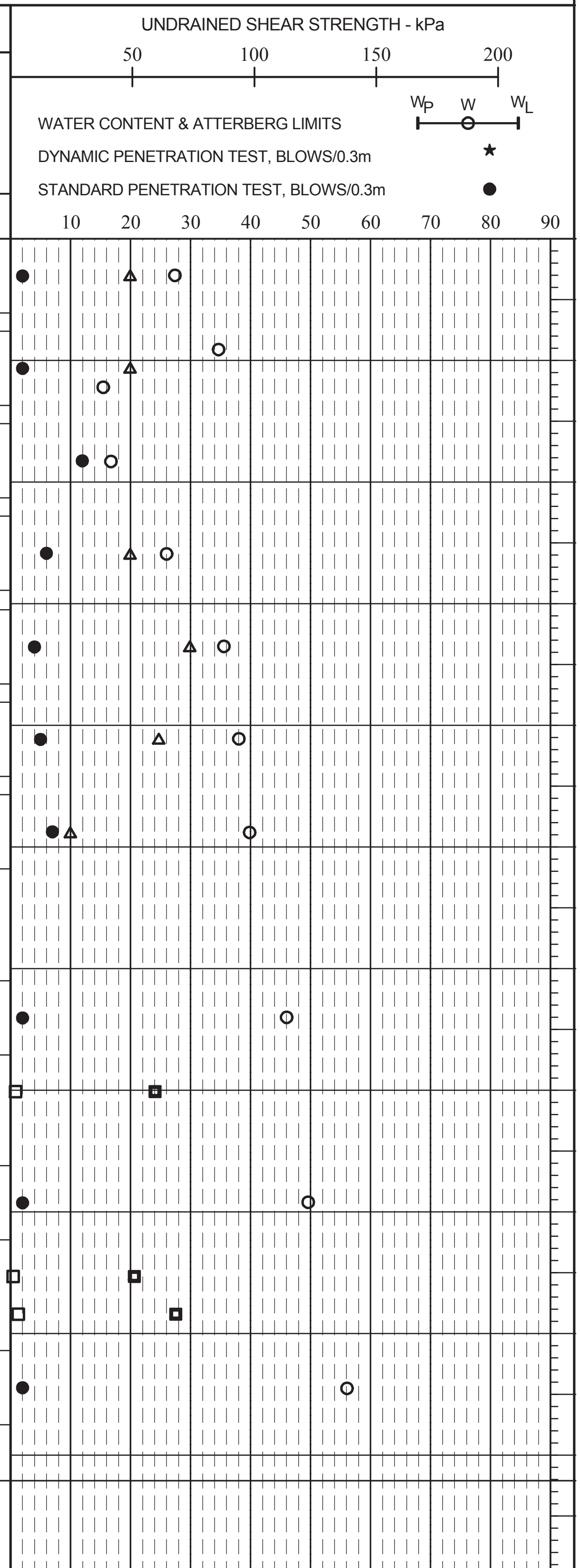
N: 5 013 826 E: 367 383

BH13-1

CLIENT Claridge Homes
LOCATION 752 River Road Ottawa ON
DATES: BORING December 6, 2013 WATER LEVEL

BOREHOLE No. BH13-1
PROJECT No. 160401052
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	87.61	Firm brownish grey sandy lean CLAY with gravel (CL)	SS	1	370	2
1	86.6	Compact brown to grey poorly graded GRAVEL with clay and sand (GP-GC)	SS	2	200	2
			SS	3	60	12
2	85.5	Firm to very stiff brown to grey lean CLAY (CL)	SS	4	410	6
3			SS	5	610	4
4			SS	6	680	5
5			SS	7	670	7
		$-s_u$ > 108 kPa @ 5.49 m				
6	81.5	Firm to very stiff dark grey to grey fat CLAY (CH)	SS	8	620	2
7		-moist				
8			SS	9	640	2
9			SS	10	640	2
10	77.5					
	77.5	$-s_u$ > 108 kPa @ 10.06 m				
		End of Borehole				
11						



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

App'd ______
Date ______

Stantec

BOREHOLE RECORD

N: 5 013 718 E: 367 400

CLIENT Claridge Homes
LOCATION 752 River Road Ottawa ON
DATES: BORING December 6, 2013 WATER LEVEL

BOREHOLE No. BH13-7
PROJECT No. 160401052
DATUM Geodetic

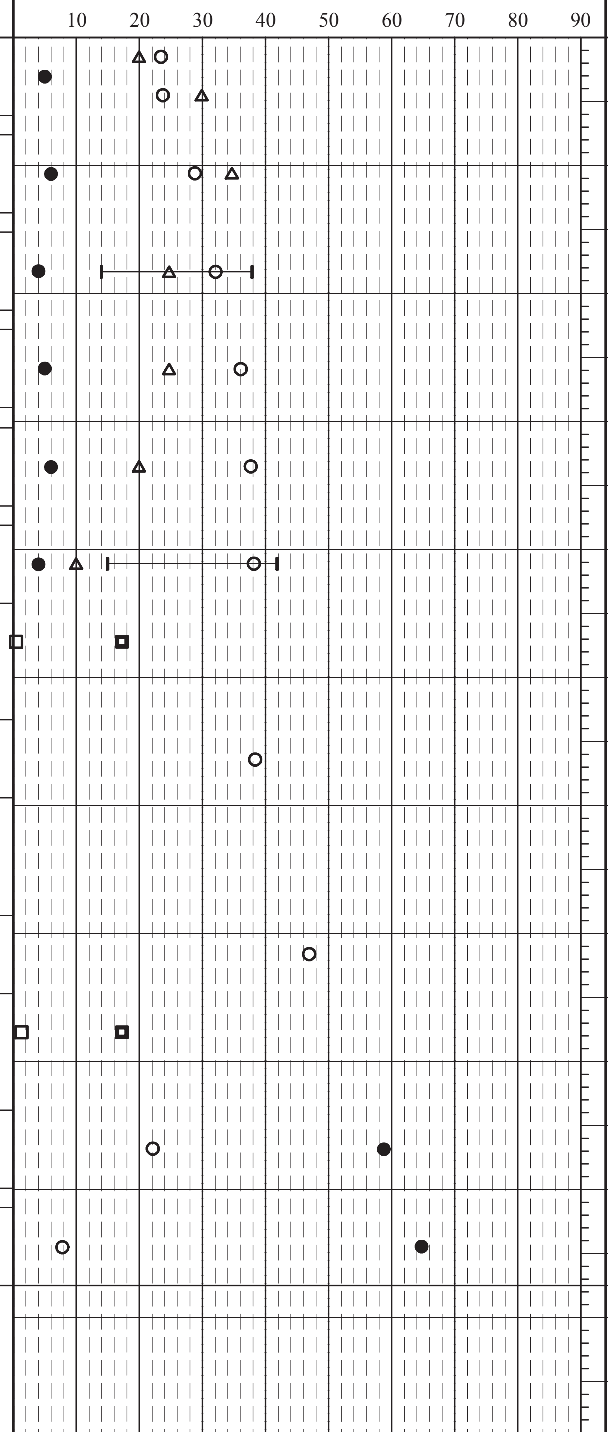
DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION
0	87.97	
0	87.8	Firm brown sandy lean clay TOPSOIL (CL) -with organics
~0.15–4.55		Firm to very stiff brown to grey lean CLAY (CL)
4.55	83.4	Firm to very stiff dark grey to grey fat CLAY (CH) -trace of sand -s_u > 108 kPa @ 6.25 m
8.4	79.6	Very dense grey well graded gravel with silty clay and sand TILL (GW-GC) Frequent cobbles and boulders
9.75	78.2	End of Borehole

SAMPLES

TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
SS	1	480	5
SS	2	660	6
SS	3	640	4
SS	4	640	5
SS	5	670	6
SS	6	670	4
SS	7	680	WH
SS	8	670	WH
SS	9	80	59
SS	10	190	65

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●



Inferred Groundwater Level
Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

App'd
Date

Stantec

BOREHOLE RECORD

N: 5 013 724 E: 367 289

BH13-8

CLIENT Claridge Homes
LOCATION 752 River Road Ottawa ON
DATES: BORING December 5, 2013 WATER LEVEL

BOREHOLE No. BH13-8
PROJECT No. 160401052
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	84.51	Firm brown sandy lean clay TOPSOIL (CL)	SS	1	440	3
	83.9	-with organics				
1		Firm to very stiff brown to grey lean CLAY (CL)	SS	2	400	4
			SS	3	660	4
2			SS	4	660	3
3			SS	5	660	7
4			SS	6	670	5
5			SS	7	640	4
			SS	8	690	4
6	78.4	Firm to very stiff dark grey to grey fat CLAY (CH)	SS	9	0	3
7						
8			SS	10	660	1
	76.3	Compact dark grey to grey silty sand TILL (SM)	SS	11	220	50/100 mm
	75.7	Frequent cobbles and boulders				
9		End of Borehole -Auger refusal on cobble or boulder at 8.78 m				
10						
11						

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

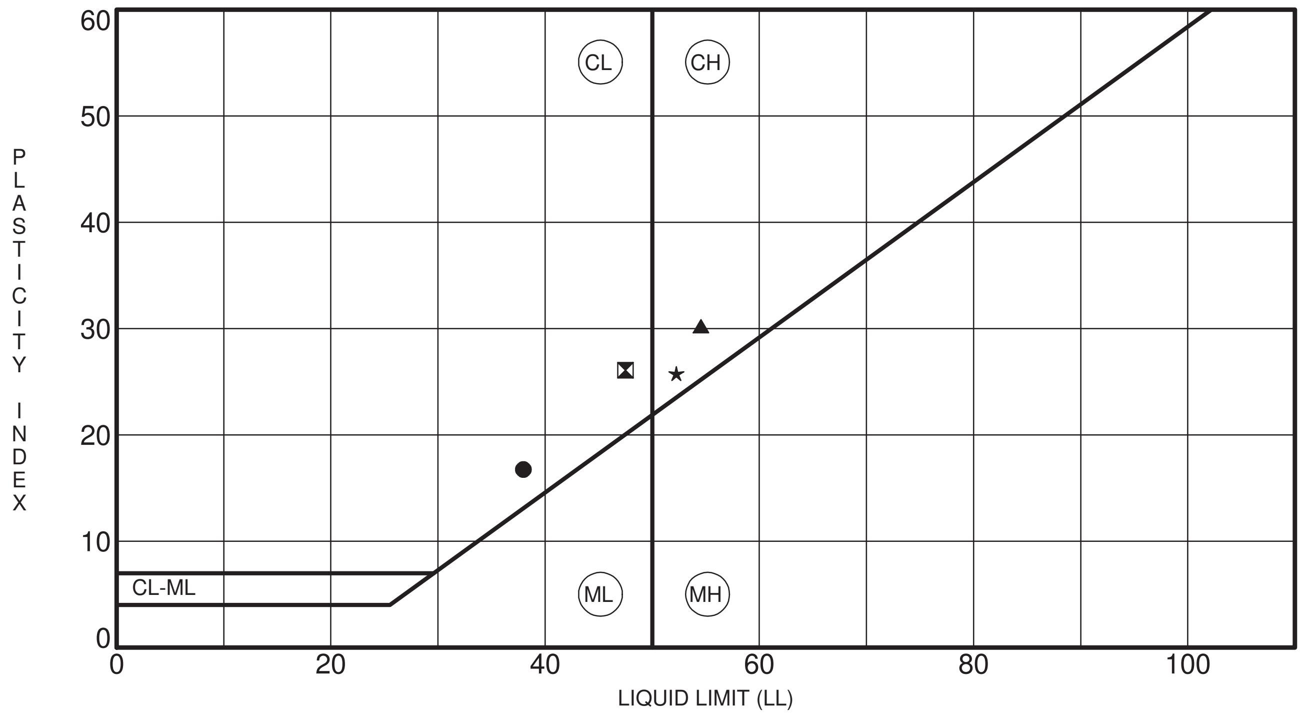
WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m
STANDARD PENETRATION TEST, BLOWS/0.3m

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level
Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

App'd
Date



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**TP 1-20**	**G 2**	**38**	**21**	**17**		**CL - Inorganic clays of low plasticity**
⧗	**TP 2-20**	**G 2**	**48**	**21**	**26**		**CL - Inorganic clays of low plasticity**
▲	**TP 3-20**	**G 2**	**55**	**24**	**30**		**CH - Inorganic clays of high plasticity**
★	**TP 4-20**	**G 2**	**52**	**26**	**26**		**CH - Inorganic clays of high plasticity**

CLIENT **Urbandale Corporation**

PROJECT **Geotechnical Investigation - 750 River Road**

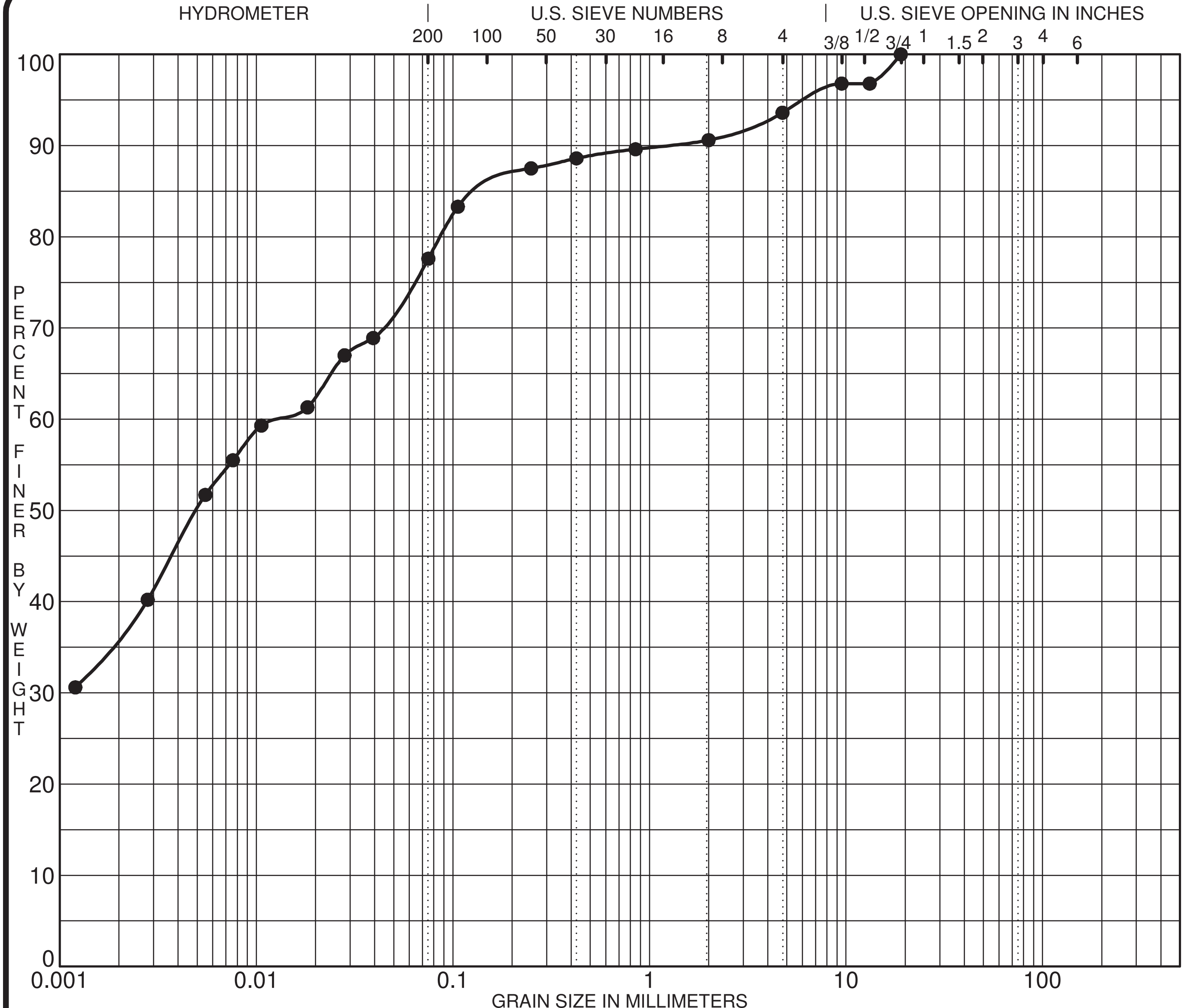
FILE NO. **PG3320**

DATE **28 Oct 20**

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
●	**TP 2-20**	**G2**	**CL - Inorganic clays of low plasticity**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP 2-20**	**G2**	**19.00**	**0.01**			**6.4**	**16.0**	**77.6**	

CLIENT **Urbandale Corporation**

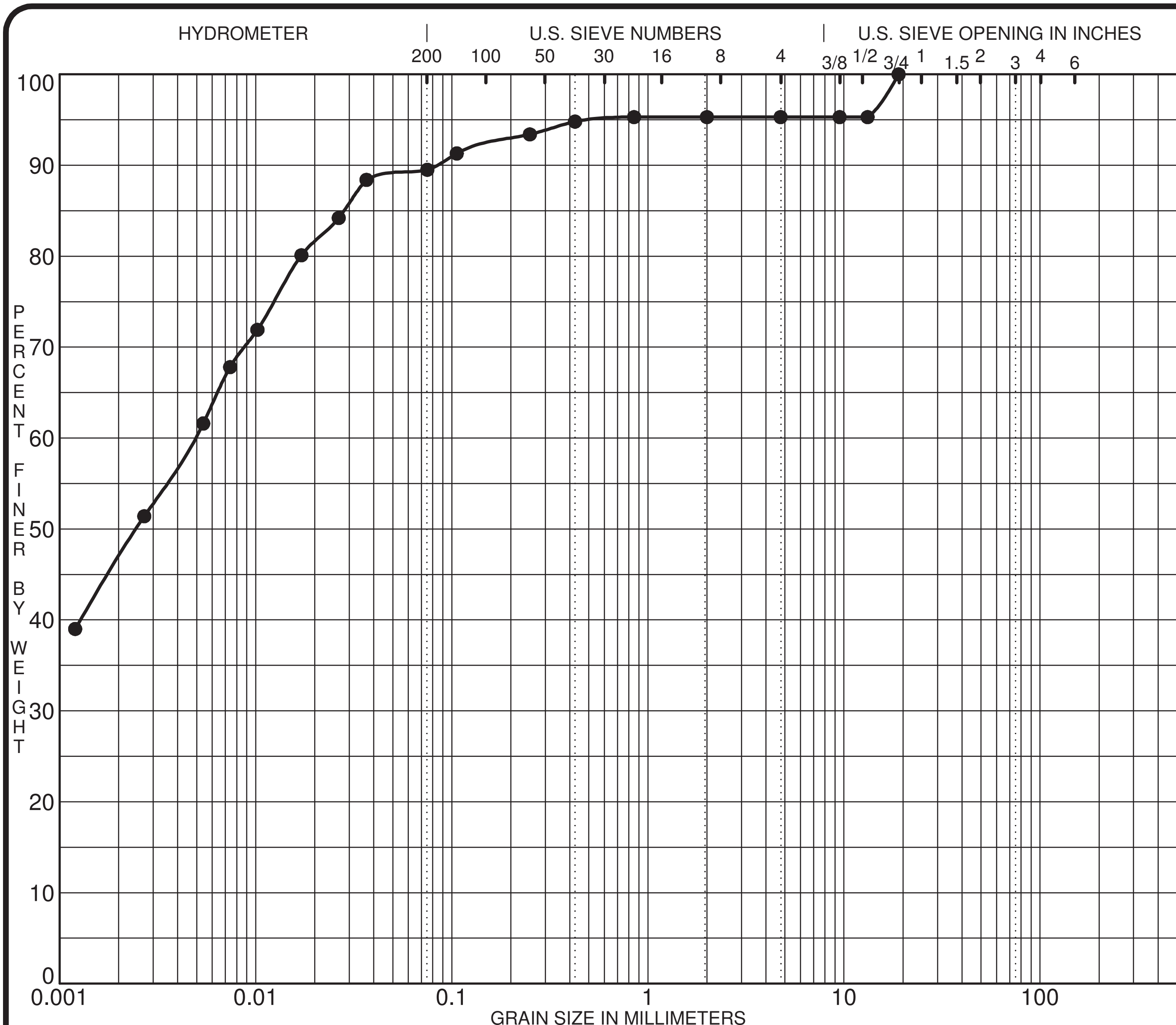
FILE NO. **PG3320**

PROJECT **Geotechnical Investigation - 750 River Road**

DATE **28 Oct 20**

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
●	**TP 4-20**	**G 3**	**CH - Inorganic clays of high plasticity**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP 4-20**	**G 3**	**19.00**	**0.00**			**4.7**	**5.8**	**89.5**	

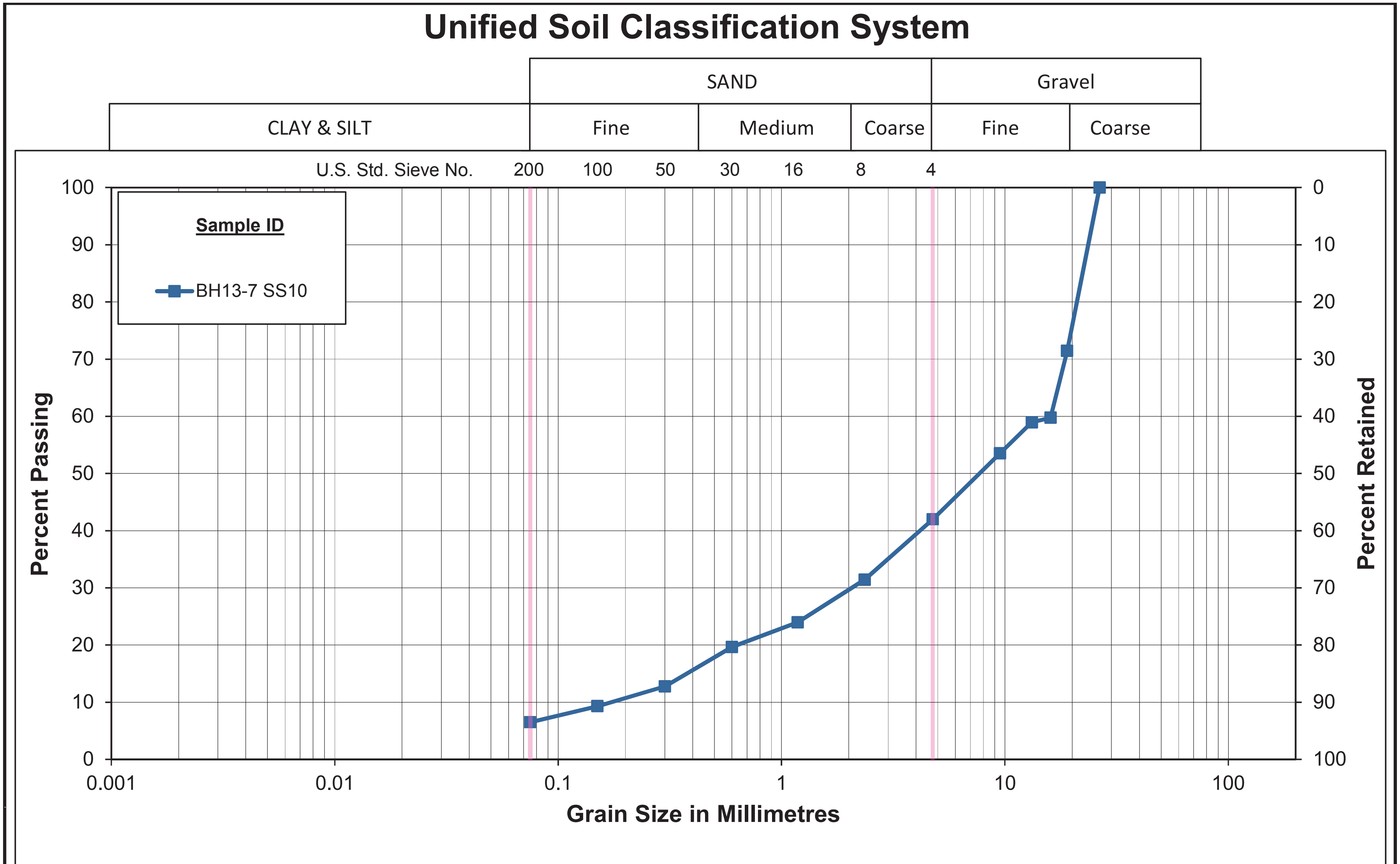
CLIENT **Urbandale Corporation** FILE NO. **PG3320**

PROJECT **Geotechnical Investigation - 750 River Road** DATE **28 Oct 20**

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



Stantec

GRAIN SIZE DISTRIBUTION

Gravel with Silty Clay and Sand: TILL

Figure No. 5

Project No. 160401052

EXOVA ENVIRONMENTAL ONTARIO

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
32 Steacie Drive
Kanata, ON
K2K 2A9
Attention: Ms. Susan Trickey
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1426815
Date Submitted: 2014-12-22
Date Reported: 2014-12-30
Project: 1406631
COC #: 792753

Group	Analyte	MRL	Units	Guideline	1153409	1153410
				Lab I.D.	1153409	1153410
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2014-12-12	2014-12-08
				Sample I.D.	BH 14-1 SA2/5'-7'	BH 14-10 SA2/5'-7'
Agri. - Soil	pH	2.0			7.4	7.5
General Chemistry	Cl	0.002	%		<0.002	0.003
	Electrical Conductivity	0.05	mS/cm		0.15	0.15
	Resistivity	1	ohm-cm		6670	6670
	SO4	0.01	%		<0.01	<0.01

Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 9 - SLOPE STABILITY ANALYSIS SECTIONS

PHOTOGRAPHS FROM SITE VISIT

DRAWING PG3320-1 - TEST HOLE LOCATION PLAN

DRAWING PG3320-2 - PERMISSIBLE GRADE RAISE PLAN

DRAWING PG3320-3 - TREE PLANTING SETBACK RECOMMENDATIONS

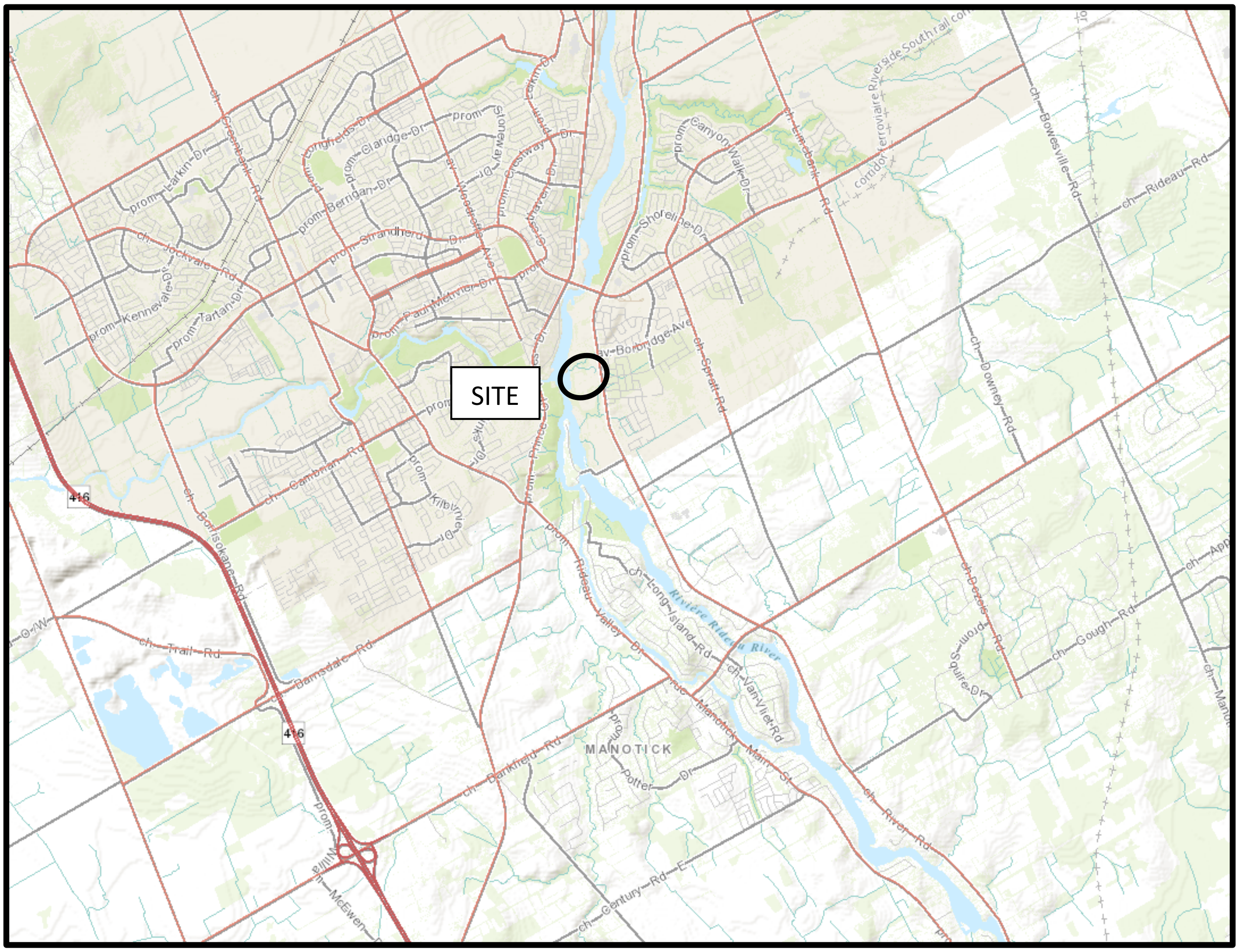


FIGURE 1

KEY PLAN

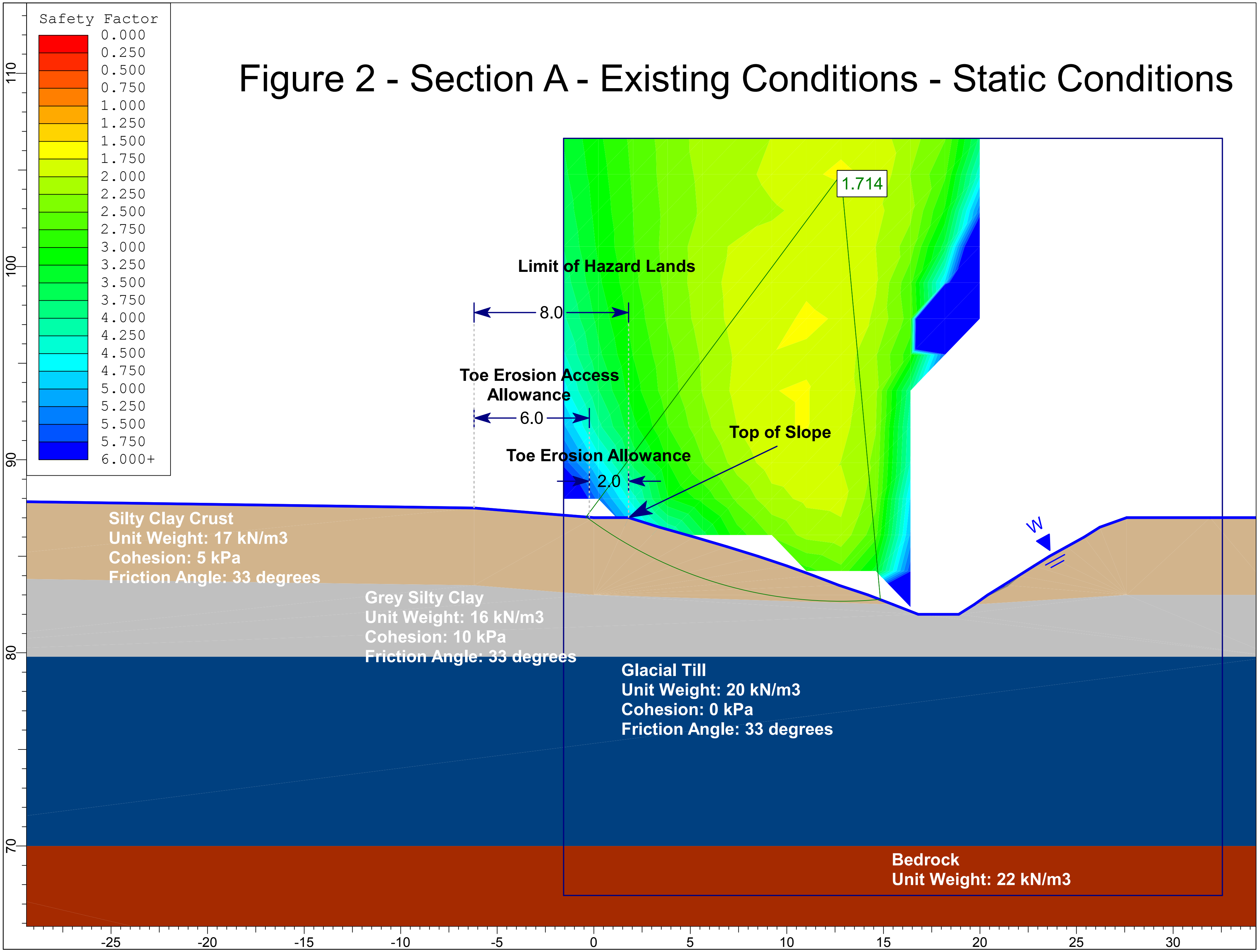


Figure 2 - Section A - Existing Conditions - Static Conditions
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.714
Limit of Hazard Lands
8.0
Toe Erosion Access Allowance
6.0
Toe Erosion Allowance
2.0
Top of Slope
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Grey Silty Clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 degrees
Bedrock
Unit Weight: 22 kN/m3
110
100
90
80
70
-25
-20
-15
-10
-5
0
5
10
15
20
25
30

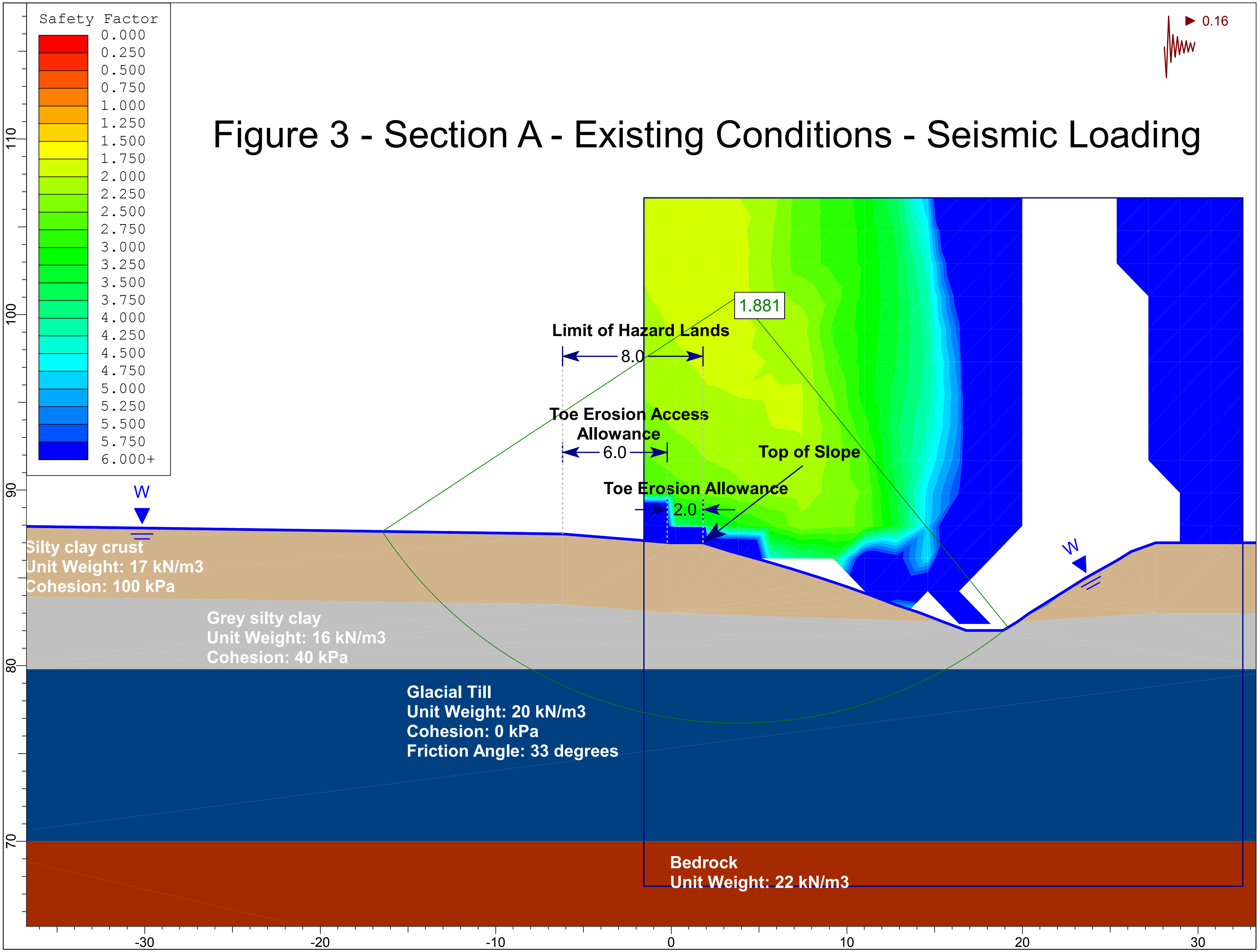


Figure 3 - Section A - Existing Conditions - Seismic Loading

Figure 4 - Section B - Existing Conditions - Static Conditions

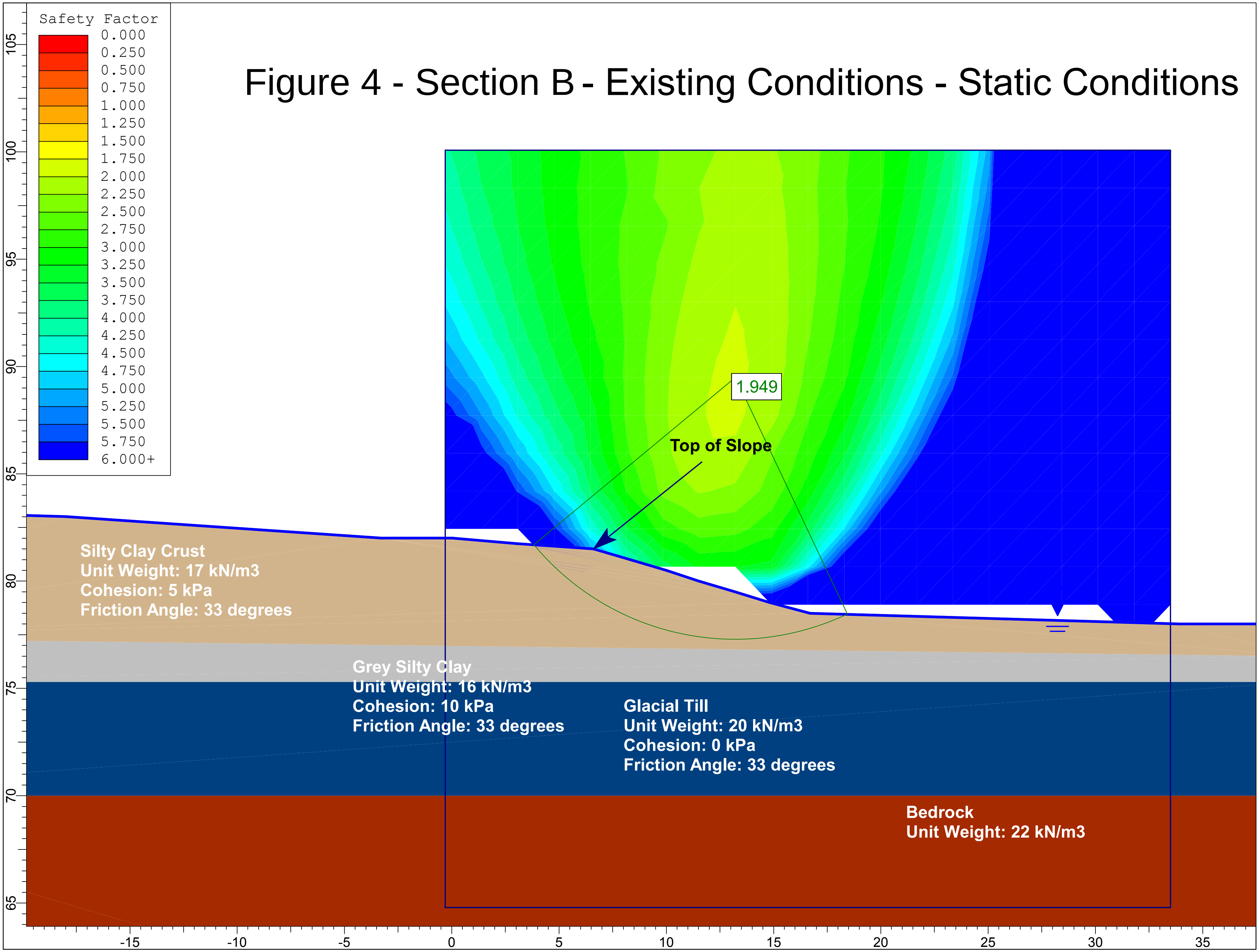
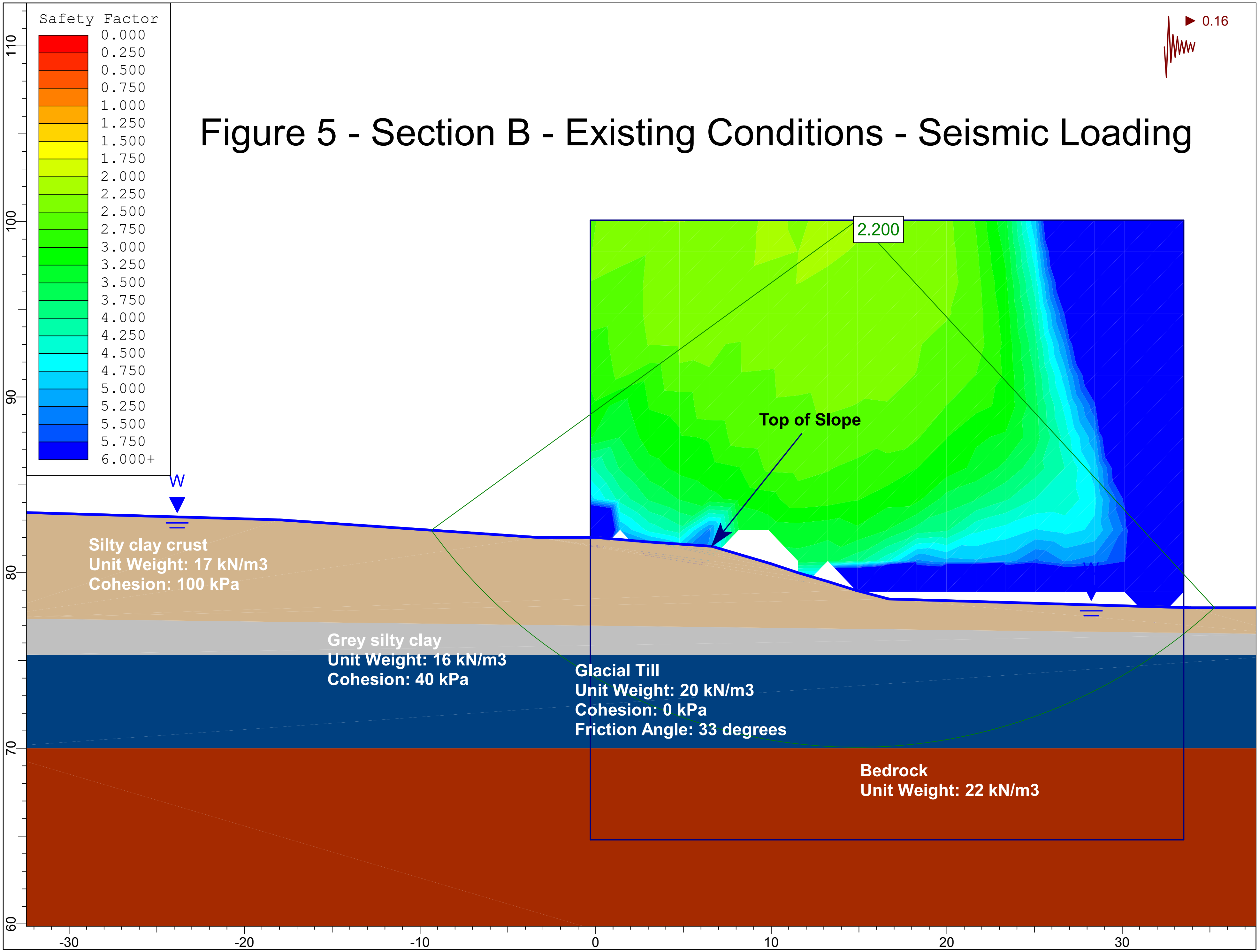


Figure 5 - Section B - Existing Conditions - Seismic Loading



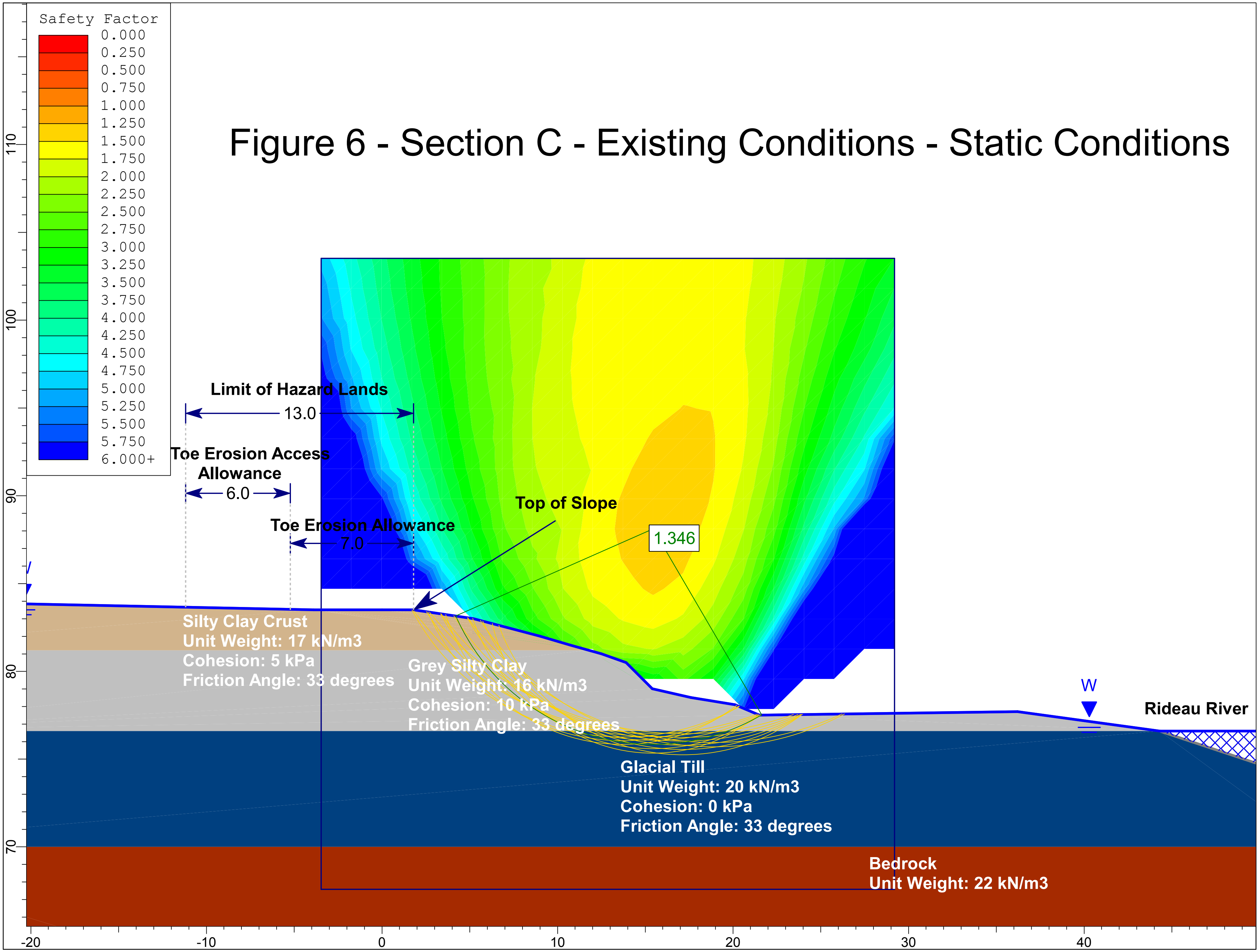
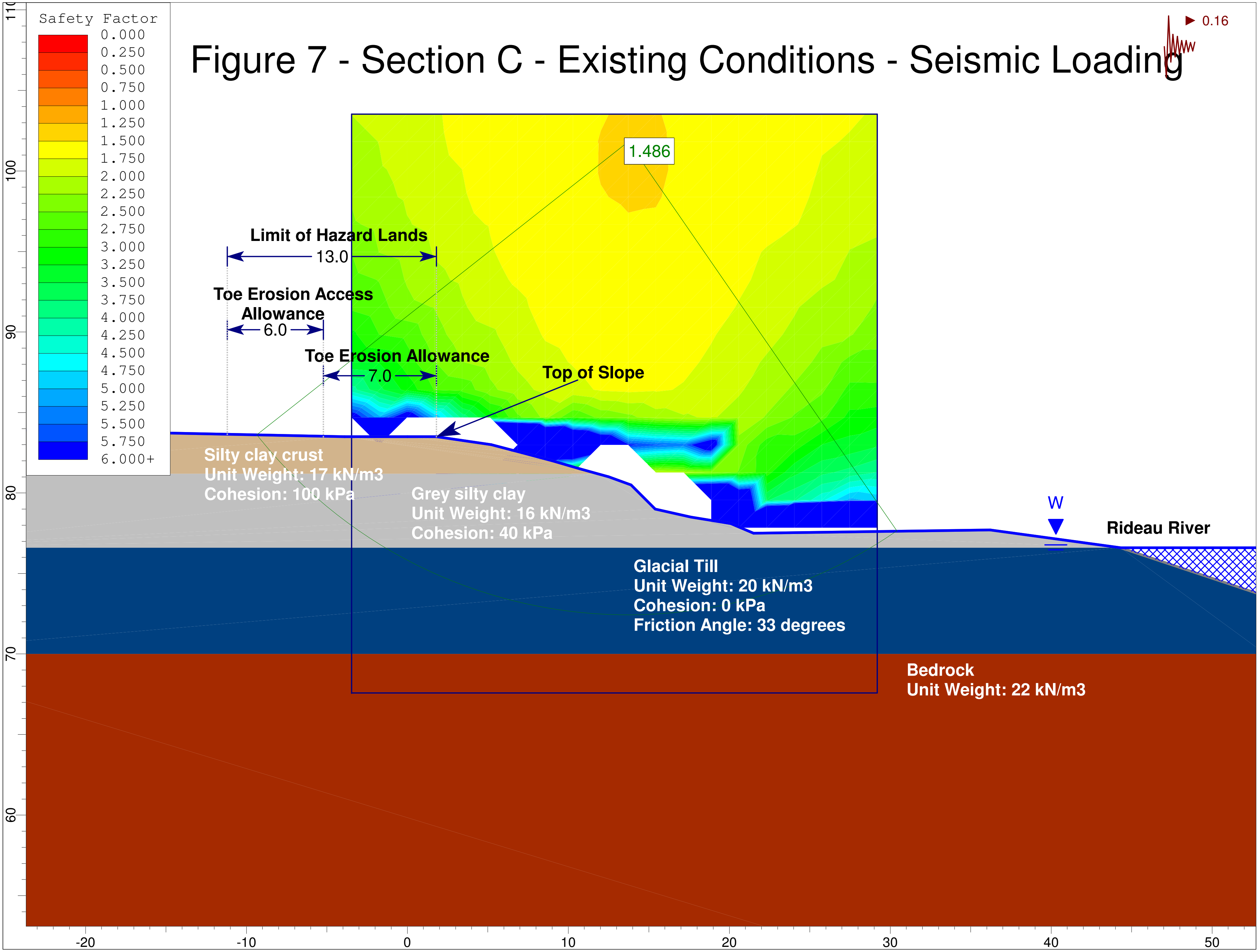


Figure 6 - Section C - Existing Conditions - Static Conditions
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
13.0
Toe Erosion Access Allowance
6.0
Toe Erosion Allowance
7.0
Top of Slope
1.346
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Grey Silty Clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 degrees
Bedrock
Unit Weight: 22 kN/m3
W
Rideau River
110
100
90
80
70
-20
-10
0
10
20
30
40

Figure 7 - Section C - Existing Conditions - Seismic Loading



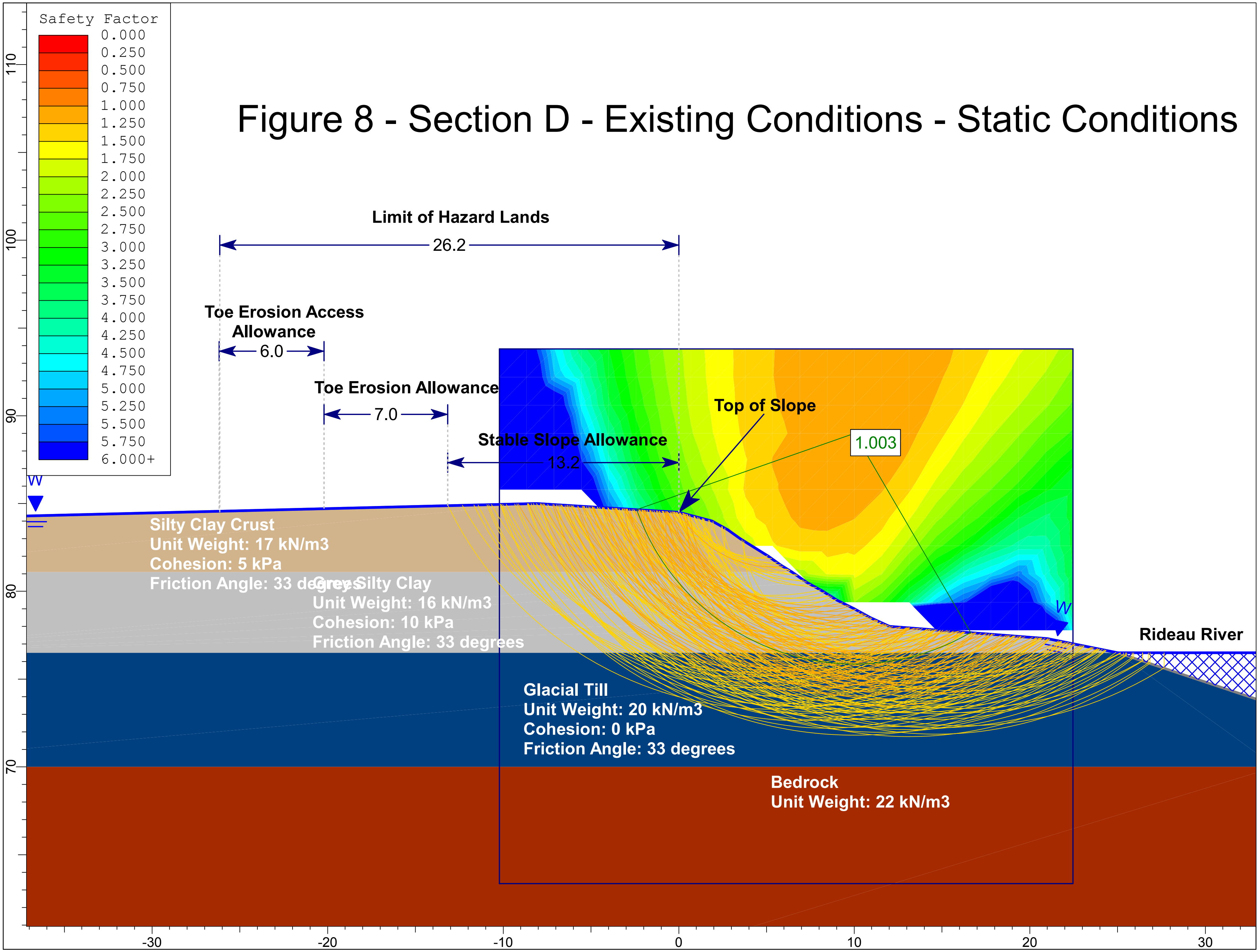
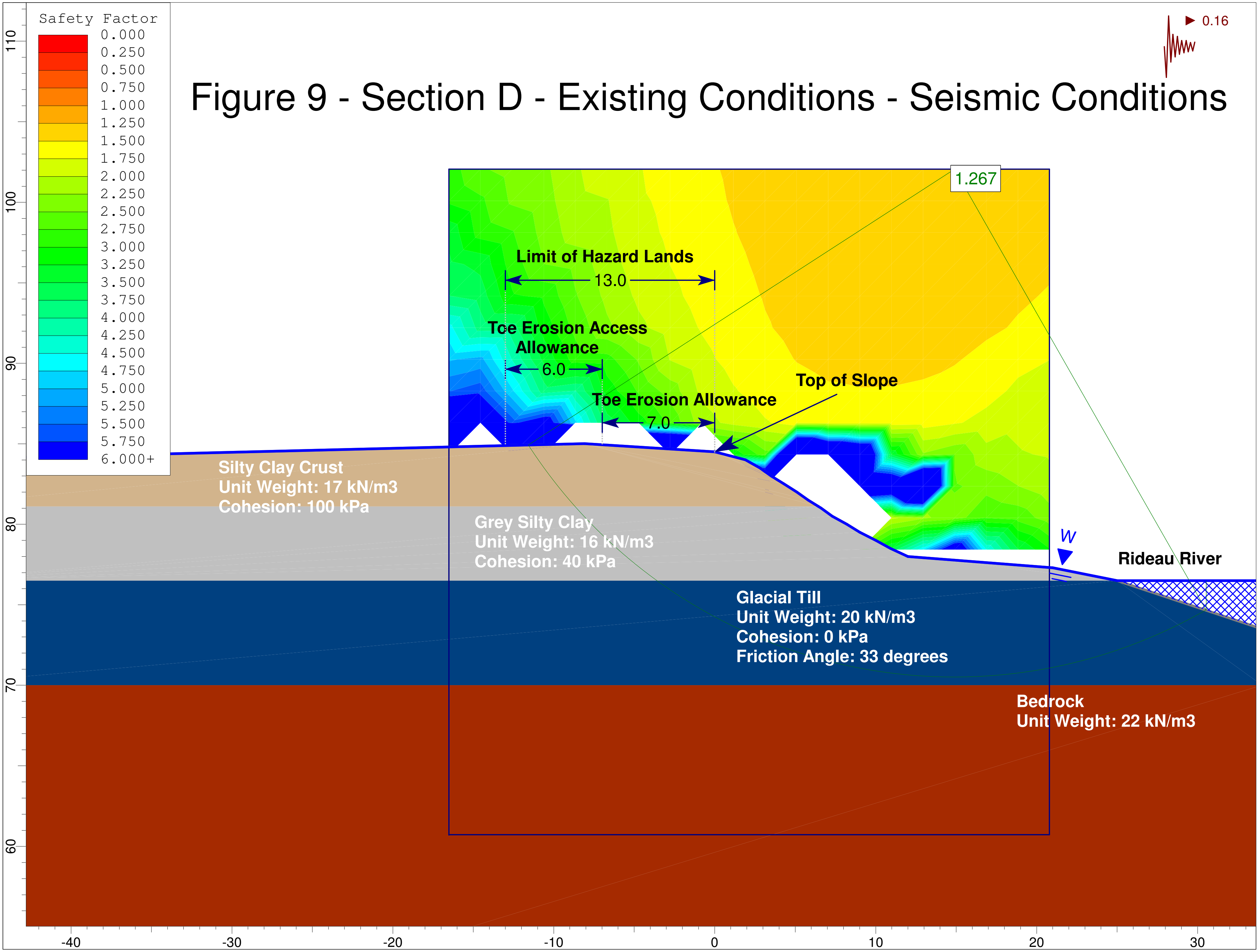


Figure 8 - Section D - Existing Conditions - Static Conditions
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
26.2
Toe Erosion Access Allowance
6.0
Toe Erosion Allowance
7.0
Stable Slope Allowance
13.2
Top of Slope
1.003
Rideau River
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Grey Silty Clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 degrees
Bedrock
Unit Weight: 22 kN/m3
110
100
90
80
70
-30
-20
-10
0
10
20
30



Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
Figure 9 - Section D - Existing Conditions - Seismic Conditions
1.267
Limit of Hazard Lands
13.0
Toe Erosion Access Allowance
6.0
Toe Erosion Allowance
7.0
Top of Slope
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 100 kPa
Grey Silty Clay
Unit Weight: 16 kN/m3
Cohesion: 40 kPa
W
Rideau River
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 degrees
Bedrock
Unit Weight: 22 kN/m3
110
100
90
80
70
60
-40
-30
-20
-10
0
10
20
30

Photo 1: Existing conditions of western slope face, looking downslope, facing west. (October 16, 2020)

Photo 2: Existing conditions of western slope face, looking upslope, facing east. (October 16, 2020)

Photo 3: Existing conditions of western slope face at SWMP outlet structure, facing southeast. (October 16, 2020)

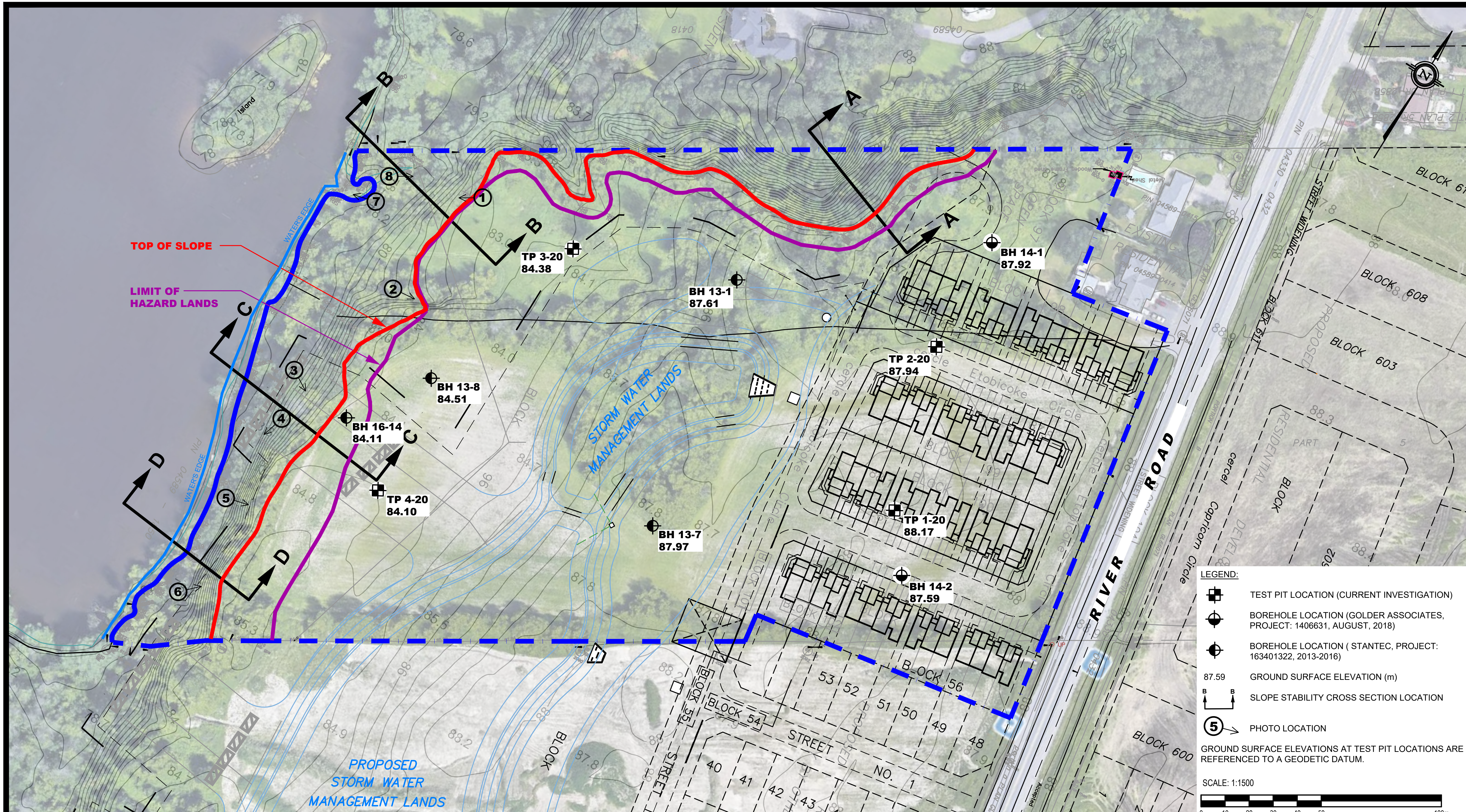
Photo 4: Existing conditions at toe of western slope face, facing southwest. (October 16, 2020)

Photo 5: Existing conditions of western slope face, looking upslope, facing east. (October 16, 2020)

Photo 6: Existing conditions of western slope face, looking upslope, facing east. (October 16, 2020)

Photo 7: Existing Conditions of northern ravine at the western portion of the subject site, facing west.

Photo 8: Existing conditions of northern ravine at the western portion of the subject site, facing east. (October 16, 2020)



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

NO.	REVISIONS	DATE	INITIAL

URBANDALE CORPORATION

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
RIVERSIDE SOUTH - PHASE 12, 750 RIVER ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1500	Date: 11/2020
Drawn by: YA	Report No.: PG3320-3
Checked by: KP	Dwg. No.: PG3320-1
Approved by: DJG	Revision No.:

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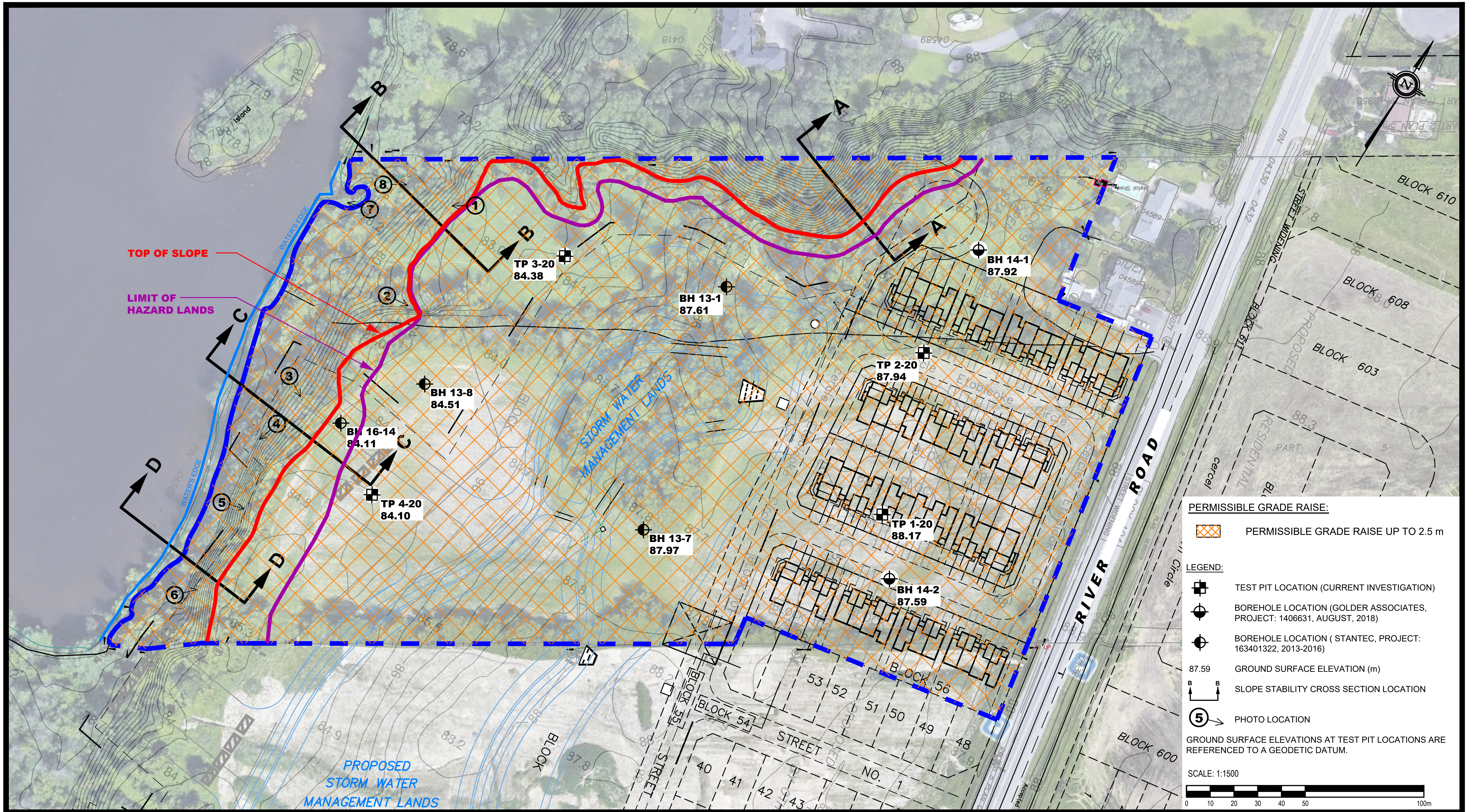

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Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

URBANDALE CORPORATION
SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
RIVERSIDE SOUTH - PHASE 12, 750 RIVER ROAD
OTTAWA, ONTARIO

Title:
PERMISSIBLE GRADE RAISE PLAN

Scale: 1:1500	Date: 11/2020
Drawn by: YA	Report No.: PG3320-3
Checked by: KP	Dwg. No.: PG3320-2
Approved by: DJG	Revision No.:

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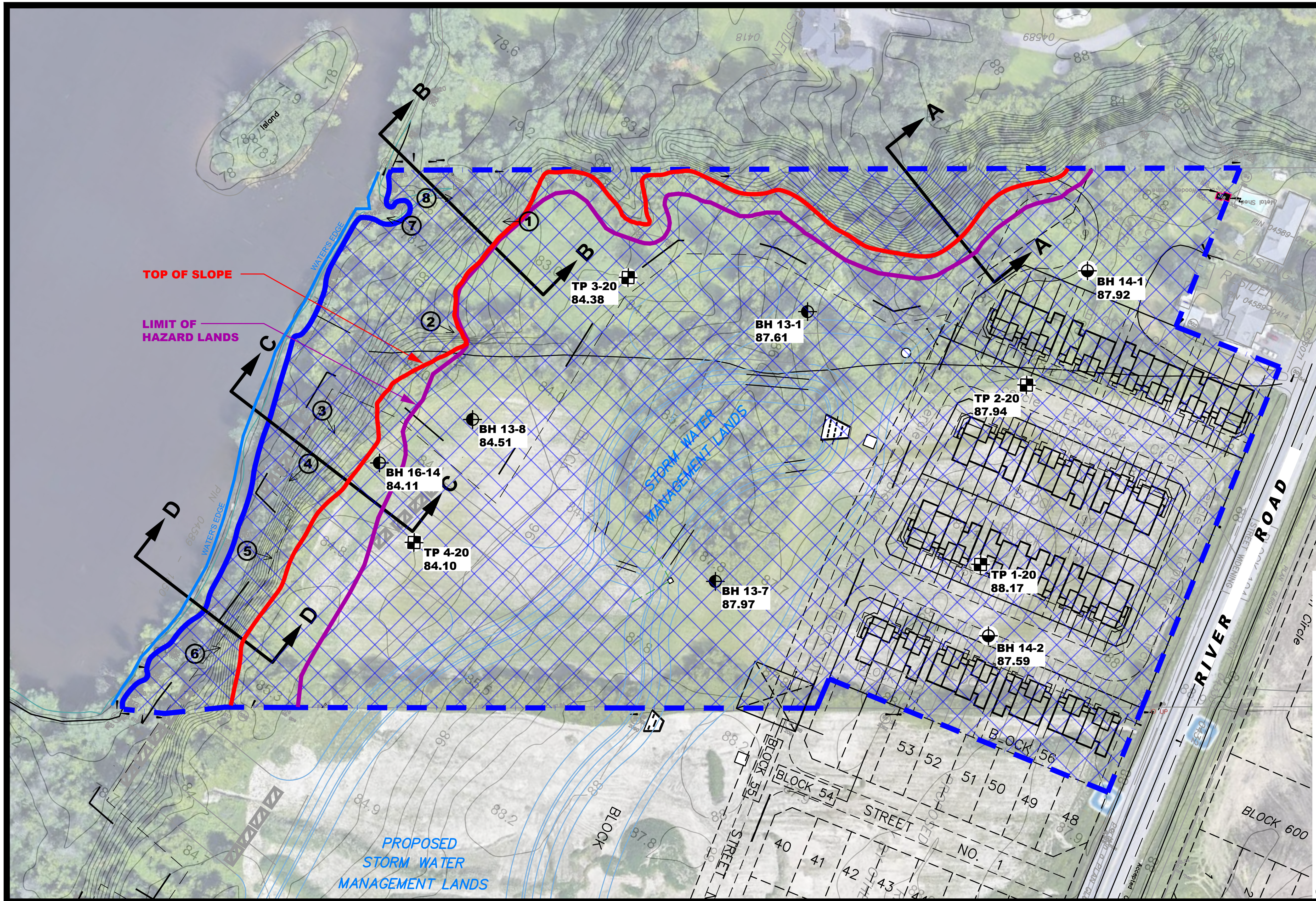

TREE PLANTING SETBACK:

LOW TO MEDIUM SENSITIVITY AREA

LEGEND:

TEST PIT LOCATION (CURRENT INVESTIGATION)

BOREHOLE LOCATION (GOLDER ASSOCIATES, PROJECT: 1406631, AUGUST, 2018)

BOREHOLE LOCATION (STANTEC, PROJECT: 163401322, 2013-2016)

87.59 GROUND SURFACE ELEVATION (m)

SLOPE STABILITY CROSS SECTION LOCATION

5 PHOTO LOCATION

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

SCALE: 1:1500

0 10 20 30 40 50 100m

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

NO.	REVISIONS	DATE	INITIAL

URBANDALE CORPORATION

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
RIVERSIDE SOUTH - PHASE 12, 750 RIVER ROAD

OTTAWA, ONTARIO

Title:

TREE PLANTING SETBACK RECOMMENDATIONS

Scale: 1:1500	Date: 11/2020
Drawn by: YA	Report No.: PG3320-3
Checked by: KP	Dwg. No.: PG3320-3
Approved by: DJG	Revision No.:

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