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Geotechnical Investigation

Proposed Development
39 Carss Street - Almonte

Prepared For

Mr. Patrick Ashby

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Report: PG5402-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Mr. Patrick Ashby to conduct a geotechnical investigation for the proposed development to be located at 39 Carss Street in the Town of Almonte (refer to Figure 1 - Key Plan presented in Appendix 2). The objective of the investigation was to:

☐ determine the subsurface soil and groundwater conditions by means of boreholes and monitoring well program.

☐ provide geotechnical recommendations for the design of the proposed development based on the results of the test holes and other soil information available.

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

2.0 Proposed Development

Although final design details were not available during the preparation of this report, it is understood based on preliminary information that the proposed development will consist of two multi-storey buildings with up to one basement level, as well as several low-rise buildings of slab-on-grade construction. Associated access roads, parking areas and landscaped areas are also anticipated at the subject site.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried out from June 26 to 30, 2020. At that time, 8 boreholes and 9 test pits were extended to a maximum depth of 5.5 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site, giving consideration to the location of the proposed structures. The test hole locations for the current investigation are presented on Drawing PG5402-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a portable drill rig as well as a track-mounted auger drill rig operated by a two person crew and the test pits were excavated using a hydraulic shovel. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All the samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at boreholes BH 5 and BH 7. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Groundwater levels were observed within the test pits which were left open for a period of three days subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel with respect to a geodetic datum. Borehole locations and ground surface elevations at the borehole locations are presented on Drawing PG5402-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples recovered from our field investigation were visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after report completion, unless otherwise directed.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was 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 further in Subsection 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a low-rise residential home located within the central portion of the site. The southern and southwestern portions of the subject site are occupied by landscaped areas and mature trees. The northern and eastern portions of the subject site are generally densely tree covered with the exception of 2 gravel-surfaced access roads, which run from the residential home to the northern and eastern limits of the property.

The subject site is bordered to the north by low-rise residential buildings, to the east by the Ottawa Valley Rail Trail, to the west by the Mississippi River and to the south by an inlet of the Mississippi River as well as densely treed areas. The existing ground surface across the subject site generally slopes down from the east to west at approximate geodetic elevation 123 to 102 m.

4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consists of an approximate 100 to 300 mm thick topsoil layer underlain by either a hard brown silty clay crust or a 0.2 to 1.2 m thick fill layer. Where encountered, the fill material was generally observed to consist of a brown silty clay with varying amounts of gravel and cobbles.

The very stiff to stiff brown silty clay was observed either underlying the topsoil layer or the fill material in all test holes. Underlying the brown silty clay crust, a firm grey silty clay was encountered at an approximate depth of 4.7 m below the existing ground surface at borehole BH 3.

A glacial till deposit was observed underlying the silty clay deposit at borehole BH 2 at an approximate depth of 1.8 m below the existing ground surface. The glacial till was noted to consist of a compact brown silty sand with gravel.

Practical refusal to the DCPT was encountered at depths of 8.5 m and 11.1 m in boreholes BH 5 and BH 7, respectively.

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

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of interbedded limestone and dolomite of the Gull River formation with an overburden thickness of 0 to 10 m.

4.3 Groundwater

Based on our field observations, the long-term groundwater table was not encountered at a majority of the subject site. This is likely due to the elevation difference across the subject site in combination with the proximity of the site to the Mississippi River

Long-term groundwater level can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected at an approximate geodetic elevation of 97 to 99 m. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is anticipated that the proposed multi-storey buildings with up to one underground level will be founded on one of the following:

☐ Conventional shallow footings bearing on an undisturbed, stiff silty clay bearing surface.

☐ A raft foundation bearing on an undisturbed stiff silty clay bearing surface.

Further, it is recommended that the proposed low-rise buildings be founded using conventional shallow footings placed on an undisturbed stiff silty clay bearing surface.

It is recommended that any portions of the underground parking levels which extend beyond the overlying building footprint be supported on conventional shallow footings bearing on the undisturbed, silty clay bearing surface.

Due to the presence of a deep silty clay deposit, a permissible grade raise restriction is required for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings and other settlement sensitive structures.

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 thickness 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 material's 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.

Protection of Subgrade (Raft Foundation)

If applicable, the subgrade material of the raft foundation will most likely consist of a silty clay deposit, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mudslab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance due to drying.

5.3 Foundation Design

Conventional Spread Footings

Pad footings, up to 7 m wide, and strip footings up to 4 m wide, placed on an undisturbed, very stiff to stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The 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.

Raft Foundation

As noted above, a raft foundation may be required to support the proposed multi-storey buildings with up to one basement level. For our design calculations, one level of underground parking was assumed which would extend 3 to 4 m below existing ground surface. The maximum SLS contact pressure is **200 kPa** for a raft foundation bearing on the undisturbed, hard to stiff silty clay. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **350 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **8.0 MPa/m** for a contact pressure of **200 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed building can be designed using the above parameters with total and differential settlements of 25 and 20 mm, respectively.

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 a silty clay bearing 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

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2 m** is recommended within 5 m of the proposed buildings. A permissible grade raise restriction of **3 m** is recommended in the parking areas and access lanes. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

A seismic site response **Class D** should be used for design of the proposed buildings at the subject site according to the OBC 2012. The soils underlying the site are not susceptible to liquefaction.

5.5 Slab on Grade Construction/Basement Slab

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the undisturbed, stiff silty clay subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade or basement slab construction.

It is recommended that the upper 200 mm of sub-floor fill consist of OPSS Granular A crushed stone. All backfill materials required to raise grade within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

If a raft slab is considered for the multi-storey buildings, a granular layer of OPSS Granular A crushed stone will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any. This is discussed further in Subsection 6.1.

5.6 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 of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) 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 will only be applicable for static analyses and should not be used 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 total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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 earth force component (P_o) under seismic conditions can be calculated using

$P_o = .5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

The earth forces calculated are unfactored. For the ULS case, the earth 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 parking areas and access lanes.

Table 1 - 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 fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 2 - Recommended Pavement Structure - Acces Lanes

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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

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 a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed buildings. The system should consist of a 150 mm diameter perforated, corrugated plastic pipe surrounded on all sides by 150 mm of 19 mm clear crushed stone which is placed at the footing level around the exterior perimeter of the structures. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Raft Slab Construction

If applicable, it is expected that the raft slab will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the raft slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 150 mm diameter perforated pipes be placed at approximate 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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 materials will be frost susceptible and, as such, are not recommended for placement as backfill against the foundation walls unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or Miradrain G100N. Imported granular materials, such as clean sand or OPSS Granular B Type I granular materials, should be placed for this purpose.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover should be provided for adequate frost protection of heated structured.

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 heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

The proposed underground parking levels may require protection against frost action depending on the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

The side slopes of the 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.

Unsupported Excavations

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 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.

Temporary Shoring

Temporary shoring may be required to support the overburden soils. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 3 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A crushed stone should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 98% of the material's SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface ares are considered above the trench backfill, the trench backfill material within the frost zone, (about 1.5 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick lifts and compacted to 95% of the materials SPMDD.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low to moderate 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 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 of 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.

6.6 Winter Construction

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

The subsoil 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.

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 carried out in a manner to avoid the introduction of frozen materials into the trenches. 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. 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 Limit of Hazard Lands

The slope condition of three existing slope faces were reviewed by Paterson field personnel as part of the geotechnical investigation. Four slope cross-sections were studied for global stability as the worst case scenarios. The cross-section locations are presented on Drawing PG5402-1 - Test Hole Location Plan attached to the current report. Photographs taken during our site visit on June 17, 2020 of the slope conditions are presented in Appendix 2 of our report.

The existing slope face fronting onto the Mississippi River, which runs along the west and southwest boundaries of the property, is currently vegetated with signs of minor active erosion, such as sloughing and oversteepening of the bank face. Based on our review, the subject slope is considered to be the valley corridor wall for the Mississippi River and our Limit of Hazard Lands designation line has been determined for the subject slope (Section D).

The other two slopes located within the subject site have been assessed for slope stability (Sections A, B, C). However, a limit of hazard lands designation has not been applied to these two slopes due to the separation from the watercourse and toe elevation of the slopes being above the 1:100 year flood line for the Mississippi River. The slopes were noted to be vegetated throughout with no signs of slope instability. The slope located along the southeast property boundary was also observed to be densely treed.

Slope Stability Analysis

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 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 to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to the 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 the risks of failure are acceptable.

A minimum factor of safety of 1.5 is generally recommended for the 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, realistic groundwater flow conditions. Subsoil conditions at the cross-sections were inferred based on nearby boreholes and test pits as well as general knowledge of the area's geology.

The effective strength soil parameters used for static analyses were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 4 below.

Table 4 - Effective Strength Soil and Material Parameters

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay Crust	17	33	12
Glacial Till	20	35	1

The total strength of the parameters for seismic analysis were chosen based on the in silty, undrained shear strengths recovered within the boreholeds 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 5 below.

Table 5 - Total Strength 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	0	100
Glacial Till	20	35	1

Static Loading Analysis

The results for the slope stability analyses under existing, static conditions at Sections A, B, C, and D are shown on Figures 2, 4, 6, and 8 and are attached to the present report. The factor of safety was found to be greater than 1.5 at all sections.

Seismic Loading Analysis

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g (50% of PGA = 0.32g) 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 at Sections A, B, C, and D 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 geotechnical setback from the top of the slope is required to achieve a factor of safety of 1.1 for the limit of the hazard lands.

Geotechnical Setback - Limit of Hazard Lands

Signs of erosion were noted along the lower portion of the valley corridor slope wall that confines the Mississippi River along the west and southwest boundary of the site. Some minor sloughing failures were noted in the upper portion of the slope, leaving some exposed tree roots. Based on our observations, a 5 m toe erosion allowance is deemed appropriate for the slope based on the cohesive nature of the soils, the observed erosion areas and the current watercourse depth and width. It is considered that a toe erosion allowance of 5 m and an erosion allowance of 6 m is required from the top of slope.

The limit of hazard lands, which include the allowances, are indicated on Drawing PG5402-1 - Test Hole Location Plan attached to the present report.

6.8 Corrosion Potential and Sulphate

One (1) sample was submitted for testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate 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.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- 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.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to placing backfilling materials.
- Field density tests to ensure 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 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 made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole log are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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

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 Mr. Patrick Ashby or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
August 26, 2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Kevin A. Pickard, EIT

David J. Gilbert, P.Eng

Report Distribution:

☐ Mr. Patrick Ashby
☐ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
39 Carrs Street
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Portable Drill

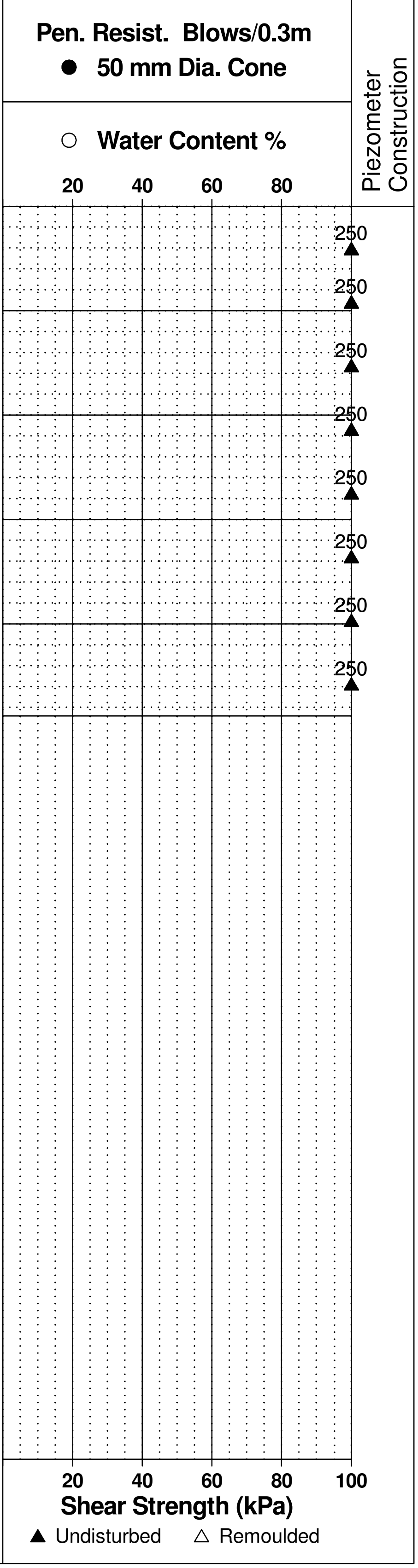
DATE June 26, 2020

FILE NO. PG5402

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.20	AU	1		
Hard, brown **SILTY CLAY,** trace to some sand	SS	2	50	10
	SS	3	75	17
	SS	4	83	19
	SS	5	75	18
	SS	6	100	15
	SS	7	100	14
	SS	8	100	16
4.88	SS	9	100	16
End of Borehole				

DEPTH (m)	ELEV. (m)
0	123.47
1	122.47
2	121.47
3	120.47
4	119.47



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SOIL PROFILE AND TEST DATA

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DATUM Geodetic

REMARKS

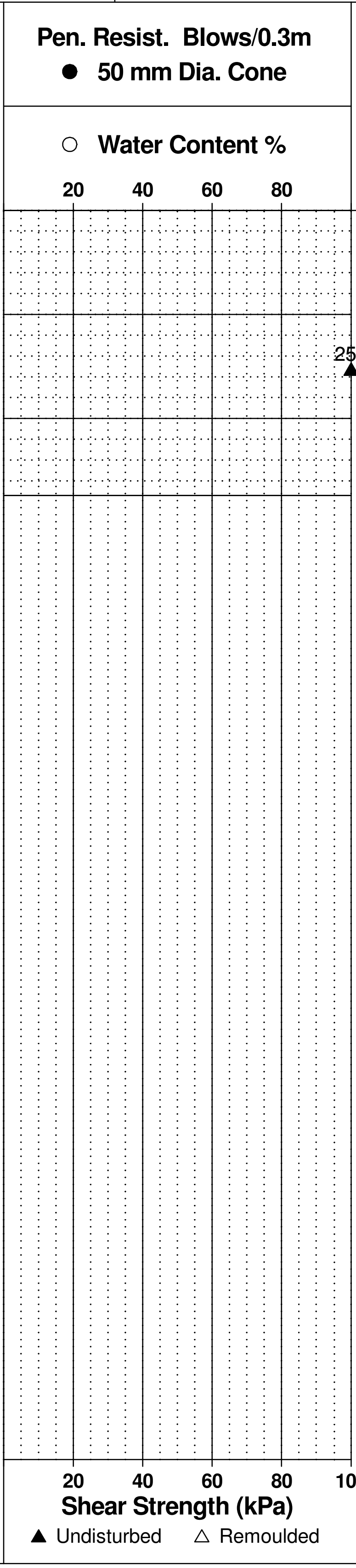
BORINGS BY Portable Drill

DATE June 26, 2020

FILE NO. PG5402

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	115.59
TOPSOIL 0.20		AU	1				
FILL: Dark brown silty sand, some gravel		SS	2	33	10		
1.22		SS	3	4	13	1	114.59
Hard, brown **SILTY CLAY,** some sand seams 1.83		SS	4	67	8		
GLACIAL TILL: Compact, brown silty sand with gravel		SS	5	58	15	2	113.59
2.74		SS	6		50+		
End of Borehole							



Piezometer Construction

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Geotechnical Investigation
39 Carrs Street
Almonte, Ontario

DATUM Geodetic

REMARKS

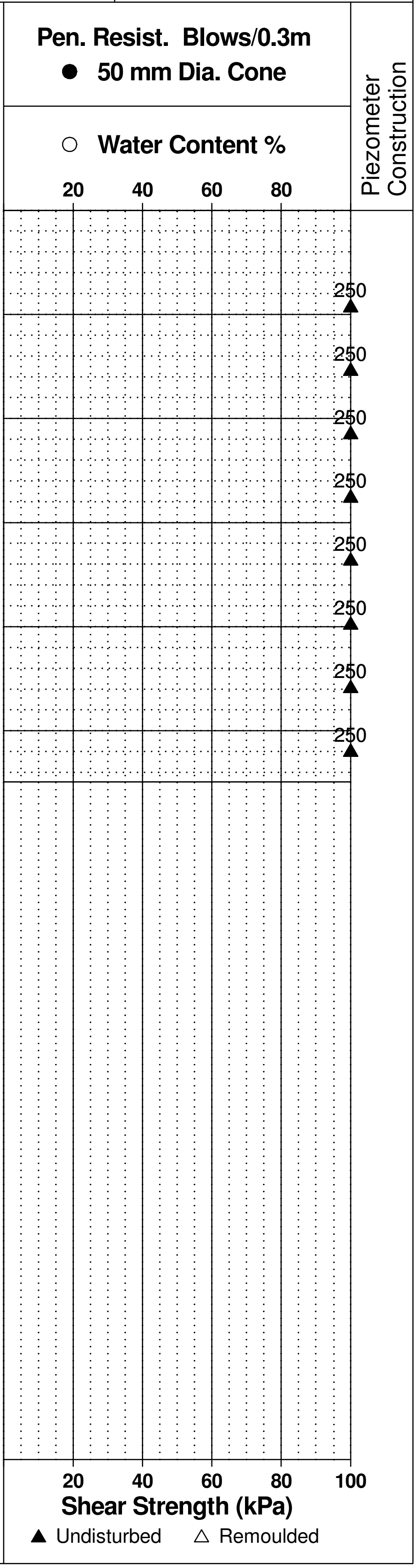
BORINGS BY Portable Drill

DATE June 26, 2020

FILE NO. PG5402

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.70
TOPSOIL 0.20		SS	1	8	16		
		SS	2	33	21	1	102.70
		SS	3	100	28	2	101.70
		SS	4	100	22		
Hard, brown **SILTY CLAY,** trace sand seams		SS	5	100	13	3	100.70
		SS	6	100	15		
		SS	7	100	13	4	99.70
- grey by 4.7m depth		SS	8	100	4	5	98.70
5.49		SS	9		7		
End of Borehole							



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SOIL PROFILE AND TEST DATA

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DATUM Geodetic

REMARKS

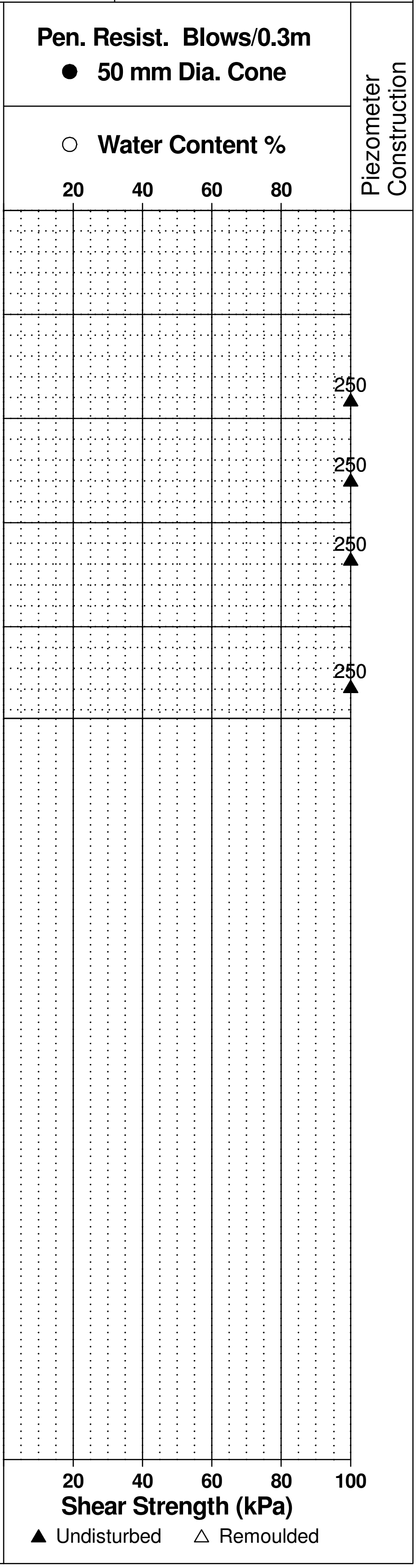
BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

FILE NO. PG5402

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	101.97
TOPSOIL 0.20							
FILL: Brown silty clay, trace gravel and cobbles 0.91		SS	1	25	15		
Hard, brown **SILTY CLAY**		SS	2	50	13	1	100.97
		SS	3	100	17	2	99.97
		SS	4	100	12	3	98.97
		SS	5	100	5	4	97.97
4.88		SS	6		2		
End of Borehole							



Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
39 Carrs Street
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

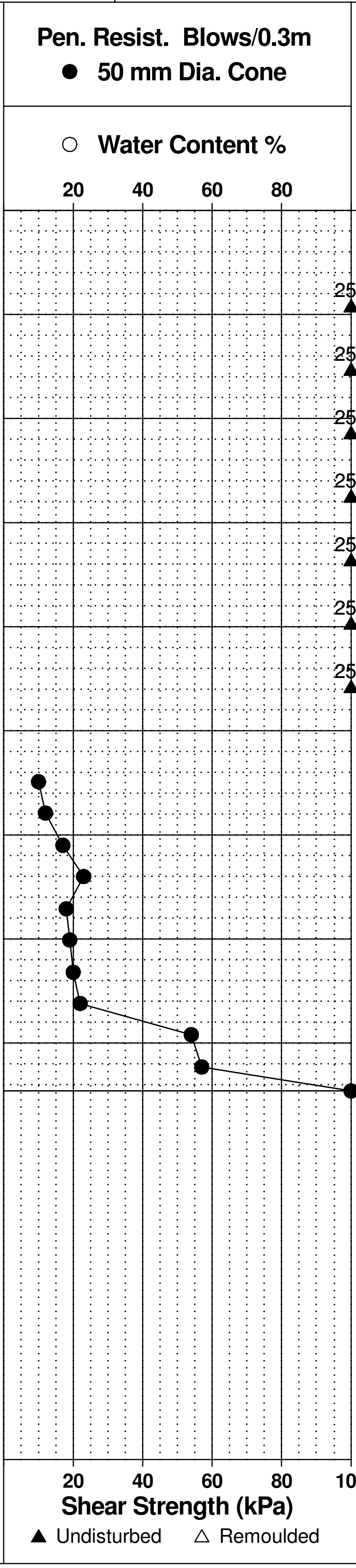
FILE NO. PG5402

HOLE NO. BH 5

SOIL DESCRIPTION	Depth
GROUND SURFACE	
TOPSOIL	0.20
Hard, brown **SILTY CLAY**	4.88
Dynamic Cone Penetration Test commenced at 4.88m depth	8.46
End of Borehole Practical DCPT refusal at 8.46m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	12	13
SS	2	100	11
SS	3	75	18
SS	4	100	21
SS	5	100	24
SS	6	67	20
SS	7	100	18
SS	8	58	20

DEPTH (m)	ELEV. (m)
0	119.32
1	118.32
2	117.32
3	116.32
4	115.32
5	114.32
6	113.32
7	112.32
8	111.32



Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
39 Carrs Street
Almonte, Ontario

DATUM Geodetic

FILE NO. **PG5402**

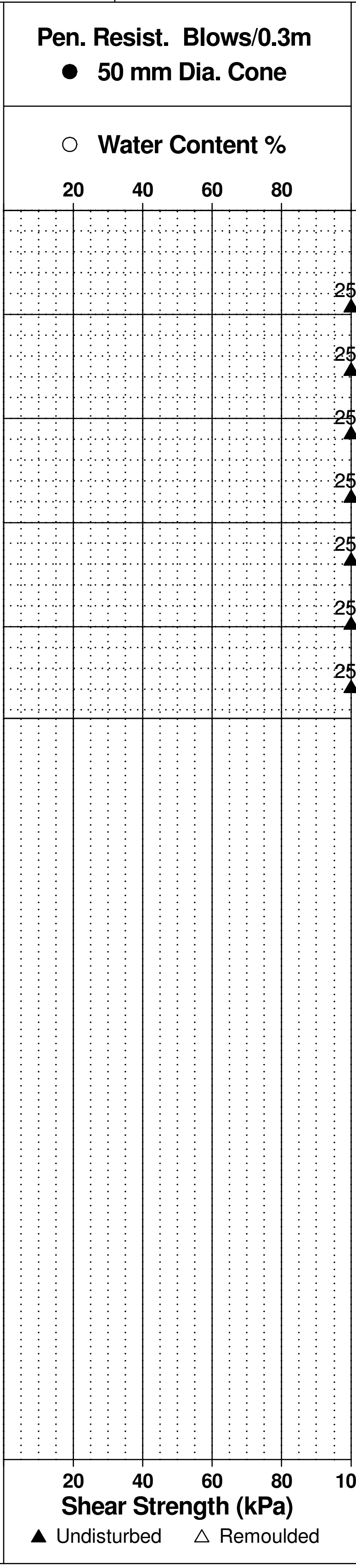
REMARKS

HOLE NO. **BH 6**

BORINGS BY Track-Mount Power Auger **DATE** June 30, 2020

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.18	SS	1	38	9
	SS	2	46	22
	SS	3	92	23
Hard, brown **SILTY CLAY**	SS	4	88	21
	SS	5	79	23
	SS	6	100	23
	SS	7	79	28
4.88	SS	8	79	24
End of Borehole				

DEPTH (m)	ELEV. (m)
0	120.30
1	119.30
2	118.30
3	117.30
4	116.30



Piezometer Construction

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SOIL PROFILE AND TEST DATA

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DATUM Geodetic

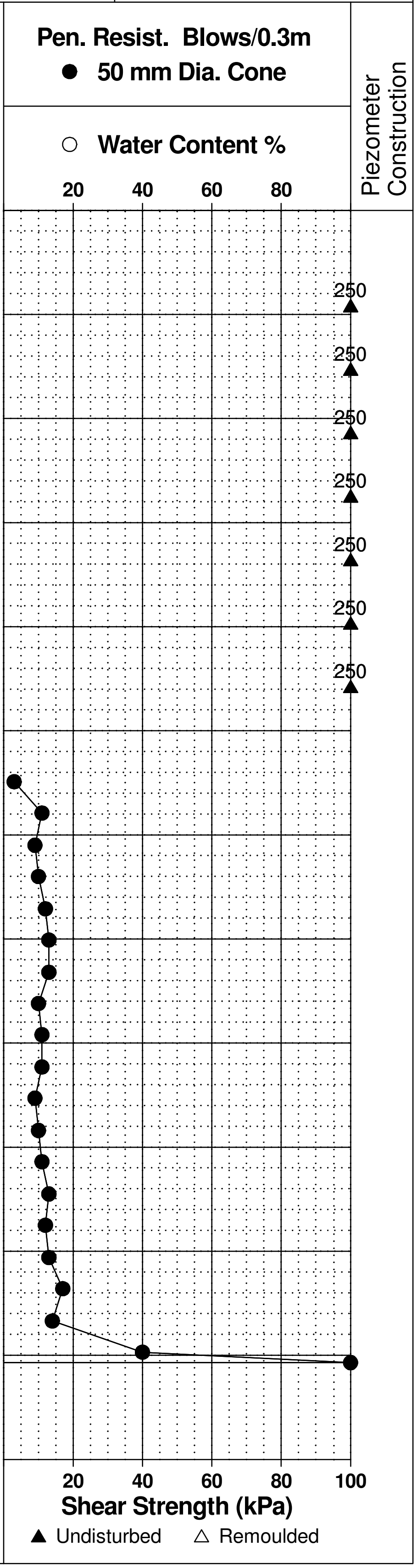
REMARKS

BORINGS BY Track-Mount Power Auger **DATE** June 30, 2020

FILE NO. PG5402

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	121.73
TOPSOIL 0.08		SS	1	58	11		
		SS	2	50	39		
						1	120.73
		SS	3	12	14		
						2	119.73
Hard, brown **SILTY CLAY**		SS	4	100	10		
		SS	5	67	12		
						3	118.73
		SS	6	75	19		
		SS	7	100	21	4	117.73
4.88		SS	8	92	19		
Dynamic Cone Penetration Test commenced at 4.88m depth						5	116.73
						6	115.73
						7	114.73
						8	113.73
						9	112.73
						10	111.73
11.07						11	110.73
End of Borehole Practical DCPT refusal at 11.07m depth							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Almonte, Ontario

DATUM Geodetic

FILE NO. PG5402

REMARKS

HOLE NO. BH 8

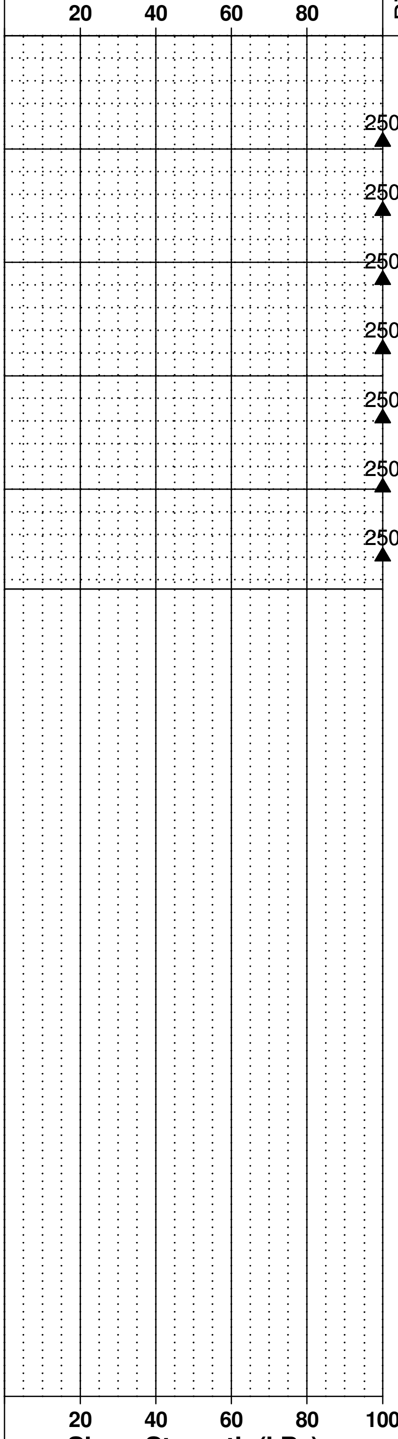
BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	121.63	
TOPSOIL 0.08		SS	1	25	8			
Hard, brown **SILTY CLAY**		SS	2	88	22	1	120.63	
		SS	3	100	29			
		SS	4	100	21	2	119.63	
		SS	5	100	18			
		SS	6	88	20	3	118.63	
		SS	7	100	20	4	117.63	
4.88		SS	8	100	21			
End of Borehole								

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

○ **Water Content %**



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
39 Carrs Street
Almonte, Ontario

DATUM Geodetic

FILE NO. PG5402

REMARKS

HOLE NO. TP 1

BORINGS BY Backhoe

DATE June 26, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	103.68		
TOPSOIL 0.15	■								
FILL: Brown silty clay, trace gravel and cobbles 0.50	▩								
Brown **SILTY CLAY**	▨					1	102.68		
	▨					2	101.68		
2.30	▨								
End of Test Pit									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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REMARKS

BORINGS BY Backhoe

DATE June 26, 2020

FILE NO. PG5402

HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	101.92		
TOPSOIL 0.18	■								
FILL: Brown silty clay, trace gravel and cobbles 0.40	▨								
Brown **SILTY CLAY** 1.80	▨					1	100.92		
End of Test Pit									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Almonte, Ontario

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FILE NO. PG5402

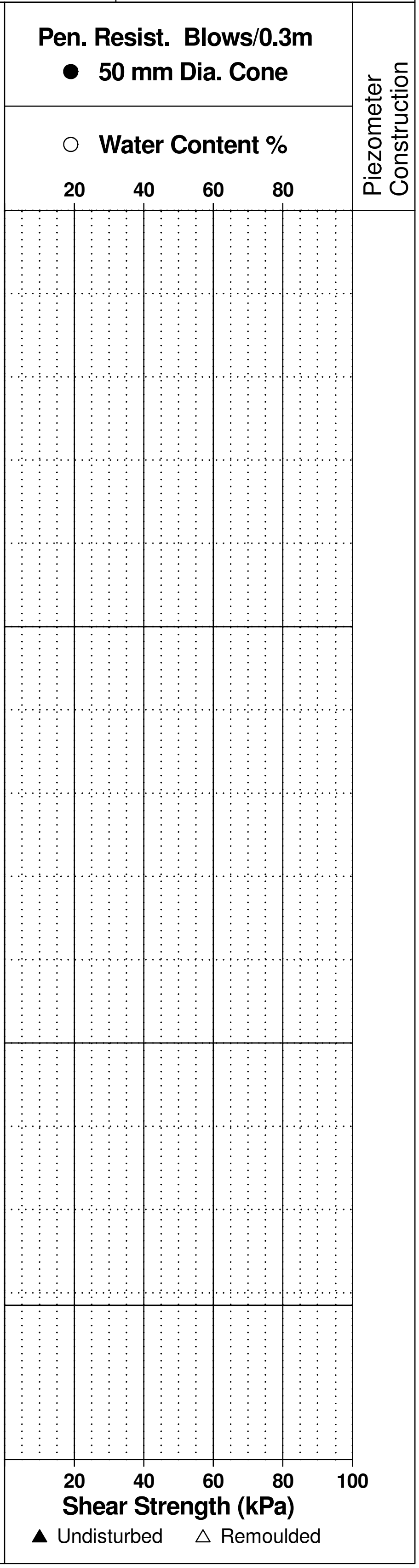
REMARKS

HOLE NO. TP 3

BORINGS BY Backhoe

DATE June 26, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	113.01
TOPSOIL 0.12	■						
Brown **SILTY CLAY**	▨					1	112.01
	▨					2	111.01
2.63							
End of Test Pit TP terminated on inferred bedrock surface at 2.63m depth.							



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SOIL PROFILE AND TEST DATA

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Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE June 26, 2020

FILE NO. PG5402

HOLE NO. TP 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	117.26		
TOPSOIL 0.14	■								
Brown **SILTY CLAY**	▨					1	116.26		
2.15	▨					2	115.26		
End of Test Pit									

Pen. Resist. / Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Almonte, Ontario

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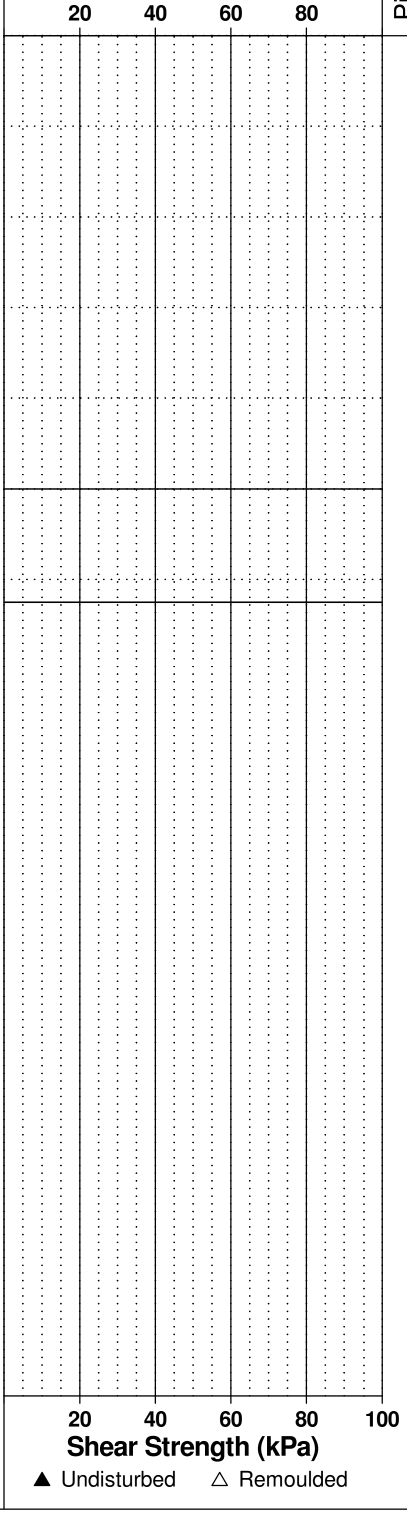
REMARKS
BORINGS BY Backhoe
DATE June 26, 2020
FILE NO. PG5402
HOLE NO. TP 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	123.29		
TOPSOIL 0.18	■								
Brown **SILTY CLAY** 1.75	▨					1	122.29		
End of Test Pit									



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SOIL PROFILE AND TEST DATA

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DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE June 26, 2020

FILE NO. PG5402

HOLE NO. TP 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	121.66		
TOPSOIL 0.21	■								
Brown **SILTY CLAY** 1.93	▨					1	120.66		
End of Test Pit									

Water Content % axis: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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FILE NO. PG5402

REMARKS

HOLE NO. TP 7

BORINGS BY Backhoe

DATE June 26, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	122.52		
TOPSOIL 0.26	■								
Brown **SILTY CLAY** 0.84	▨								
End of Test Pit									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
39 Carrs Street
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FILE NO. PG5402

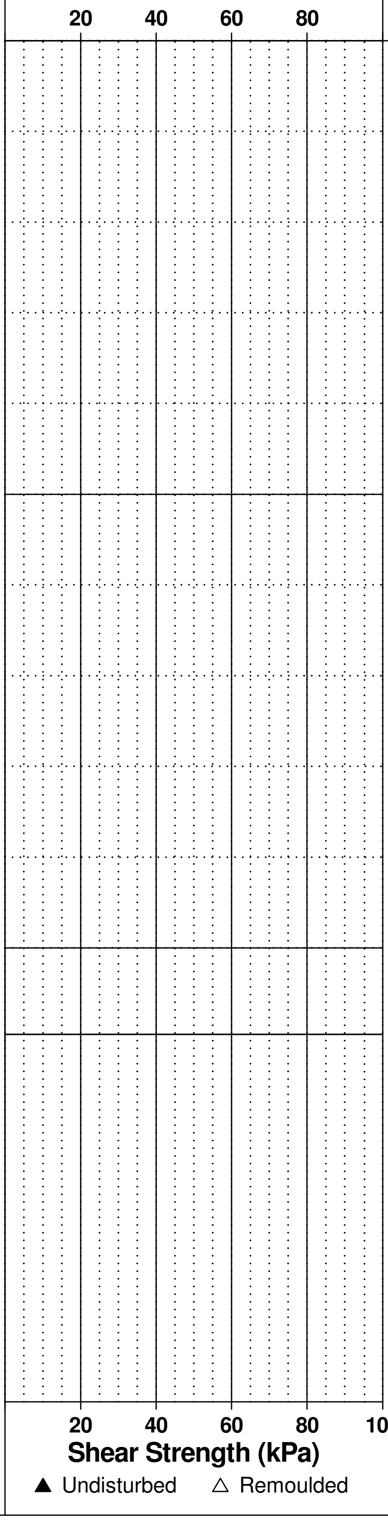
REMARKS

HOLE NO. TP 8

BORINGS BY Backhoe

DATE June 26, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	119.06		
TOPSOIL 0.20	■								
Brown **SILTY CLAY**	▨					1	118.06		
2.19						2	117.06		
End of Test Pit									



SOIL PROFILE AND TEST DATA

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39 Carrs Street
Almonte, Ontario

DATUM Geodetic

REMARKS

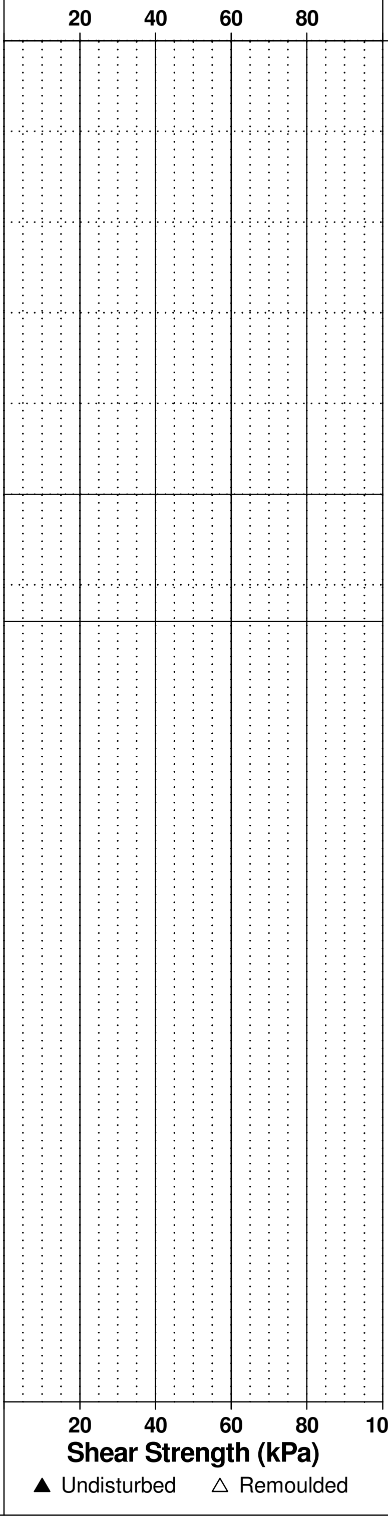
BORINGS BY Backhoe

DATE June 26, 2020

FILE NO. PG5402

HOLE NO. TP 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	120.70		
TOPSOIL	■								
0.27									
Brown **SILTY CLAY**	▨					1	119.70		
1.28									
End of Test Pit									



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

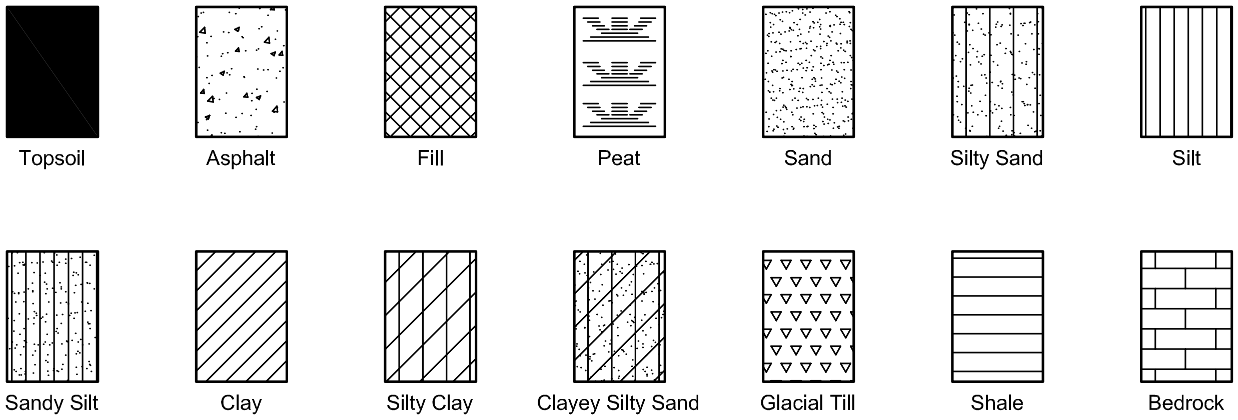
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

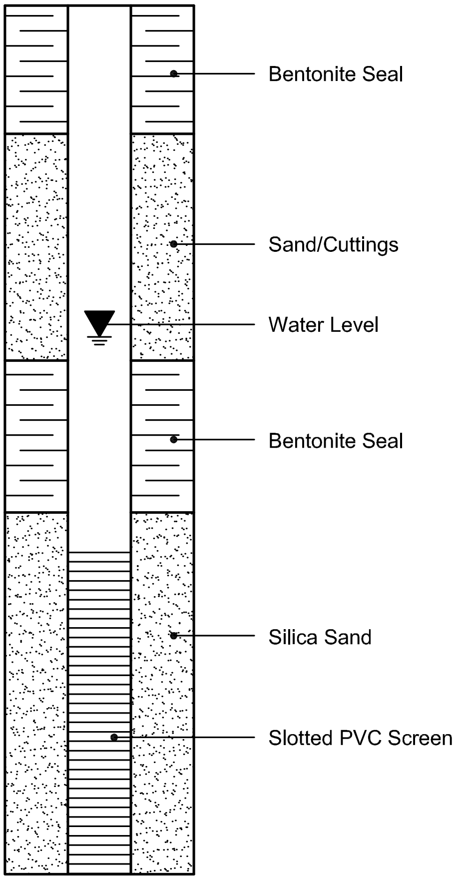
SYMBOLS AND TERMS (continued)

STRATA PLOT

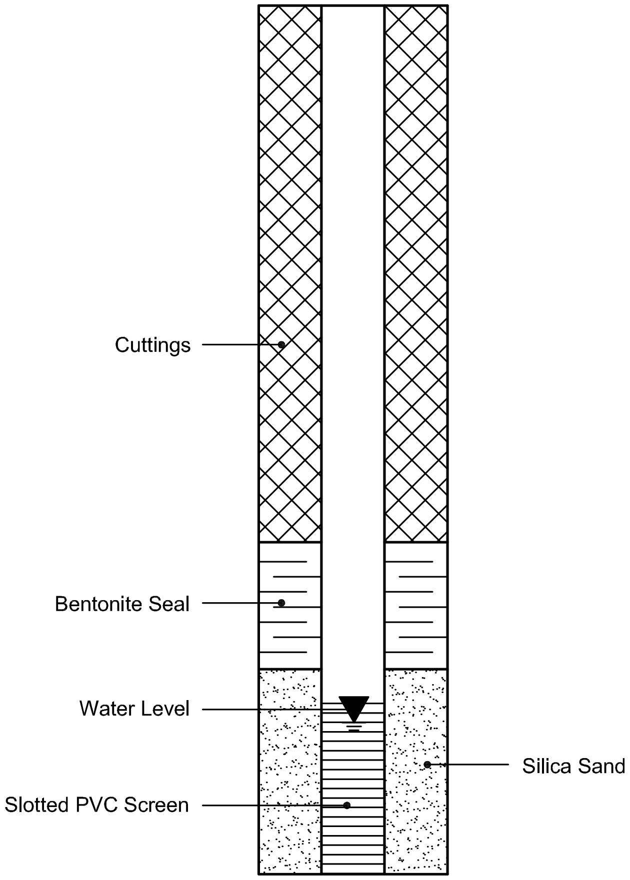


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 30469

Report Date: 31-Jul-2020

Order Date: 27-Jul-2020

Project Description: PG5402

Client ID:		BH5-SS4	-	-	-
Sample Date:		30-Jun-20 09:00	-	-	-
Sample ID:		2031080-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	79.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.80	-	-	-
Resistivity	0.10 Ohm.m	67.4	-	-	-
Anions					
Chloride	5 ug/g dry	15	-	-	-
Sulphate	5 ug/g dry	9	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 9 - SLOPE STABILITY ANALYSIS SECTIONS

PHOTOGRAPHS FROM SITE VISIT

DRAWING PG5402-1 - TEST HOLE LOCATION PLAN

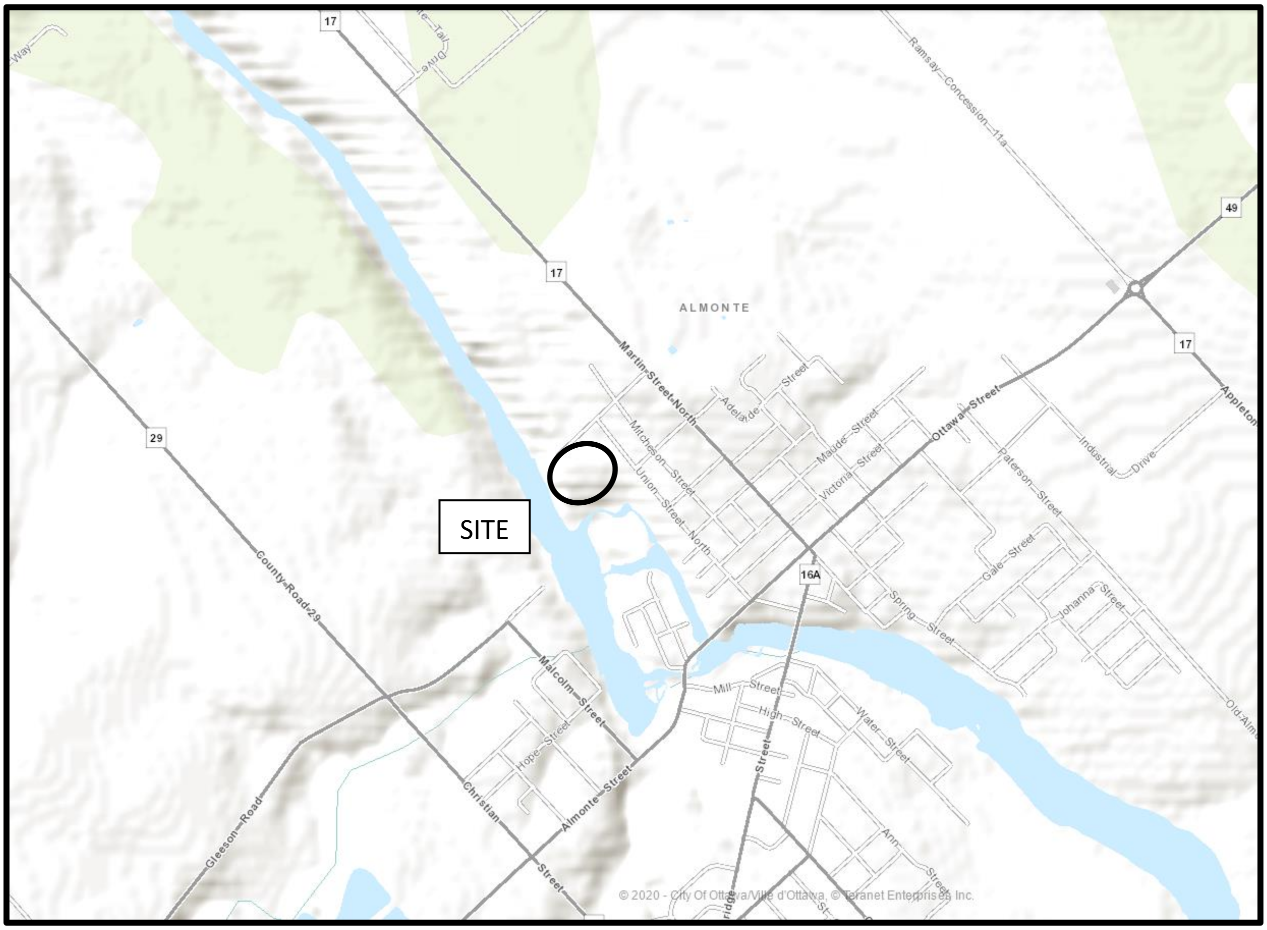


FIGURE 1

KEY PLAN

patersongroup

Figure 2 - Section A - Existing Conditions - Static Analysis

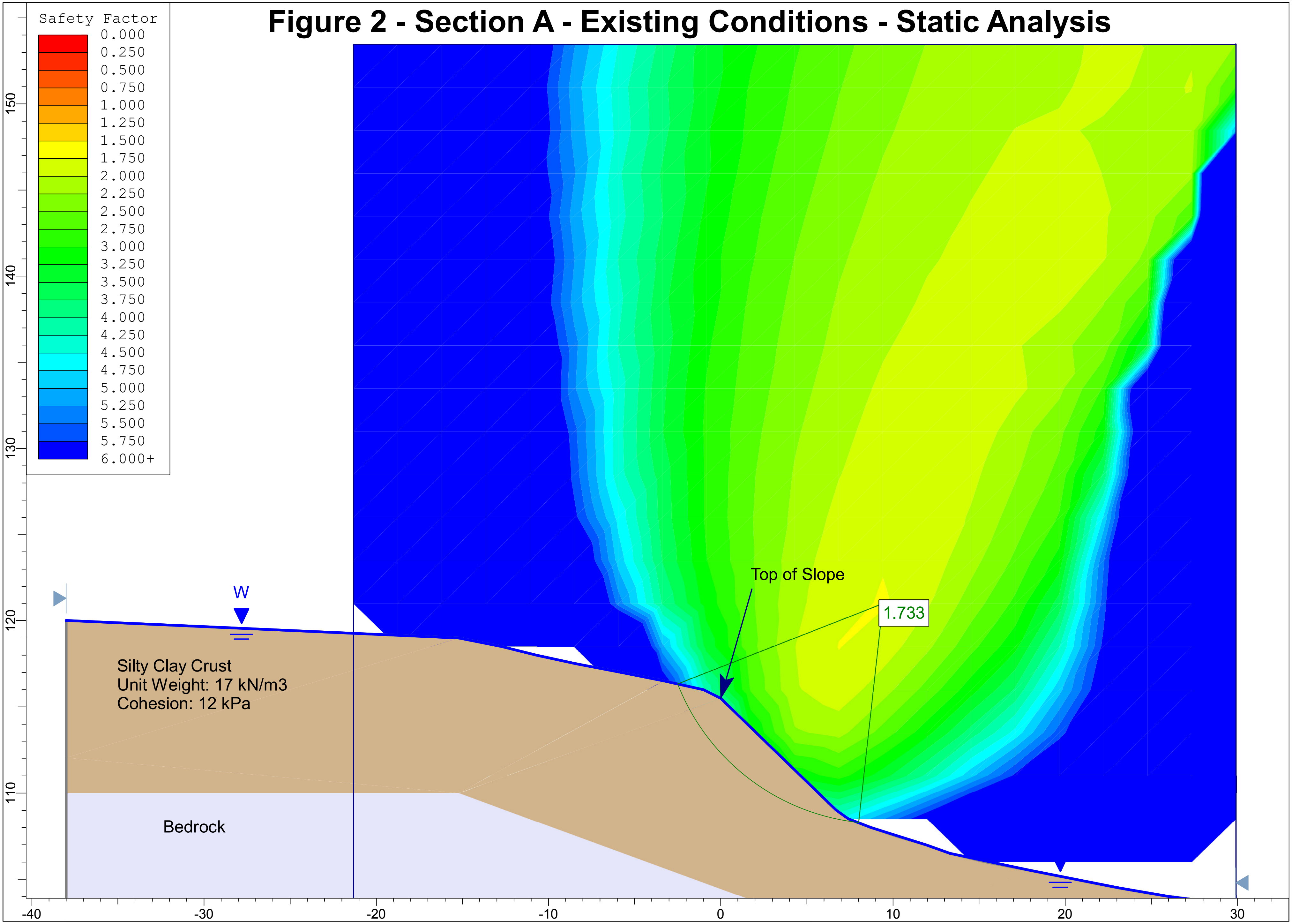
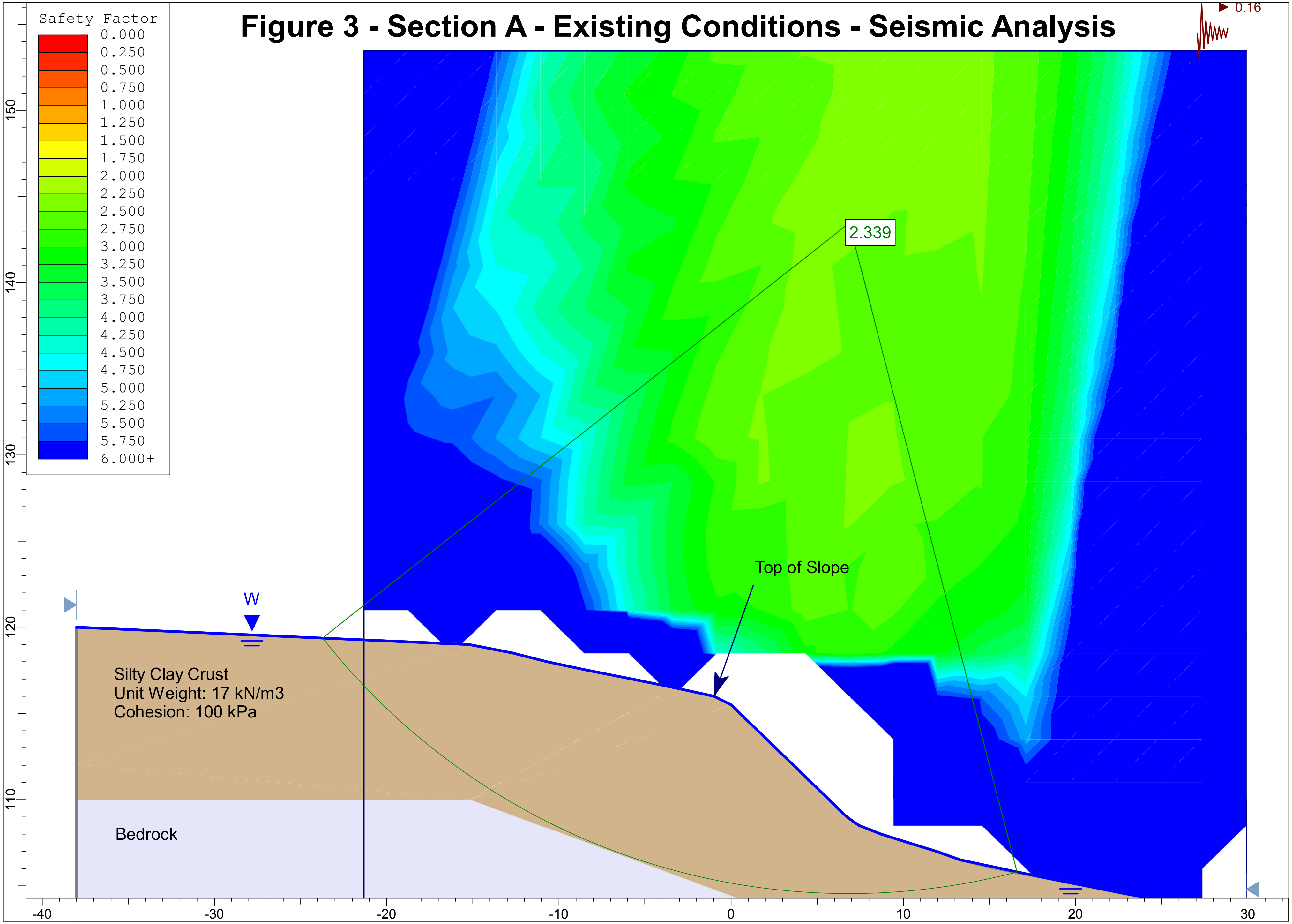


Figure 3 - Section A - Existing Conditions - Seismic Analysis



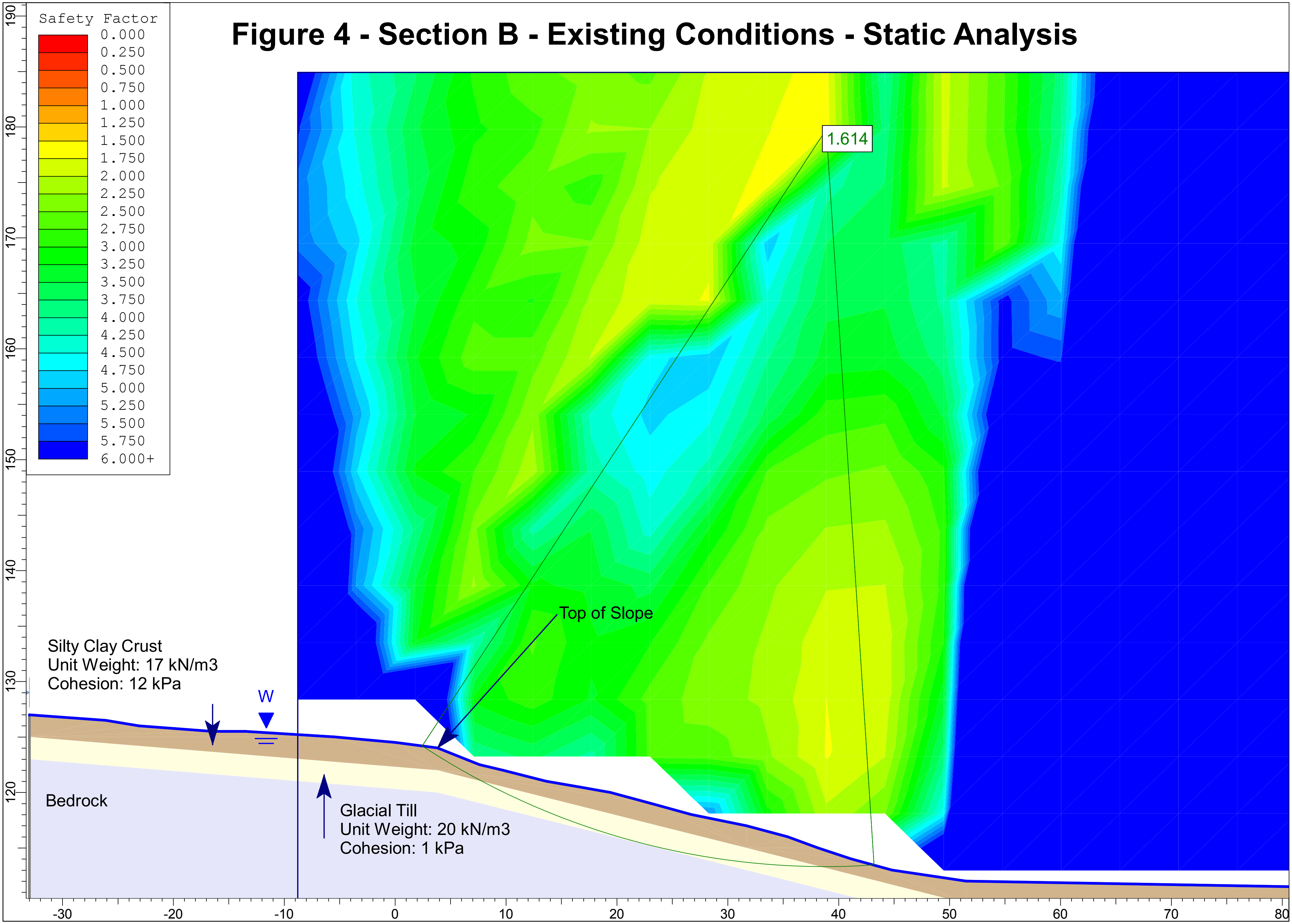


Figure 4 - Section B - Existing Conditions - Static Analysis

Figure 5 - Section B - Existing Conditions - Seismic Analysis

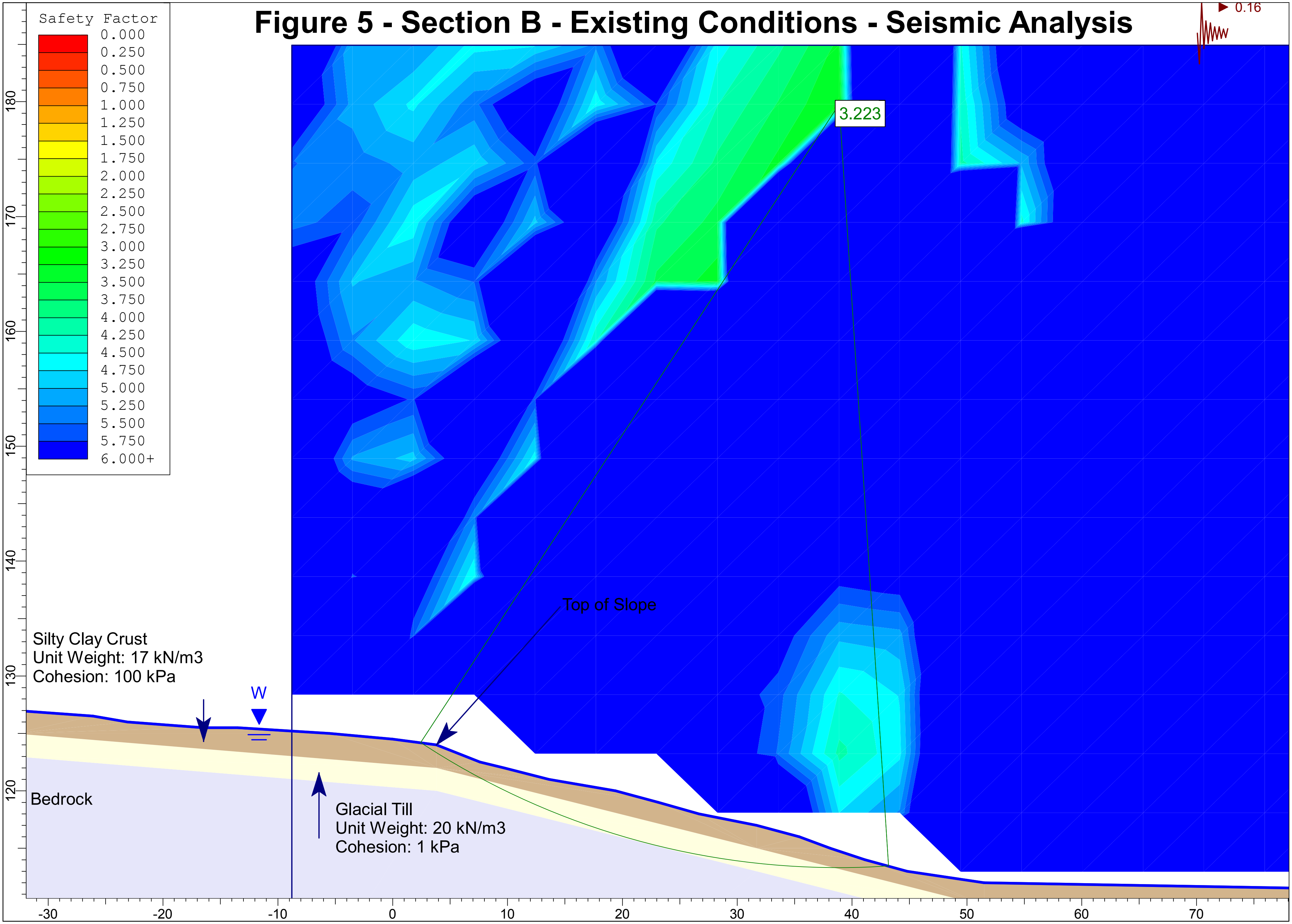
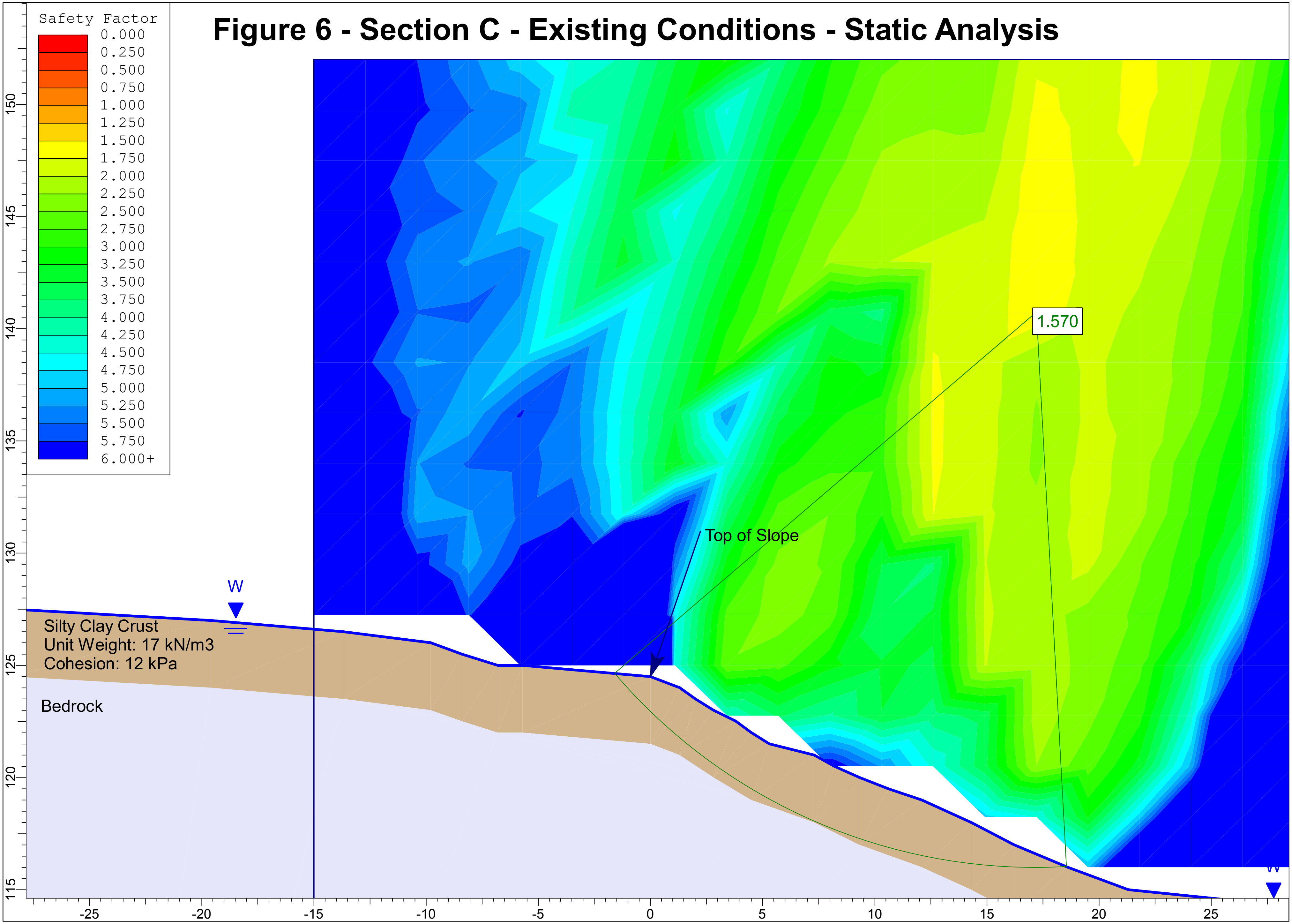


Figure 6 - Section C - Existing Conditions - Static Analysis



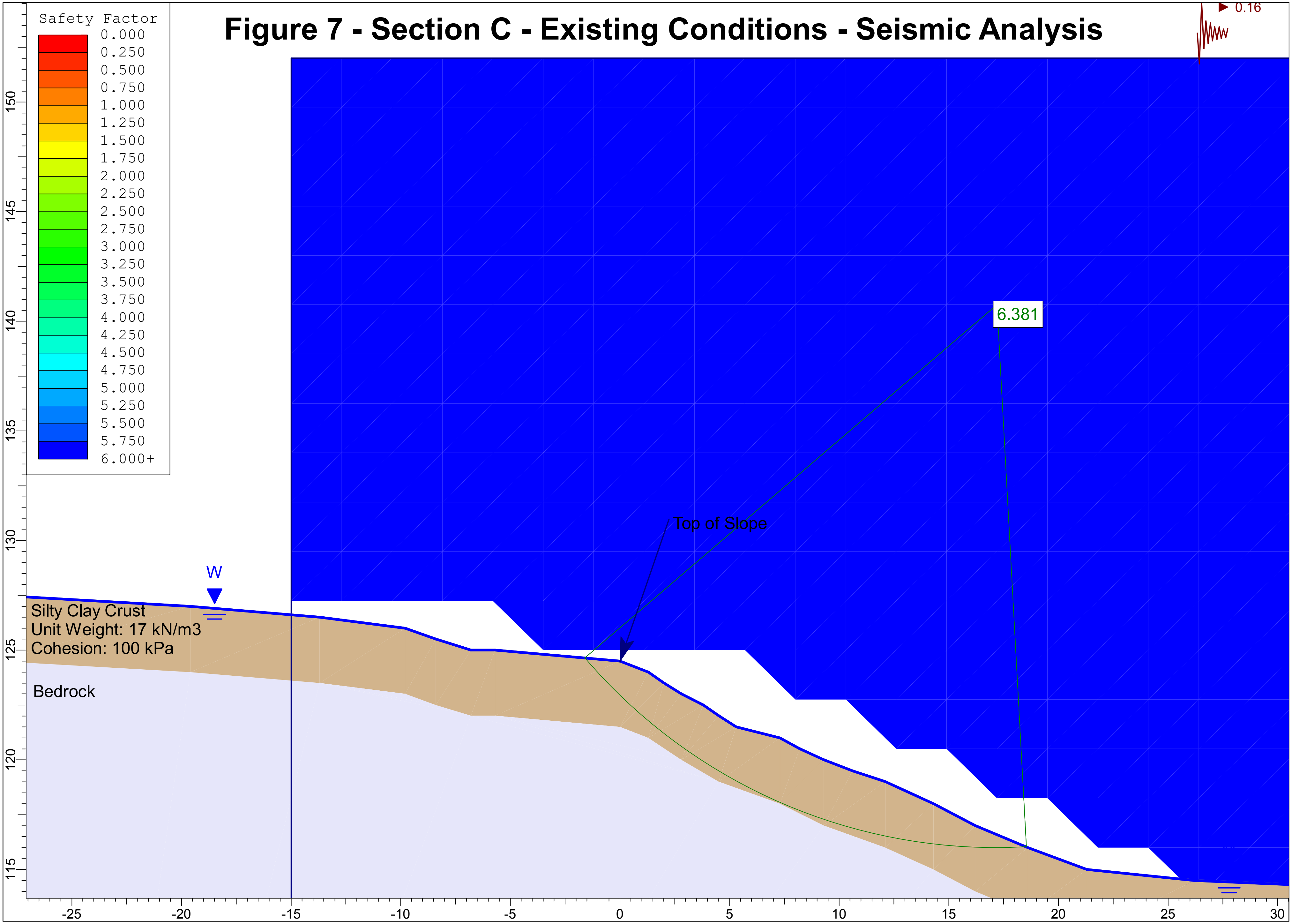
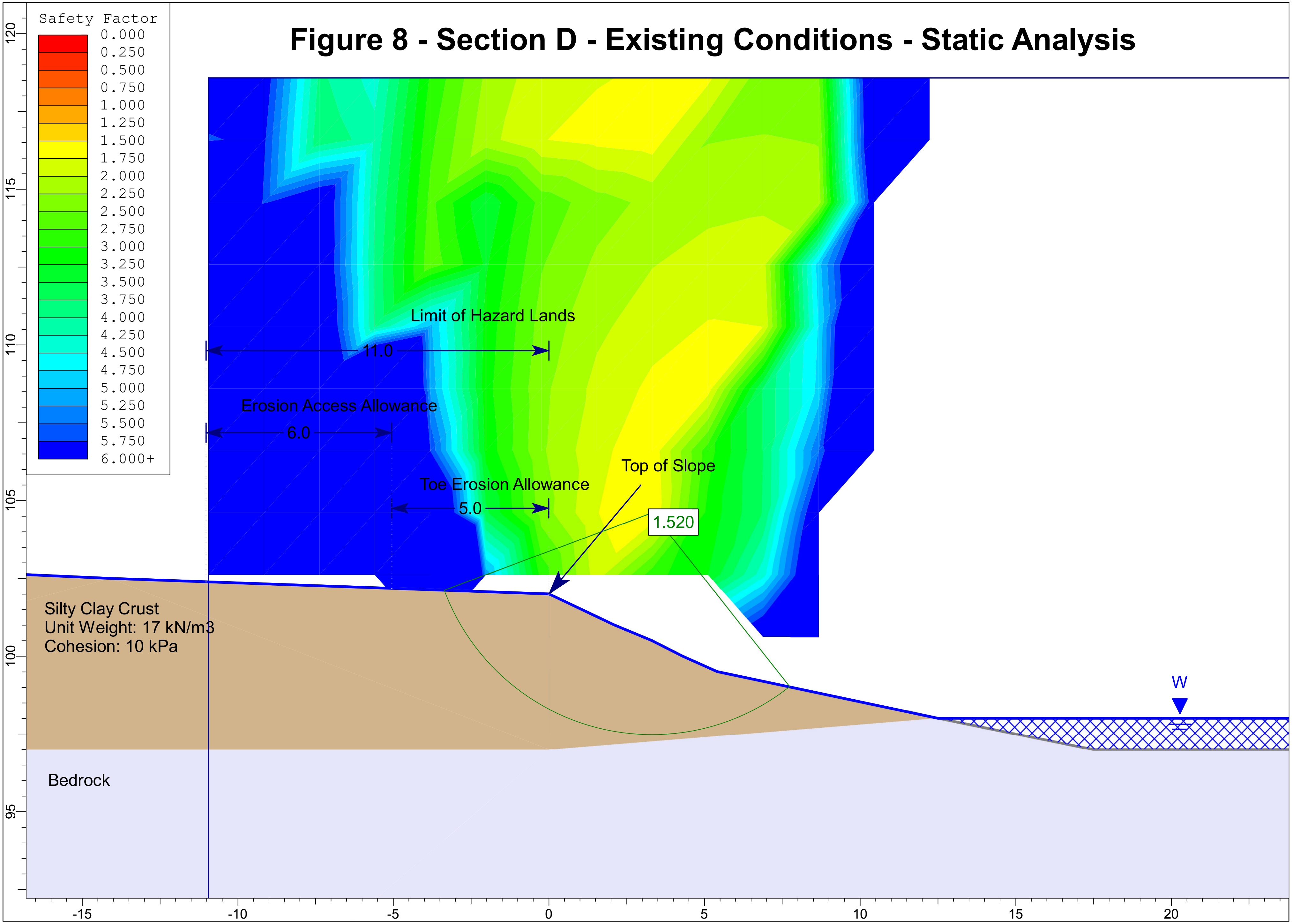


Figure 7 - Section C - Existing Conditions - Seismic Analysis

Figure 8 - Section D - Existing Conditions - Static Analysis



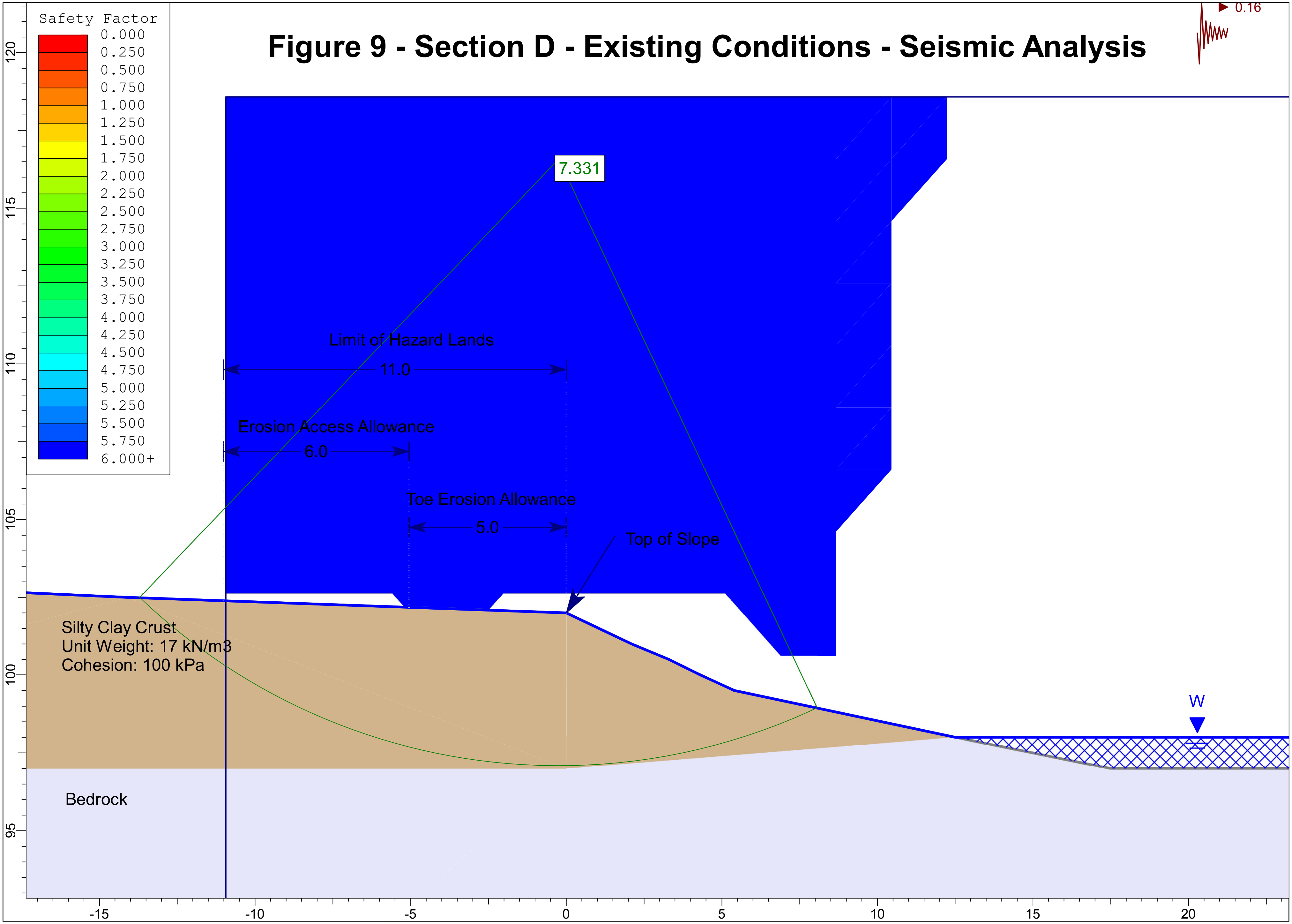


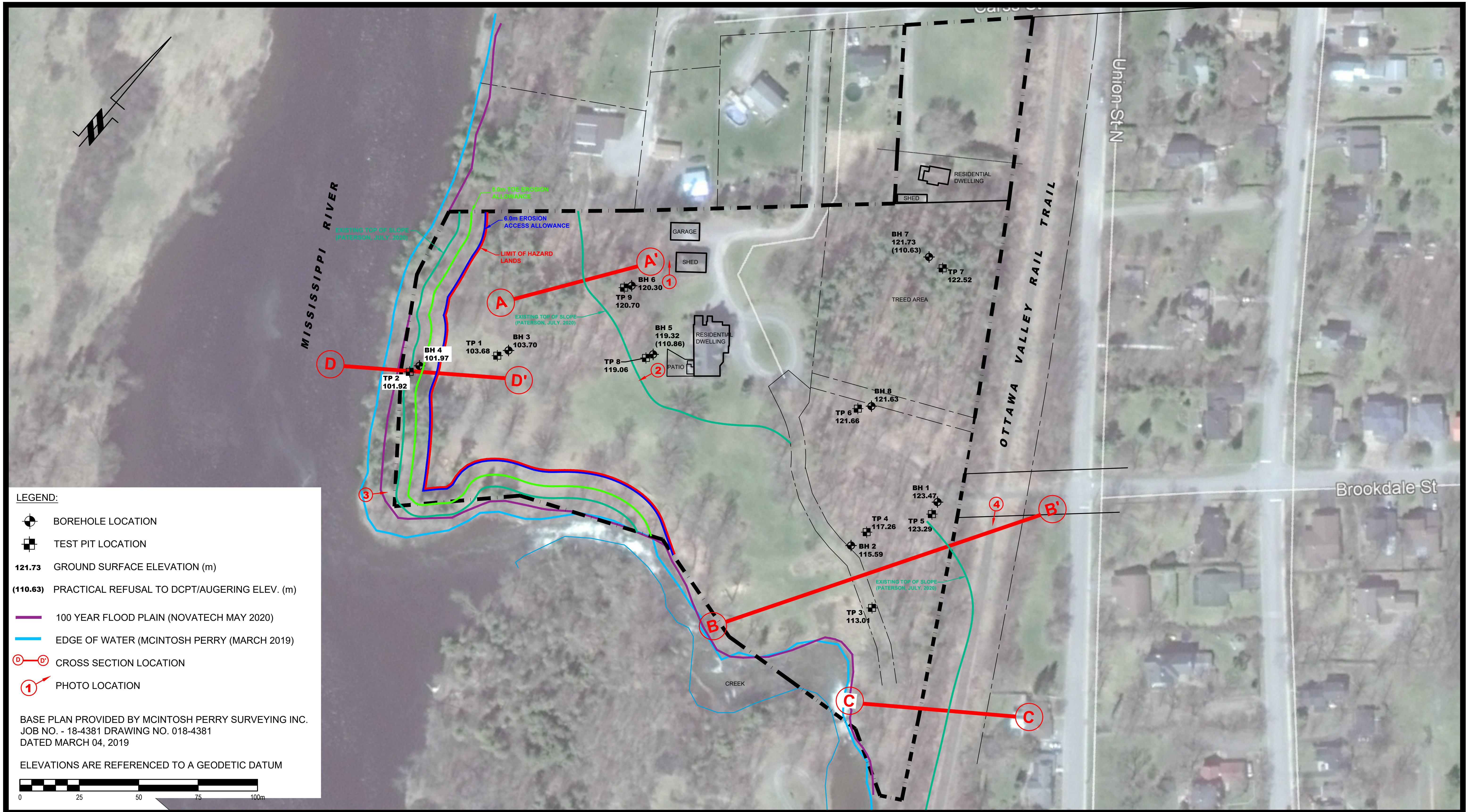
Figure 9 - Section D - Existing Conditions - Seismic Analysis

Photo 1: Existing conditions of northwest slope face, looking north. (June 17, 2020)

Photo 2: Existing conditions of northwest slope face, looking west. (June 17, 2020)

Photo 3: Existing conditions of slope face fronting onto the Mississippi River, looking east. (June 17, 2020)

Photo 4: Existing conditions of southeast slope face, looking south. (June 17, 2020)



LEGEND:

⊕	BOREHOLE LOCATION
▣	TEST PIT LOCATION
121.73	GROUND SURFACE ELEVATION (m)
(110.63)	PRACTICAL REFUSAL TO DCPT/AUGERING ELEV. (m)
🟪	100 YEAR FLOOD PLAIN (NOVATECH MAY 2020)
🟦	EDGE OF WATER (MCINTOSH PERRY (MARCH 2019)
Ⓓ—Ⓓ'	CROSS SECTION LOCATION
①	PHOTO LOCATION

BASE PLAN PROVIDED BY MCINTOSH PERRY SURVEYING INC.
JOB NO. - 18-4381 DRAWING NO. 018-4381
DATED MARCH 04, 2019

ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM

0 25 50 75 100m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

MR. PATRICK ASHBY

GEOTECHNICAL INVESTIGATION - PROPOSED DEVELOPMENT
39 CARSS STREET

ALMONTE, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1500
Date: 07/2020
Drawn by: RCG
Report No.: PG5402
Checked by: KP
Approved by: DJG
PG5402-1
Revision No.:

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