Geotechnical Engineering

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# Geotechnical Investigation

Proposed Residential Development
147 Langstaff Drive
Ottawa, Ontario

Prepared For

Inverness Homes
c/o The Stirling Group

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## Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned by Inverness Homes, c/o The Stirling Group, to conduct a geotechnical investigation for the proposed residential development to be located at 147 Langstaff Drive in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2). The objectives of the investigation were to:

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

❏ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect 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 understood that the proposed residential development will consist of townhouses of slab-on-grade construction, a clubhouse of slab-on-grade construction, and several low-rise apartment buildings each with 1 level of underground parking. The proposed development will also include associated access lanes and parking areas with landscaped margins. In addition, parkland is proposed along the east and west sides of the central ravine at the site. It is understood that the site will be municipally serviced.

# 3.0 Method of Investigation

## 3.1 Field Investigation

The field program for the current investigation was carried out on July 11 and July 12, 2019. At that time, a total of seven (7) boreholes were advanced to a maximum depth of 9.1 m below existing ground surface. A previous field investigation was also completed by Paterson on October 23 and 24, 2008. During that time a total of six (6) boreholes were advanced to a maximum depth of 9.8 m. The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG4918-1 - Test Hole Location Plan included in Appendix 2.

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

**Sampling and In Situ Testing**

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler or from the auger flights. The depths at which the auger and split spoon samples were recovered from the test holes 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. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 2 and BH 7-19. 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 monitoring wells were installed at BH 1-19, BH 2-19, and BH 3-19, and flexible PVC standpipes were installed in the remaining boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented on the Soil Profile and Test Data sheets in Appendix 1.

**Sample Storage**

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

## 3.2 Field Survey

The test hole locations from the current geotechnical investigation were determined by Paterson personnel and were located and surveyed in the field by Robinson Land Development. These boreholes are understood to be referenced to a geodetic datum.

The ground surface elevations at the previous borehole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle on the fire hydrant located along Langstaff Drive in the northwest corner of the subject site. An assumed elevation of 100.00 m was assigned to the TBM. The locations of the boreholes are presented on Drawing PG4918-1 - Test Hole Location Plan in Appendix 2.

## 3.3 Laboratory Testing

The soil samples recovered from the field investigation were examined in our laboratory to review the results of the field logging.

## 3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was 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 6.7.

# 4.0 Observations

## 4.1 Surface Conditions

The subject site is currently undeveloped and has a 3 to 5 m deep ravine running in an approximate north-south direction through the central portion of the site, and a 4 to 5 m deep ravine running along the west property boundary. The site generally has a grassed surface, with the exception of forested areas along the ravines. The northern boundary of the site is at grade with Langstaff Drive, and slopes downward gradually to the southeast.

## 4.2 Subsurface Profile

**Overburden**

The subsurface profile encountered at the borehole locations generally consisted of a 150 to 200 mm thick layer of topsoil underlain by a compact silty sand extending to approximate depths of 0.6 to 1.4 m below the existing ground surface.

Underlying the topsoil and/or silty sand, a silty clay deposit was encountered. The silty clay was generally observed to consist of a stiff, brown silty clay crust, becoming a stiff to firm grey silty clay at approximate depths of 4.5 to 6 m below the existing ground surface.

Underlying the silty clay, a grey silty sand to sandy silt was encountered at approximate depths of 6 to 8.5 m below the existing ground surface.

**Bedrock**

Practical refusal to the DCPT was encountered at depths of 18.9 and 24.9 m in boreholes BH 2 and BH 7-19, respectively. 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, bedrock in the area consists of interbedded limestone and shale of the Verulam Formation with overburden drift thickness ranging between 15 and 50 m.

## 4.3 Groundwater

Groundwater levels (GWL) were measured in BH 1-19 through BH 7-19 on July 18, 2019 and in BH 1 through BH 6 on November 3, 2008. The measured GWL readings are presented in Table 1 below. Groundwater levels can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected between a 4.5 to 6 m depth.

However, it should be noted that the groundwater levels can fluctuate periodically throughout the year and higher levels could be encountered at the time of construction.

**Table 1 - Summary of Groundwater Level Readings**

| Borehole Number | Ground Elevation (m) | Measured Groundwater Level (m) | | Recording Date |
|---|---|---|---|---|
| | | Depth | Elevation | |
| BH 1-19* | 106.71 | 6.17 | 100.54 | July 18, 2019 |
| BH 2-19* | 106.86 | 5.30 | 101.56 | July 18, 2019 |
| BH 3-19* | 105.94 | 6.91 | 99.03 | July 18, 2019 |
| BH 4-19 | 104.31 | 2.07 | 102.24 | July 18, 2019 |
| BH 5-19 | 100.75 | Blocked | - | July 18, 2019 |
| BH 6-19 | 100.81 | Blocked | - | July 18, 2019 |
| BH 7-19 | 102.86 | 2.8 | 100.06 | July 18, 2019 |
| BH 1 | - | 5.75 | - | November 3, 2008 |
| BH 2 | - | 4.76 | - | November 3, 2008 |
| BH 3 | - | 6.31 | - | November 3, 2008 |
| BH 4 | - | 6.95 | - | November 3, 2008 |
| BH 5 | - | 7.05 | - | November 3, 2008 |
| BH 6 | - | 6.7 | - | November 3, 2008 |

**Notes:**
- * Denotes borehole instrumented with a 51 mm diameter monitoring well.
- Current boreholes elevations (BH 1-19 through BH 6-19) surveyed by Robinson Land Development and are understood to be referenced to a geodetic datum.

# 5.0 Discussion

## 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed residential development. It is expected that the proposed residential buildings will be founded on conventional shallow footings placed on an undisturbed, compact silty sand and/or undisturbed stiff silty clay bearing surface.

Due to the presence of a silty clay deposit, permissible grade raise restrictions are recommended for this site.

A construction setback defined as the Limit of Hazard Lands has been defined along the existing ravines, as presented on Drawing PG4918-1 - Test Hole Location Plan. This is discussed further in Section 6.8.

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

## 5.2 Site Grading and Preparation

### Stripping Depth

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

### Fill Placement

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

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of the SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

## 5.3 Foundation Design

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, compact silty sand or undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**.

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 placement of concrete for footings.

The bearing resistance value at SLS for shallow footings bearing on the above noted soils will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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 sand or 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 bearing soil.

**Permissible Grade Raise Recommendations**

Consideration must also be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill. The foundation loads to be considered for the settlement case are the continuously applied loads which consist of the unfactored dead loads and the portion of the unfactored live load that is considered to be continuously applied. For buildings, a minimum value of 50% of the live load is often recommended by Paterson. A post-development groundwater lowering of 0.5 m was assumed.

Our permissible grade raise recommendations for the proposed residential development are presented in Drawing PG4918-2 - Permissible Grade Raise Plan in Appendix 2. It should be noted that grade raises are not permitted within the Hazard Lands as identified on Drawing PG4918-2.

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 Ontario Building Code (OBC) 2012. The soils underlying the site are not considered susceptible to liquefaction.

## 5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the native soil surface or approved engineered fill surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

For structures with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone. Further, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the basement slabs. 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 proposed buildings. 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 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 12.2 $kN/m^3$, 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 $K_o \gamma H$ where:

$K_o$ = At-rest earth pressure coefficient of the applicable retained soil (0.5)
$\gamma$ = unit weight of fill of the applicable retained soil ($kN/m^3$)
H = height of the basement wall (m)

An additional pressure having a magnitude equal to $K_o$ 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 stay at least 0.3 m away 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 ($\Delta P_{AE}$).

The seismic earth force ($\Delta P_{AE}$) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

$a_c$ = $(1.45 - a_{max}/g)a_{max}$
$\gamma$ = 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 $P_o = 0.5\ K_o\ \gamma\ H^2$, 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 structure presented in the following tables is recommended for the design of car only parking areas, local roadways and arterial roadways with bus traffic.

**Table 2 - Recommended Pavement Structure - Driveways / 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 3 - Recommended Pavement Structure - Local Roads**

| 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 | |

**Table 4 - Recommended Pavement Structure - Roadways with Bus Traffic**

| Thickness mm | Material Description |
|---|---|
| 40 | **Wear Course -** Superpave 12.5 Asphaltic Concrete |
| 50 | **Upper Binder Course** - Superpave 19.0 Asphaltic Concrete |
| 50 | **Lower Binder Course** - Superpave 19.0 Asphaltic Concrete |
| 150 | **BASE** - OPSS Granular A Crushed Stone |
| 600 | **SUBBASE** - OPSS Granular B Type II |
| **SUBGRADE** - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill | |

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

**Pavement Structure Drainage**

Satisfactory performance of the pavement structure is largely dependent on maintaining the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage and Backfill

### Foundation Drainage

A perimeter foundation drainage system is recommended for each proposed structure. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated, corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone which is 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.

### Sub-slab Drainage

For the proposed low-rise apartment buildings, which are understood to contain 1 level of underground parking, sub-slab drainage will be required to control water infiltration. For preliminary design purposes, it is recommended that 100 or 150 mm perforated pipes be placed at approximate 6 m centres underlying the basement floor. 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 excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided for any elevator pits (pit bottom and walls).

## 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 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, 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.

Unheated structures, such as the access ramps for the underground parking levels, may require foundation insulation for protection against the deleterious effects of frost action.

## 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).

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

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the City of Ottawa. These recommendations are for standard, open cut excavation placed services.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. 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 extend 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.

Generally, it should 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.

## 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 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.

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

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

## 6.6 Winter Construction

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

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

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

## 6.7 Tree Planting Restrictions

The silty clay deposit encountered at the site was hard to firm and is considered to be low to medium sensitivity clay and should not be considered a sensitive marine clay. Therefore, where footings are founded over a silty clay bearing surface, large trees (mature height over 14 m) can be planted provided a tree to foundation setback equal to the full mature height of the tree is utilized (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). It should be noted that shrubs and other small plantings are permitted within the 4.5 m setback area.

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

## 6.8 Slope Stability Assessment

The slope condition was reviewed by Paterson field personnel as part of the geotechnical investigation. Eight slope cross-sections were studied as the worst case scenarios. The cross section locations are presented on Drawing PG4918-1 - Test Hole Location Plan attached to the current report.

The existing slope face of the ravine located within the central portion of the site is currently vegetated and tree covered with signs of minor erosion within the lower portion of the slope face. However, the slope face was not observed to be in direct contact with the watercourse. The upper slope is observed to be well vegetated and stable with little to no signs of active erosion. A 0.5 to 1 m wide watercourse was observed at the base of the central ravine. The watercourse was observed to be less than 75 mm deep with little to no flow. Two existing stormwater management ponds (SWMP) were observed along the existing watercourse. The first SWMP is located north of Langstaff Drive and the second is located south of Langstaff Drive within our subject site.

The existing slope face of the ravine located along the west property boundary was noted to be vegetated and treed throughout. A watercourse was not observed in this ravine at the time of our site visit, however, it is understood that intermittent water flow occurs through this corridor. A 0.6 m diameter corrugated steel drainage pipe drains into the ravine approximately 20 m south of the previously in-filled area at the north end of the ravine. No evidence of active erosion was noted along this slope.

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 PG4918-1 - Test Hole Location Plan attached to the current report.

### 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 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, realistic groundwater flow conditions. Subsoil conditions at the cross-sections were inferred based on nearby boreholes and general knowledge of the area's geology.

The effective strength soil parameters used for static analysis 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 5 below.

**Table 5 - Effective Strength Soil and Material Parameters (Static Analysis)**

| Soil Layer | Unit Weight ($kN/m^3$) | Friction Angle (degrees) | Cohesion (kPa) |
|---|---|---|---|
| Brown Silty Clay Crust | 17 | 33 | 7 |
| Silty Sand | 20 | 35 | 1 |

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the open boreholes completed at the time of our geotechnical investigation and based on our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 6 below.

**Table 6 - 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 | 150 |
| Silty Sand | 20 | 35 | 1 |

### Static Loading Analysis

The results for the slope stability analyses under static conditions at Sections A, B, C, D, E and F are shown on Figures 2, 4, 8, 12, 14 and 16 are attached to the present report. The factor of safety was found to be greater than 1.5 at Sections A, B, E, and F. However, Sections C and D require setbacks of 4.8 and 3.5 m, respectively, from top of slope to obtain a factor of safety greater than 1.5.

### 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 are shown on Figures 3, 5, 9, 13, 15, and 17 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 slope face that confines the watercourse located across the central portion of the site. Some minor sloughing failures were noted in the lower portion of the slope, leaving some exposed tree roots. No evidence of active erosion was noted along the toe of slope.

The general characteristics of the central and western ravines are also discussed in detail in the report titled “Fluvial Geomorphological and Erosion Hazard Assessment” prepared for this site by Geomorphix dated October 1, 2019.

Based on our observations and the recommendations in the above-noted report prepared by Geomorphix, a 2 m toe erosion allowance is deemed appropriate for central ravine 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 2 m and an erosion access allowance of 6 m is required from the top of slope.

For the slope that confines the intermittent watercourse located on the western end of the site, a 1 m toe erosion allowance is deemed appropriate based on our observations on-site and the recommendations provided by Geomorphix. Given the low risk of erosion for the western ravine, an erosion access allowance of 4.5 m is considered sufficient in addition to the 1 m toe erosion allowance from the top of stable slope (i.e. - slope with factor of safety greater than 1.5). The 4.5 m erosion access allowance is sufficient for a hydraulic excavator and truck access, if ever required.

The limit of hazard lands, which include these allowances, are indicated on Drawing PG4918-1 - Test Hole Location Plan attached to the present report.

It is recommended that the existing vegetation on the slope faces not be removed as it contributes to the stability of the slope and reduces erosion.

**Global Stability Analysis**

Where retaining walls are proposed in the vicinity of the Limit of Hazard Lands at the subject site, global stability analyses were conducted for these walls. Further, the adjacent slopes were also analyzed to determine if the proposed retaining walls and associated grade raises will impact the stability of the slope and/or the Limit of Hazard Lands.

The results for the global stability analyses under static conditions at Sections B, C, F, G, and H are shown on Figures 6, 10, 18, 20, and 22, which are attached to the present report. The results of these global stability analyses indicate that the factor of safety exceeds 1.5 under static conditions.

The results for the global stability analyses under seismic conditions at Sections B, C, F, G, and H are shown on Figures 7, 11, 19, 21, and 23, which are attached to the present report. The results of these analyses indicate that the factor of safety exceeds 1.1 under seismic conditions.

Further, the presence of the retaining walls and associated grade raises will not impact the factor of safety of the adjacent slopes or the position of the Limit of Hazard Lands.

## 6.9 Corrosion Potential and Sulphate

One (1) soil sample was submitted for analytical 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 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 moderate to aggressive corrosive environment.

# 7.0 Recommendations

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

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

A report confirming that these works have been conducted in general accordance with 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 also be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole logs 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 Inverness Homes 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
Jan. 20, 2020
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott S. Dennis, P.Eng

Andrew J. Tovell, P.Eng.

**Report Distribution:**

- ❑ Inverness Homes (e-mail copy)
- ❑ Paterson Group (1 copy)

# APPENDIX 1

**SOIL PROFILE AND TEST DATA SHEETS**

**SYMBOLS AND TERMS**

**ANALYTICAL TEST RESULTS**

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**147 Langstaff Drive**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Robinson Land Development.

**REMARKS**

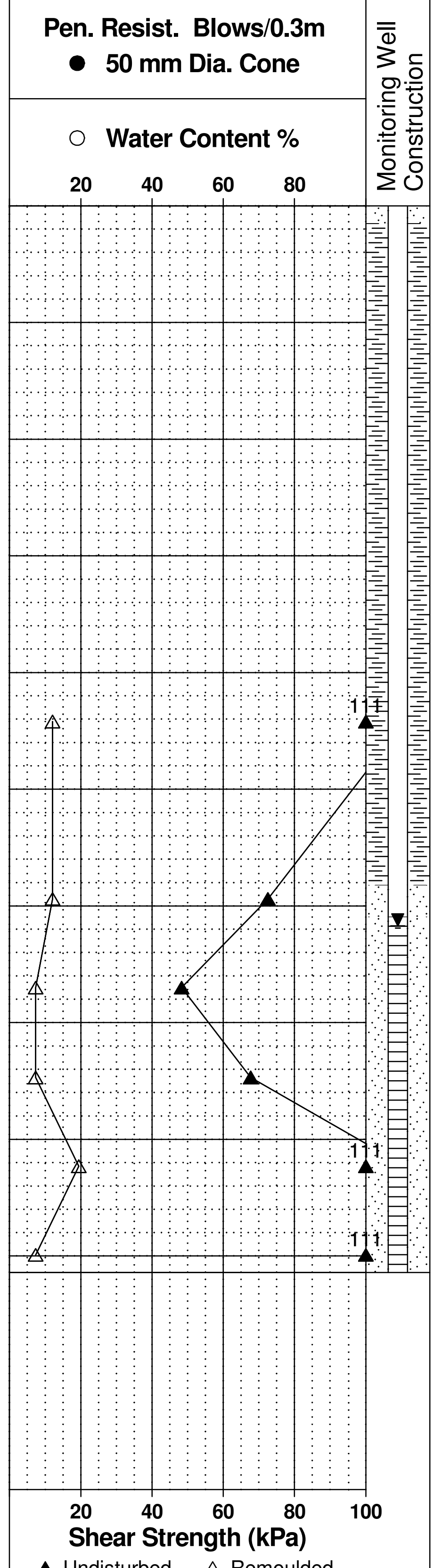
**BORINGS BY** CME 55 Power Auger

**DATE** 2019 July 11

**FILE NO.** PG4918

**HOLE NO.** BH 1-19

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 106.71 |
| TOPSOIL 0.20 | | AU | 1 | | | | |
| Very stiff to stiff, brown **SILTY CLAY** | | SS | 2 | 58 | 13 | 1 | 105.71 |
| | | SS | 3 | 100 | 10 | 2 | 104.71 |
| | | SS | 4 | 100 | 8 | 3 | 103.71 |
| | | SS | 5 | 100 | 5 | 4 | 102.71 |
| | | SS | 6 | 100 | 4 | 5 | 101.71 |
| - firm to very stiff and grey by 6.1m depth | | | | | | 6 | 100.71 |
| | | | | | | 7 | 99.71 |
| | | | | | | 8 | 98.71 |
| 9.14 | | | | | | 9 | 97.71 |
| End of Borehole | | | | | | | |
| (GWL @ 6.17m - July 18, 2019) | | | | | | | |



**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**147 Langstaff Drive**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Robinson Land Development.

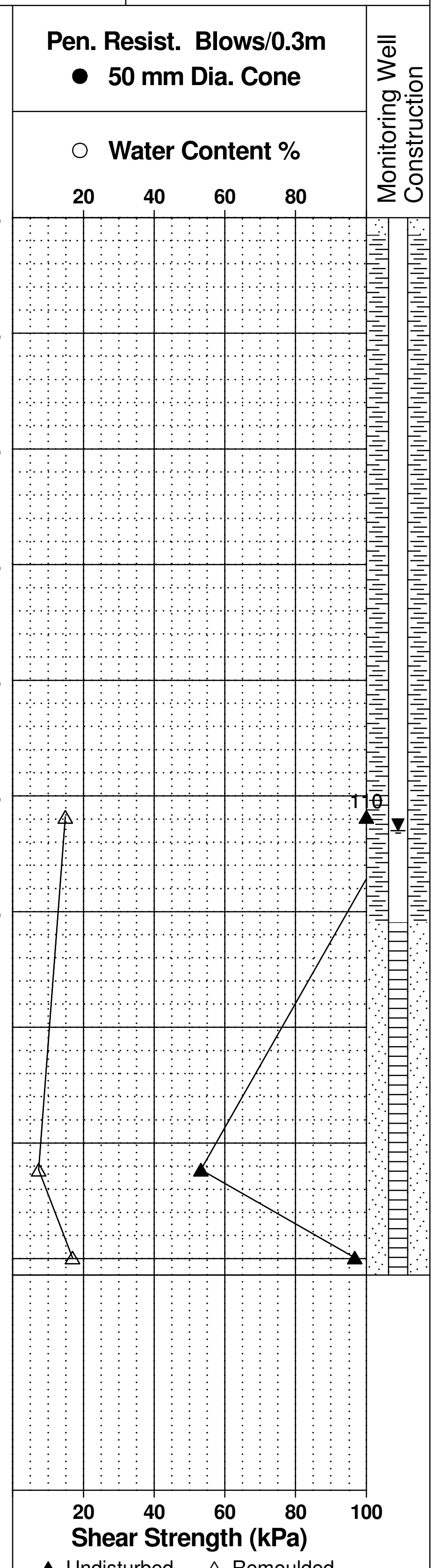
**REMARKS**

**BORINGS BY** CME 55 Power Auger **DATE** 2019 July 11

**FILE NO.** PG4918

**HOLE NO.** BH 2-19

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 106.86 |
| TOPSOIL 0.23 | | AU | 1 | | | | |
| Very stiff to stiff, brown **SILTY CLAY** | | SS | 2 | 100 | 15 | 1 | 105.86 |
| - grey by 3.0m depth | | SS | 3 | 100 | 14 | 2 | 104.86 |
| | | SS | 4 | 100 | 11 | 3 | 103.86 |
| | | SS | 5 | 100 | 8 | 4 | 102.86 |
| 5.30 | | SS | 6 | 79 | 6 | 5 | 101.86 |
| Brown **SILTY CLAY** trace sand | | SS | 7 | 62 | 9 | 6 | 100.86 |
| 6.90 | | | | | | 7 | 99.86 |
| Stiff, grey **SILTY CLAY** with sand | | SS | 8 | 100 | 2 | 8 | 98.86 |
| 9.14 | | | | | | 9 | 97.86 |
| End of Borehole | | | | | | | |
| (GWL @ 5.30m - July 18, 2019) | | | | | | | |



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**147 Langstaff Drive**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Robinson Land Development.

**FILE NO.** PG4918

**REMARKS**

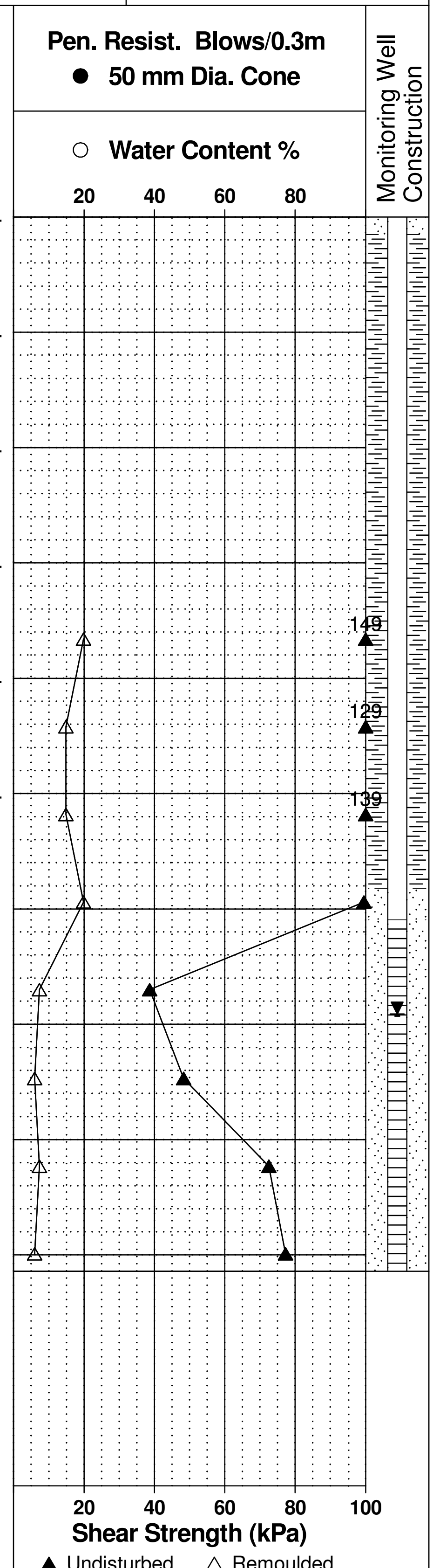
**HOLE NO.** BH 3-19

**BORINGS BY** CME 55 Power Auger **DATE** 2019 July 11

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 105.94 |
| TOPSOIL 0.18 | | AU | 1 | | | | |
| Very stiff to stiff, brown SILTY CLAY | | SS | 2 | 12 | 11 | 1 | 104.94 |
| | | SS | 3 | 79 | 7 | 2 | 103.94 |
| | | SS | 4 | 83 | 5 | 3 | 102.94 |
| | | SS | 5 | 100 | 4 | | |
| | | SS | 6 | 100 | 5 | 4 | 101.94 |
| | | SS | 7 | 100 | 4 | 5 | 100.94 |
| | | SS | 8 | 100 | 2 | 6 | 99.94 |
| - firm to stiff and grey by 6.1m depth | | SS | 9 | 100 | W | | |
| | | SS | 10 | 100 | W | 7 | 98.94 |
| | | SS | 11 | 100 | W | 8 | 97.94 |
| | | SS | 12 | 0 | W | 9 | 96.94 |
| 9.14 | | | | | | | |

End of Borehole

(GWL @ 6.91m - July 18, 2019)



# patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**147 Langstaff Drive**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Robinson Land Development.

**REMARKS**

**BORINGS BY** CME 55 Power Auger **DATE** 2019 July 11

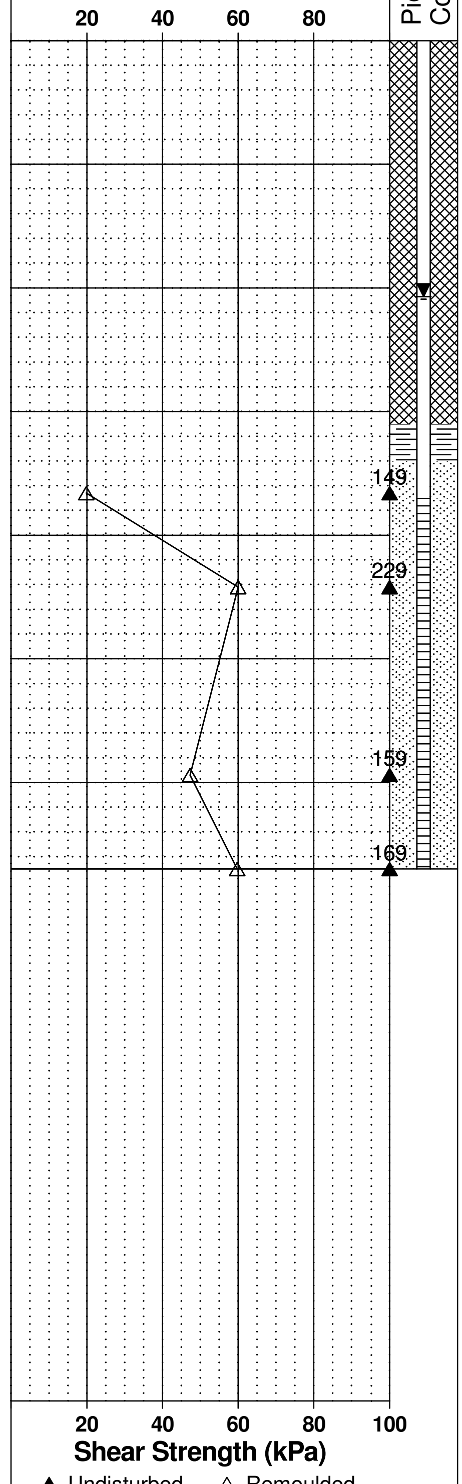
**FILE NO.** PG4918

**HOLE NO.** BH 4-19

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 104.31 |
| TOPSOIL 0.20 | | AU | 1 | | | | |
| | | SS | 2 | 100 | 14 | 1 | 103.31 |
| | | SS | 3 | 100 | 9 | 2 | 102.31 |
| | | SS | 4 | 100 | 6 | | |
| Very stiff, brown **SILTY CLAY,** trace sand | | | | | | 3 | 101.31 |
| | | | | | | 4 | 100.31 |
| - grey by 4.6m depth | | SS | 5 | 100 | 4 | 5 | 99.31 |
| 6.70 | | | | | | 6 | 98.31 |
| End of Borehole | | | | | | | |
| (GWL @ 2.07m - July 18, 2019) | | | | | | | |

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

○ Water Content %



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# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
147 Langstaff Drive
Ottawa, Ontario

**DATUM** Ground surface elevations provided by Robinson Land Development.

**FILE NO.** PG4918

**REMARKS**

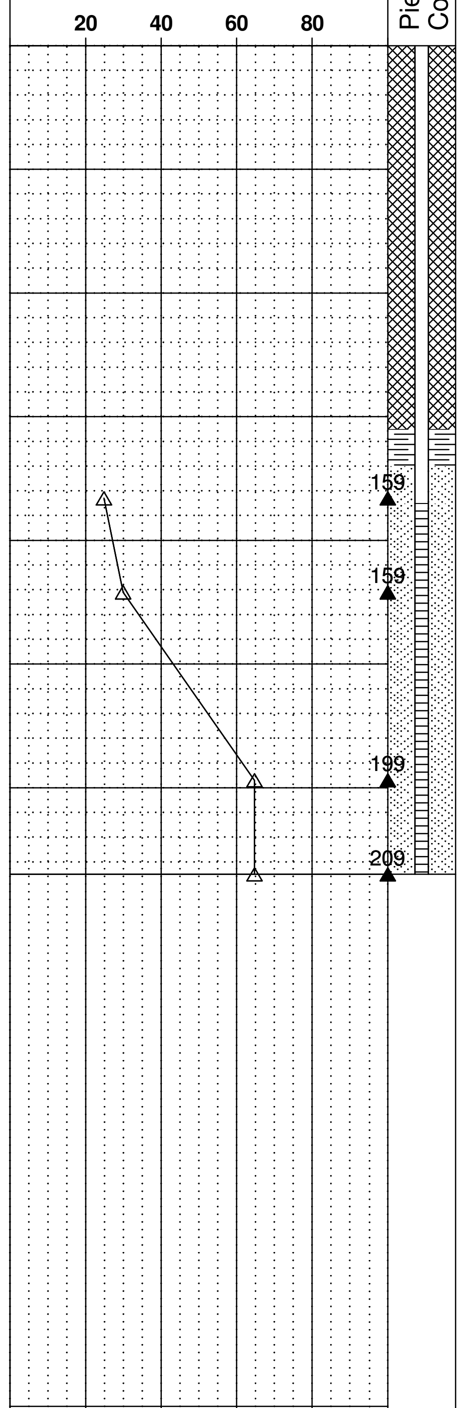
**HOLE NO.** BH 5-19

**BORINGS BY** CME 55 Power Auger **DATE** 2019 July 12

| SOIL DESCRIPTION | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | 0 | 100.75 |
| **TOPSOIL** 0.20 | AU | 1 | | | | |
| | SS | 2 | 88 | 11 | 1 | 99.75 |
| | SS | 3 | 100 | 10 | 2 | 98.75 |
| | SS | 4 | 100 | 9 | | |
| Very stiff, brown **SILTY CLAY** | | | | | 3 | 97.75 |
| | | | | | 4 | 96.75 |
| - grey by 4.6m depth | SS | 5 | 100 | 7 | 5 | 95.75 |
| | | | | | 6 | 94.75 |
| 6.70 End of Borehole | | | | | | |
| (Piezometer blocked - July 18, 2019) | | | | | | |

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

**Geotechnical Investigation**
**147 Langstaff Drive**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Robinson Land Development.

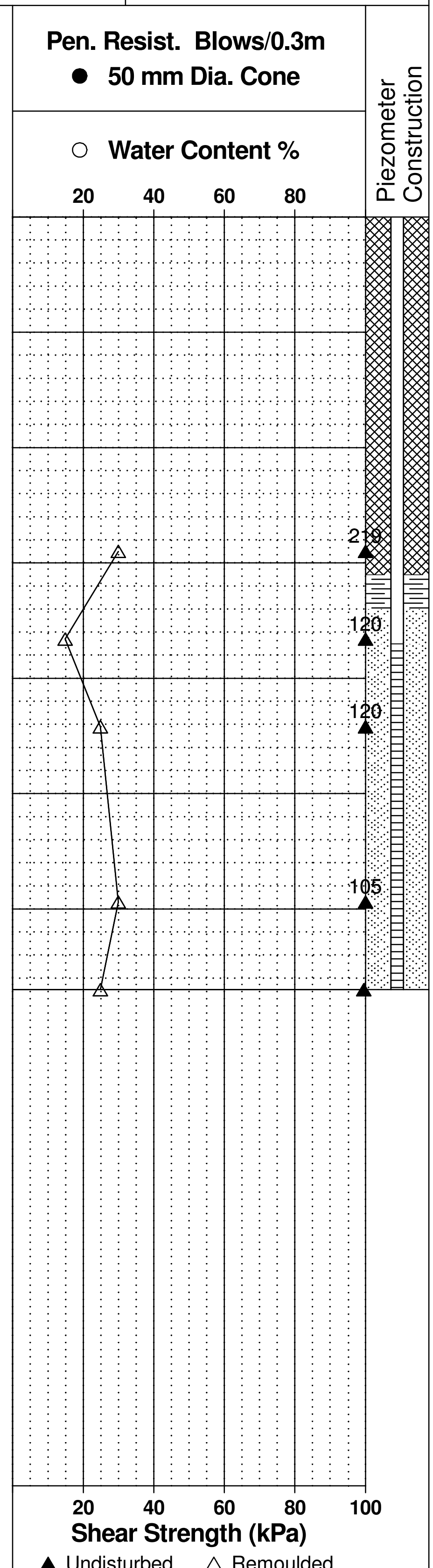
**REMARKS**

**BORINGS BY** CME 55 Power Auger **DATE** 2019 July 12

**FILE NO.** PG4918

**HOLE NO.** BH 6-19

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 100.81 |
| TOPSOIL 0.15 | | AU | 1 | | | | |
| Hard to stiff, brown **SILTY CLAY** | | SS | 2 | 58 | 8 | 1 | 99.81 |
| | | SS | 3 | 75 | 9 | 2 | 98.81 |
| | | | | | | 3 | 97.81 |
| | | | | | | 4 | 96.81 |
| - grey by 4.6m depth | | SS | 4 | 100 | 2 | 5 | 95.81 |
| 6.70 | | | | | | 6 | 94.81 |
| End of Borehole | | | | | | | |
| (Piezometer blocked - July 18, 2019) | | | | | | | |



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
147 Langstaff Drive
Ottawa, Ontario

**DATUM** Ground surface elevations provided by Robinson Land Development.

**FILE NO.** PG4918

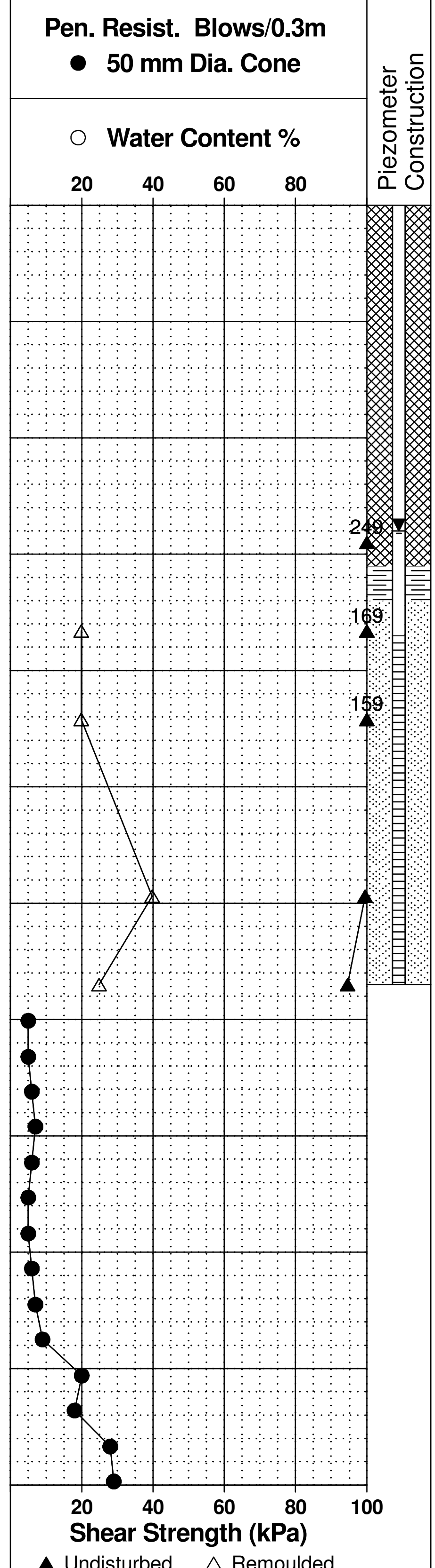
**REMARKS**

**HOLE NO.** BH 7-19

**BORINGS BY** CME 55 Power Auger

**DATE** 2019 July 12

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 102.86 |
| TOPSOIL 0.18 | | AU | 1 | | | | |
| | | SS | 2 | 100 | 13 | 1 | 101.86 |
| | | SS | 3 | 100 | 13 | 2 | 100.86 |
| Very stiff to stiff, brown **SILTY CLAY** | | | | | | 3 | 99.86 |
| | | | | | | 4 | 98.86 |
| - grey by 4.6m depth | | SS | 4 | 88 | 5 | 5 | 97.86 |
| 6.70 | | | | | | 6 | 96.86 |
| Dynamic Cone Penetration Test commenced at 6.70m depth. | | | | | | 7 | 95.86 |
| | | | | | | 8 | 94.86 |
| | | | | | | 9 | 93.86 |
| | | | | | | 10 | 92.86 |
| | | | | | | 11 | 91.86 |



**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**147 Langstaff Drive**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Robinson Land Development.

**REMARKS**

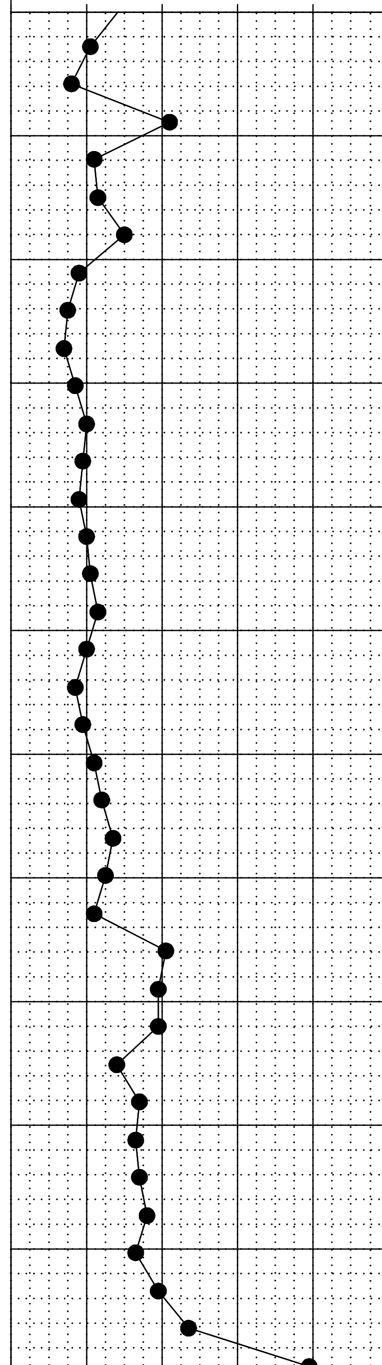
**BORINGS BY** CME 55 Power Auger

**DATE** 2019 July 12

**FILE NO.** PG4918

**HOLE NO.** BH 7-19

| 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 | | | | | | | | 20 40 60 80 | |
| | | | | | | 11 | 91.86 | | |
| | | | | | | 12 | 90.86 | | |
| | | | | | | 13 | 89.86 | | |
| | | | | | | 14 | 88.86 | | |
| | | | | | | 15 | 87.86 | | |
| | | | | | | 16 | 86.86 | | |
| | | | | | | 17 | 85.86 | | |
| | | | | | | 18 | 84.86 | | |
| | | | | | | 19 | 83.86 | | |
| | | | | | | 20 | 82.86 | | |
| | | | | | | 21 | 81.86 | | |
| | | | | | | 22 | 80.86 | | |



20 40 60 80 100

**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**147 Langstaff Drive**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Robinson Land Development.

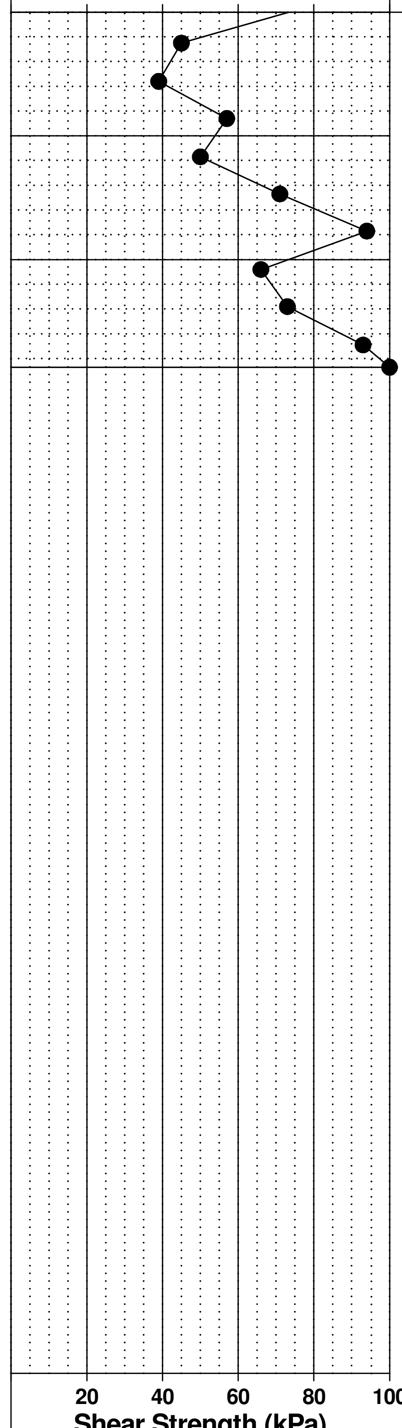
**FILE NO.** PG4918

**REMARKS**

**HOLE NO.** BH 7-19

**BORINGS BY** CME 55 Power Auger **DATE** 2019 July 12

| 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 | | | | | | | | 20 40 60 80 | |
| | | | | | | 22 | 80.86 | | |
| | | | | | | 23 | 79.86 | | |
| | | | | | | 24 | 78.86 | | |
| 24.87 | | | | | | | | | |
| End of Borehole<br>Practical DCPT refusal at 24.87m depth<br>(GWL @ 2.80m - July 18, 2019) | | | | | | | | | |



**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-Langstaff Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in the northwest corner of hte subject site. Assumed elevation = 100.00m.

REMARKS

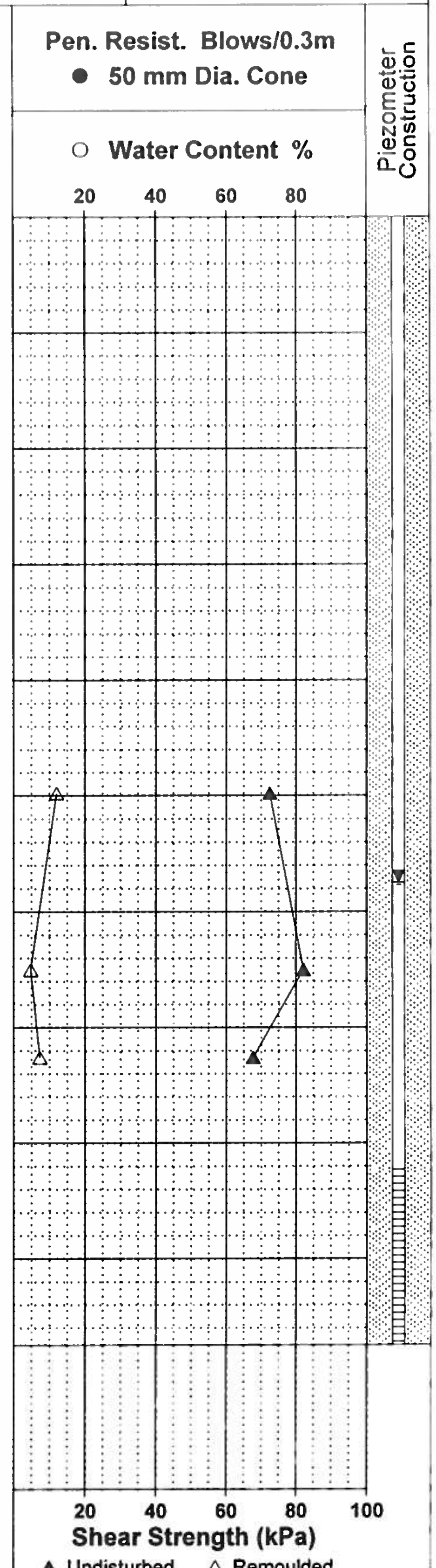
BORINGS BY CME 55 Power Auger

DATE 23 Oct 08

FILE NO. PG1773

HOLE NO. BH 1

| SOIL DESCRIPTION | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | 0 | 97.20 |
| 25mm Topsoil over compact, brown **SILTY SAND** (0.60) | AU | 1 | | | | |
| Very stiff to stiff, brown **SILTY CLAY**, trace to some sand | SS | 2 | 92 | 13 | 1 | 96.20 |
| | SS | 3 | 100 | 8 | 2 | 95.20 |
| | SS | 4 | 100 | 4 | 3 | 94.20 |
| | SS | 5 | 100 | 4 | | |
| - grey by 4.4m depth | SS | 6 | 100 | 4 | 4 | 93.20 |
| | | | | | 5 | 92.20 |
| | SS | 7 | 100 | 2 | 6 | 91.20 |
| | | | | | 7 | 90.20 |
| (8.38) | SS | 8 | 100 | 3 | 8 | 89.20 |
| Very loose, grey **SILTY SAND**, trace clay | SS | 9 | 75 | 2 | 9 | 88.20 |
| (9.75) | SS | 10 | 100 | 3 | | |
| End of Borehole | | | | | | |
| (GWL @ 5.75m-Nov. 3/08) | | | | | | |



**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-Langstaff Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in the northwest corner of hte subject site. Assumed elevation = 100.00m.

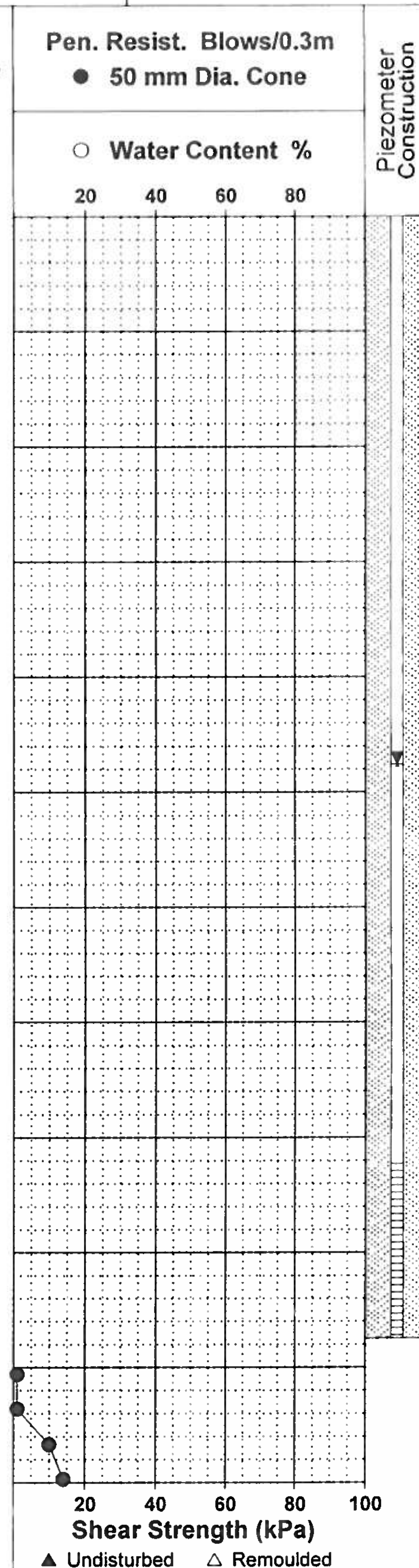
REMARKS

BORINGS BY CME 55 Power Auger DATE 23 Oct 08

FILE NO. PG1773

HOLE NO. BH 2

| SOIL DESCRIPTION | Layer depth (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 97.30 |
| 38mm Topsoil over compact, brown **SILTY SAND** with clay | 0.60 | AU | 1 | | | | |
| Very stiff to stiff, brown **SILTY CLAY**, some sand | | SS | 2 | 71 | 13 | 1 | 96.30 |
| | | SS | 3 | 83 | 8 | 2 | 95.30 |
| | | SS | 4 | 100 | 7 | | |
| | | SS | 5 | 100 | 4 | 3 | 94.30 |
| | | SS | 6 | 100 | 4 | 4 | 93.30 |
| | | SS | 7 | 100 | 2 | 5 | 92.30 |
| - grey by 5.2m depth | | SS | 8 | 100 | 2 | 6 | 91.30 |
| | | SS | 9 | 100 | 3 | | |
| | | SS | 10 | 100 | 3 | 7 | 90.30 |
| | | SS | 11 | 100 | 2 | 8 | 89.30 |
| | | SS | 12 | 100 | 2 | 9 | 88.30 |
| | 9.45 | SS | 13 | 100 | 3 | | |
| Very loose, grey **SANDY SILT**, trace clay | 9.75 | | | | | 10 | 87.30 |
| Dynamic Cone Penetration Test commenced @ 9.75m depth Inferred **SANDY SILT** | 10.70 | | | | | 11 | 86.30 |



**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-Langstaff Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in the northwest corner of hte subject site. Assumed elevation = 100.00m.

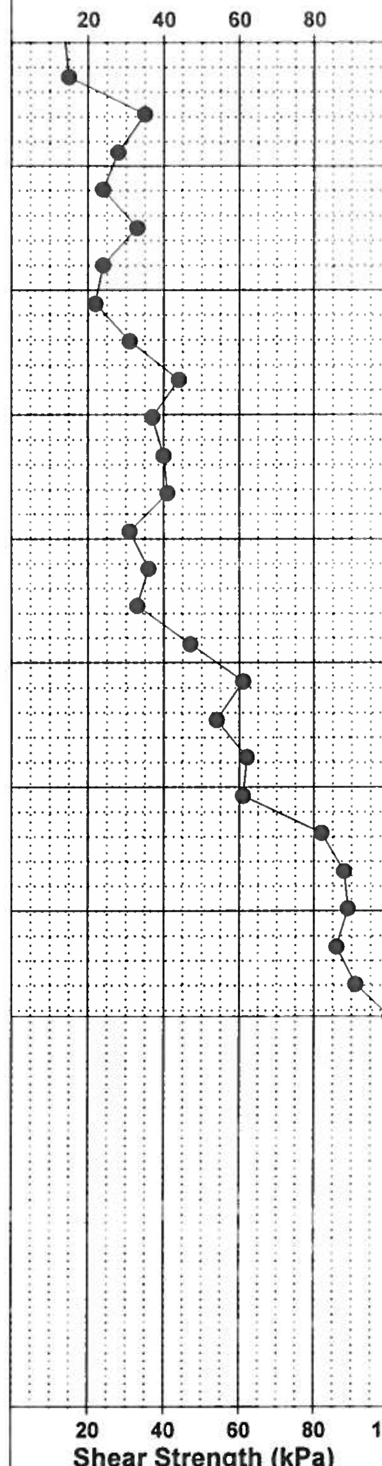
REMARKS

BORINGS BY CME 55 Power Auger DATE 23 Oct 08

FILE NO. PG1773

HOLE NO. BH 2

| 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 | | | | | | | | | |
| Inferred **GLACIAL TILL** | | | | | | 11 | 86.30 | | |
| | | | | | | 12 | 85.30 | | |
| | | | | | | 13 | 84.30 | | |
| | | | | | | 14 | 83.30 | | |
| | | | | | | 15 | 82.30 | | |
| | | | | | | 16 | 81.30 | | |
| | | | | | | 17 | 80.30 | | |
| 18.85 | | | | | | 18 | 79.30 | | |
| End of Borehole<br><br>Practical DCPT refusal @ 18.85m depth<br><br>(GWL @ 4.76m-Nov. 3/08) | | | | | | | | | |



**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-Langstaff Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in the northwest corner of hte subject site. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger DATE 23 Oct 08

FILE NO. PG1773

HOLE NO. BH 3

| SOIL DESCRIPTION | Depth of base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 95.62 |
| TOPSOIL | 0.25 | AU | 1 | | | | |
| Compact, brown **SANDY SILTY**, trace clay | 1.37 | SS | 2 | 100 | 12 | 1 | 94.62 |
| Very stiff to stiff, brown **SILTY CLAY** | | SS | 3 | 100 | 9 | 2 | 93.62 |
| | | SS | 4 | 100 | 5 | 3 | 92.62 |
| | | SS | 5 | 100 | 5 | 4 | 91.62 |
| | | SS | 6 | 100 | 4 | 5 | 90.62 |
| | | SS | 7 | 100 | 5 | 6 | 89.62 |
| - firm and grey by 5.9m depth | | SS | 8 | 100 | 4 | 7 | 88.62 |
| | | SS | 9 | 100 | 2 | | |
| | 8.00 | SS | 10 | 100 | 2 | 8 | 87.62 |
| Grey **CLAYEY SILT** | | SS | 11 | 100 | 2 | | |
| | | SS | 12 | 100 | 2 | 9 | 86.62 |
| | 9.75 | SS | 13 | 100 | 2 | | |
| End of Borehole | | | | | | | |
| (GWL @ 6.31m-Nov. 3/08) | | | | | | | |

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

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Prop. Residential Development-Langstaff Drive**
**Ottawa, Ontario**

DATUM TBM - Top spindle of fire hydrant located in the northwest corner of hte subject site. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 23 Oct 08

FILE NO. PG1773

HOLE NO. BH 4

| SOIL DESCRIPTION | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | 0 | 95.74 |
| TOPSOIL 0.25 | AU | 1 | | | | |
| Very stiff to stiff, brown **SILTY CLAY** with sand | SS | 2 | 100 | 11 | 1 | 94.74 |
| | SS | 3 | 100 | 10 | 2 | 93.74 |
| | SS | 4 | 100 | 6 | 3 | 92.74 |
| | SS | 5 | 100 | 4 | | |
| | SS | 6 | 100 | 5 | 4 | 91.74 |
| | SS | 7 | 100 | 10 | 5 | 90.74 |
| 5.94 | SS | 8 | 100 | 6 | 6 | 89.74 |
| Dense to compact, grey **SILTY SAND** | SS | 9 | 100 | 31 | | |
| | SS | 10 | 25 | 36 | 7 | 88.74 |
| | SS | 11 | 25 | 26 | 8 | 87.74 |
| | SS | 12 | 33 | 25 | 9 | 86.74 |
| 9.75 | SS | 13 | 42 | 22 | | |
| End of Borehole | | | | | | |
| (GWL @ 6.95m-Nov. 3/08) | | | | | | |

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-Langstaff Drive
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant located in the northwest corner of hte subject site. Assumed elevation = 100.00m.

REMARKS

BORINGS BY: CME 55 Power Auger

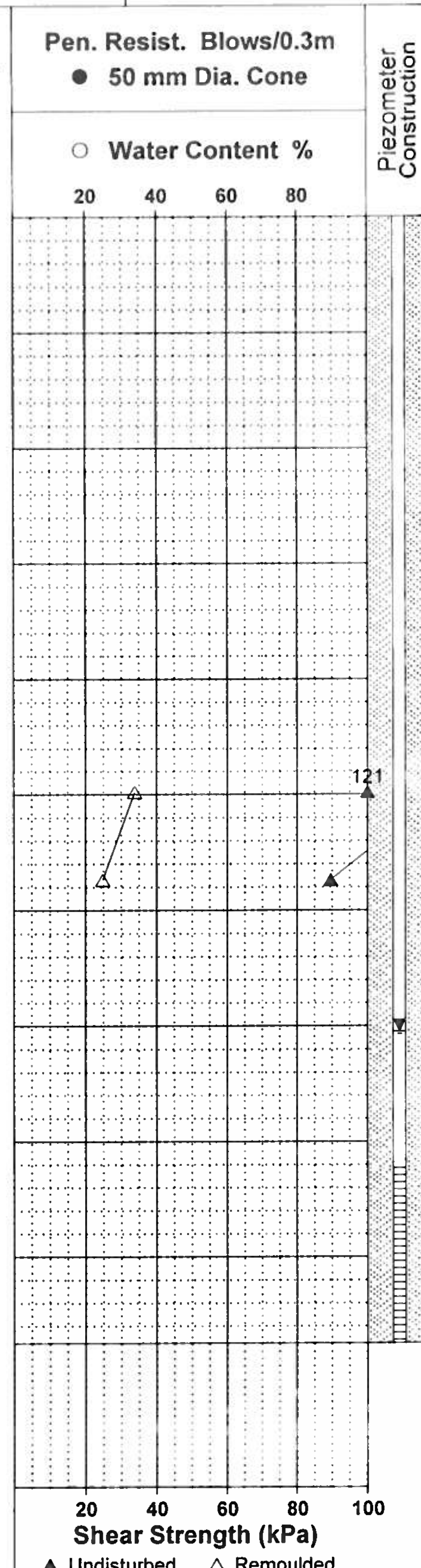
DATE: 24 Oct 08

FILE NO. PG1773

HOLE NO. BH 5

| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|---|
| GROUND SURFACE | | | | | |
| 25mm Topsoil over compact, brown **SILTY SAND** | 0.60 | AU | 1 | | |
| Very stiff to stiff, brown **SILTY CLAY**, trace sand | 7.77 | SS | 2 | 67 | 15 |
| | | SS | 3 | 58 | 11 |
| | | SS | 4 | 100 | 5 |
| | | SS | 5 | 100 | 5 |
| | | SS | 6 | 100 | 6 |
| | | SS | 7 | 100 | 5 |
| | | SS | 8 | 100 | 5 |
| Very dense to dense, grey **SILTY SAND** | 9.75 | SS | 9 | 83 | 32 |
| | | SS | 10 | 33 | 77 |
| | | SS | 11 | 50 | 41 |
| End of Borehole | | | | | |
| (GWL @ 7.05m-Nov. 5/08) | | | | | |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 98.11 |
| 1 | 97.11 |
| 2 | 96.11 |
| 3 | 95.11 |
| 4 | 94.11 |
| 5 | 93.11 |
| 6 | 92.11 |
| 7 | 91.11 |
| 8 | 90.11 |
| 9 | 89.11 |



**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

## SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-Langstaff Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in the northwest corner of hte subject site. Assumed elevation = 100.00m.

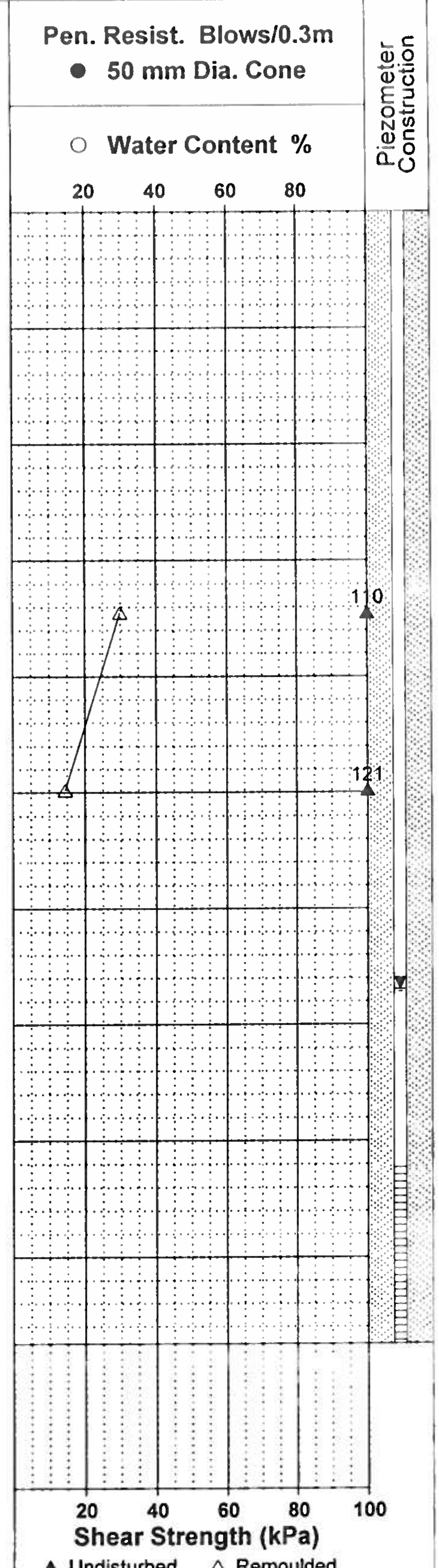
REMARKS

BORINGS BY CME 55 Power Auger    DATE 24 Oct 08

FILE NO. PG1773

HOLE NO. BH 6

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 98.20 |
| 25mm Topsoil over compact, brown **SILTY SAND** with clay — 0.60 | | AU | 1 | | | | |
| Very stiff to stiff, brown **SILTY CLAY** with sand | | SS | 2 | 100 | 20 | 1 | 97.20 |
| | | SS | 3 | 100 | 20 | 2 | 96.20 |
| | | SS | 4 | 100 | 7 | 3 | 95.20 |
| | | SS | 5 | 100 | 4 | 4 | 94.20 |
| | | | | | | 5 | 93.20 |
| | | SS | 6 | 100 | 3 | 6 | 92.20 |
| | | SS | 7 | 100 | 11 | | |
| | | SS | 8 | 100 | 6 | 7 | 91.20 |
| — 8.08 | | SS | 9 | 100 | 2 | 8 | 90.20 |
| Very loose, grey **SANDY SILT** with clay | | SS | 10 | 100 | 2 | 9 | 89.20 |
| — 9.75 | | SS | 11 | 100 | 4 | | |
| End of Borehole | | | | | | | |
| (GWL @ 6.70m-Nov. 3/08) | | | | | | | |



# 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<br>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

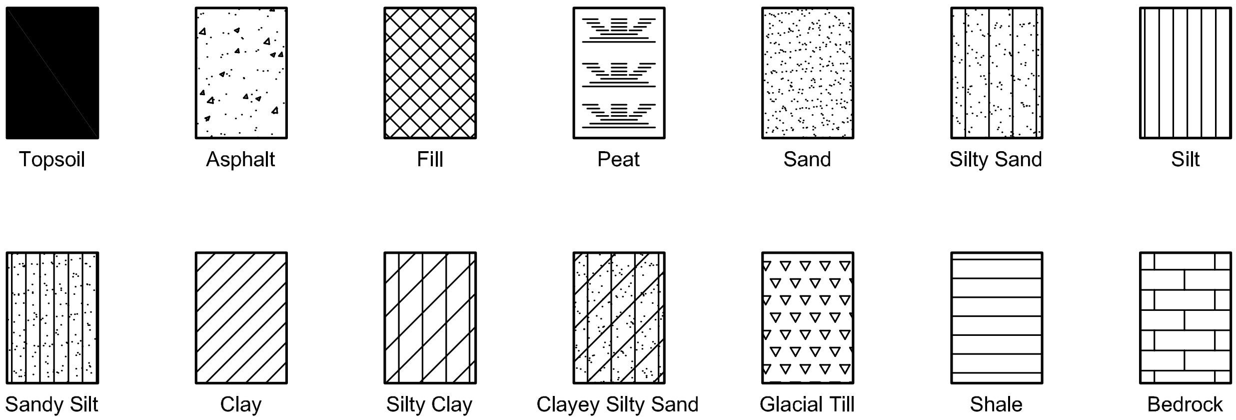
| | | |
|---|---|---|
| $p'_o$ | - | Present effective overburden pressure at sample depth |
| $p'_c$ | - | Preconsolidation pressure of (maximum past pressure on) sample |
| Ccr | - | Recompression index (in effect at pressures below $p'_c$) |
| Cc | - | Compression index (in effect at pressures above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

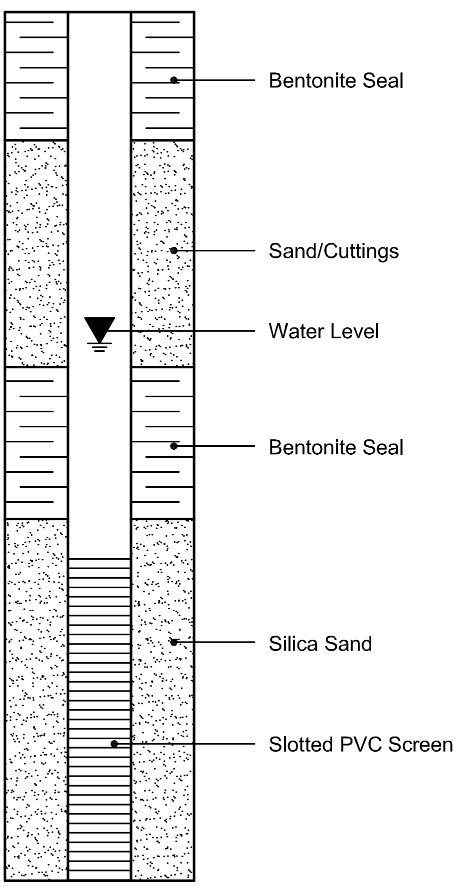
## SYMBOLS AND TERMS (continued)

### STRATA PLOT

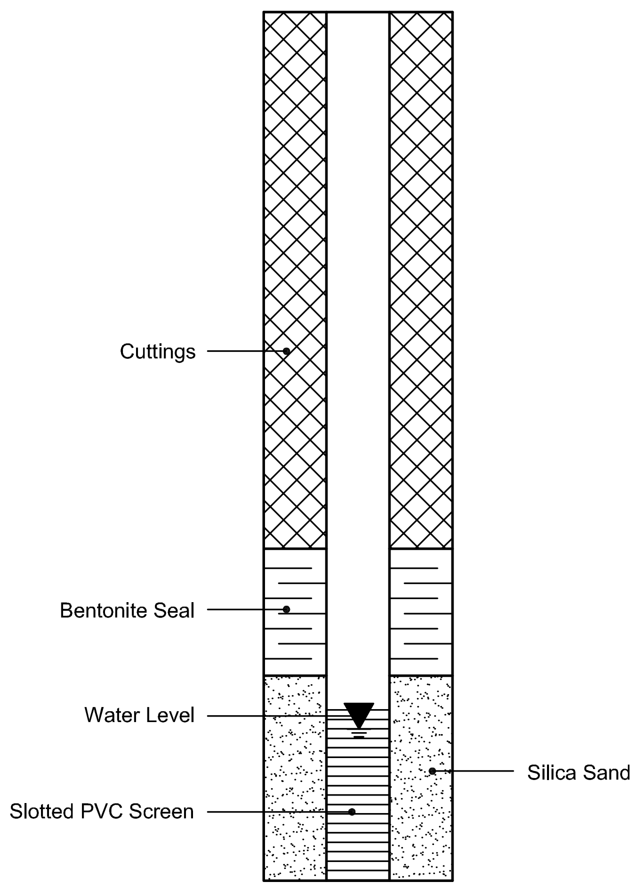



### MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION




PIEZOMETER CONSTRUCTION



PARACEL

| Order #: 0844027 |
|---|

*Certificate of Analysis*

Report Date: 31-Oct-2008

Order Date: 27-Oct-2008

Client: **Paterson Group Consulting Engineers**

Client PO: 7283

Project Description: PG1773

| | MDL/Units | | | | |
|---|---|---|---|---|---|
| Client ID: | | BH6 SS3 | - | - | - |
| Sample Date: | | 27-Oct-08 | - | - | - |
| Sample ID: | | 0844027-01 | - | - | - |
| | MDL/Units | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 77.3 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 7.42 | - | - | - |
| Resistivity | 0.10 Ohm.m | 39.8 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 12 | - | - | - |
| Sulphate | 5 ug/g dry | 27 | - | - | - |

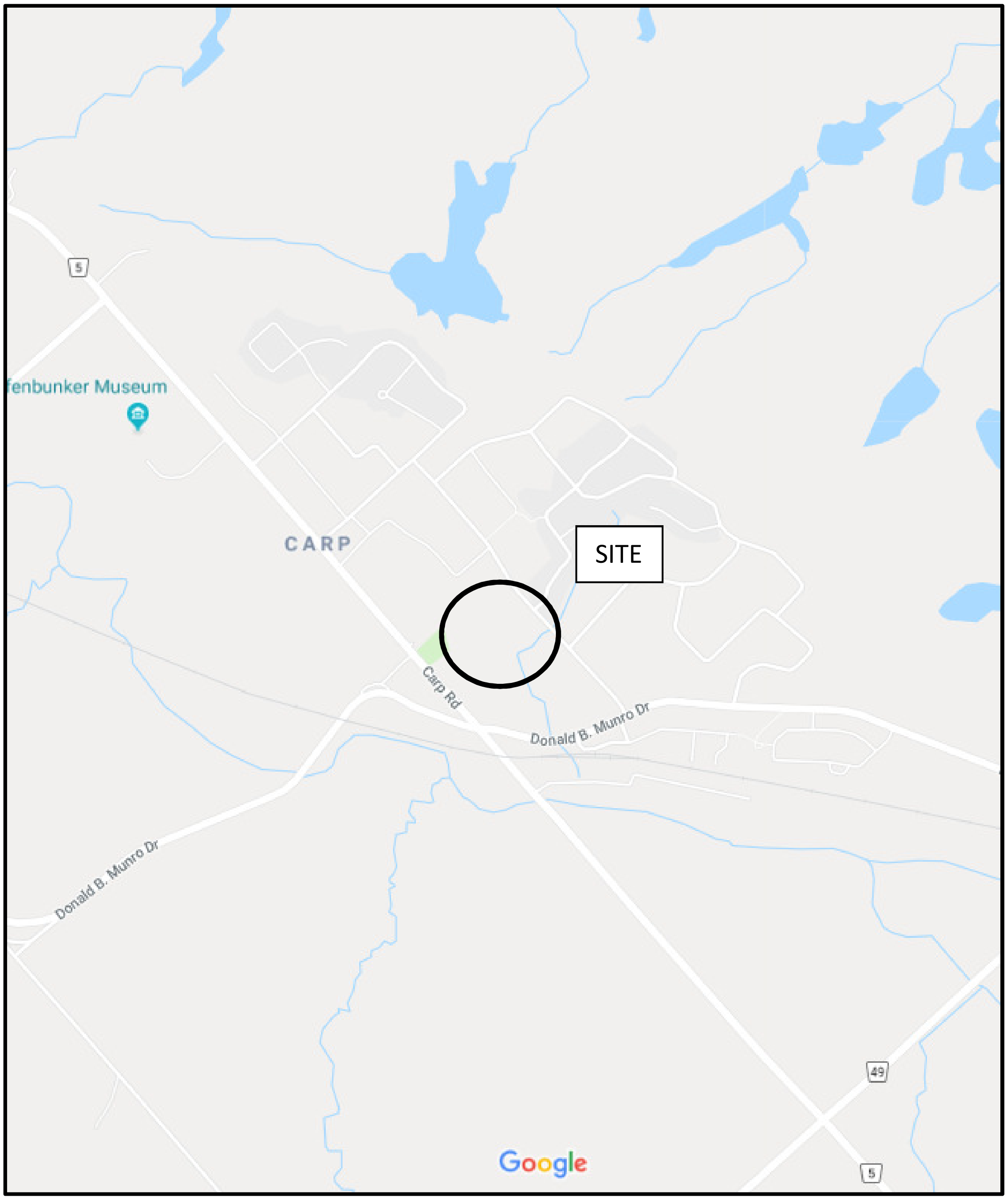
# APPENDIX 2

**FIGURE 1 - KEY PLAN**

**FIGURES 2 TO 23 - SLOPE STABILITY ANALYSIS SECTIONS**

**DRAWING PG4918-1 - TEST HOLE LOCATION PLAN**

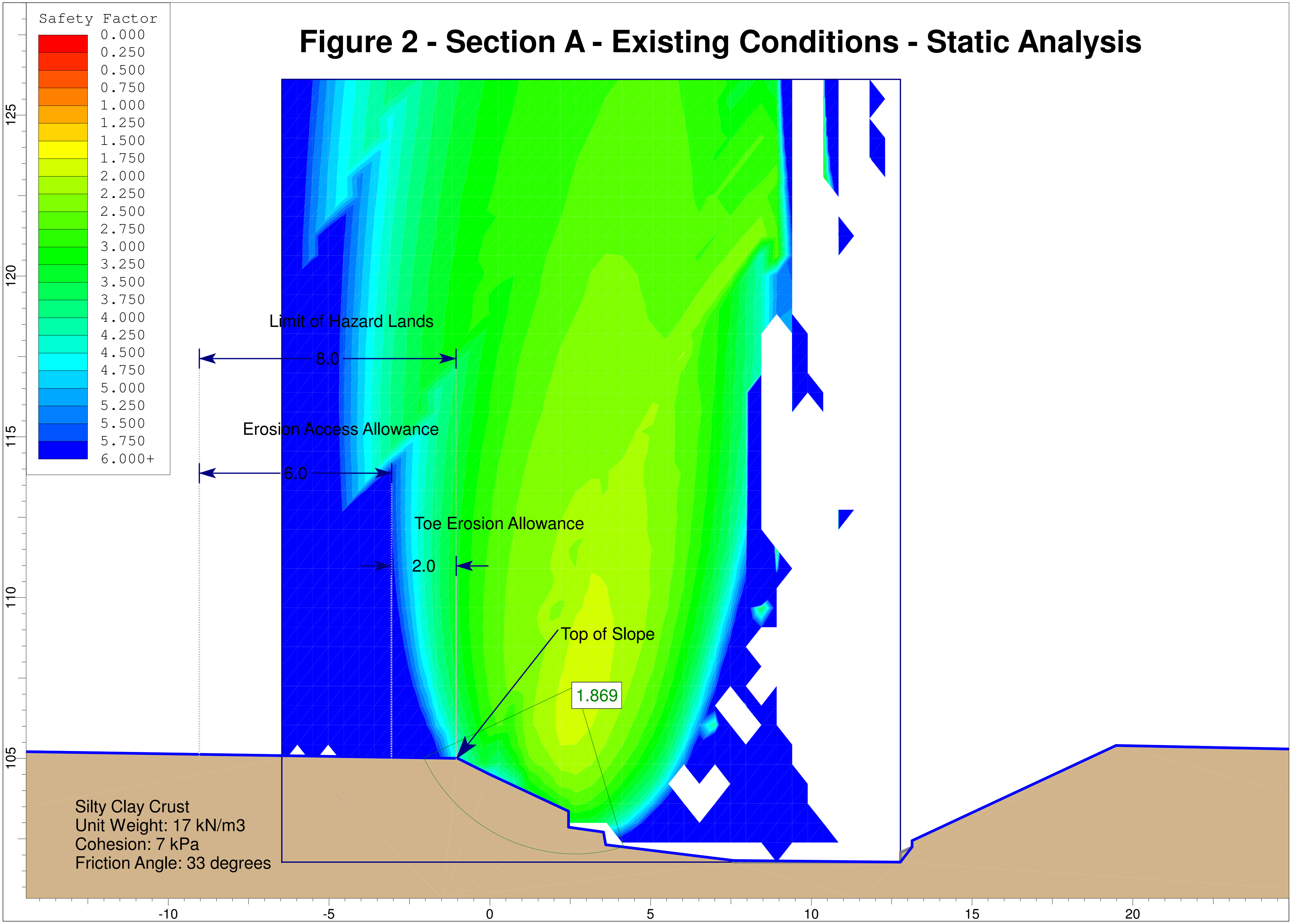
**DRAWING PG4918-2 - PERMISSIBLE GRADE RAISE PLAN**




# FIGURE 1

## KEY PLAN

patersongroup


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+
Figure 2 - Section A - Existing Conditions - Static Analysis
Limit of Hazard Lands
8.0
Erosion Access Allowance
6.0
Toe Erosion Allowance
2.0
Top of Slope
1.869
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
125
120
115
110
105
-10
-5
0
5
10
15
20

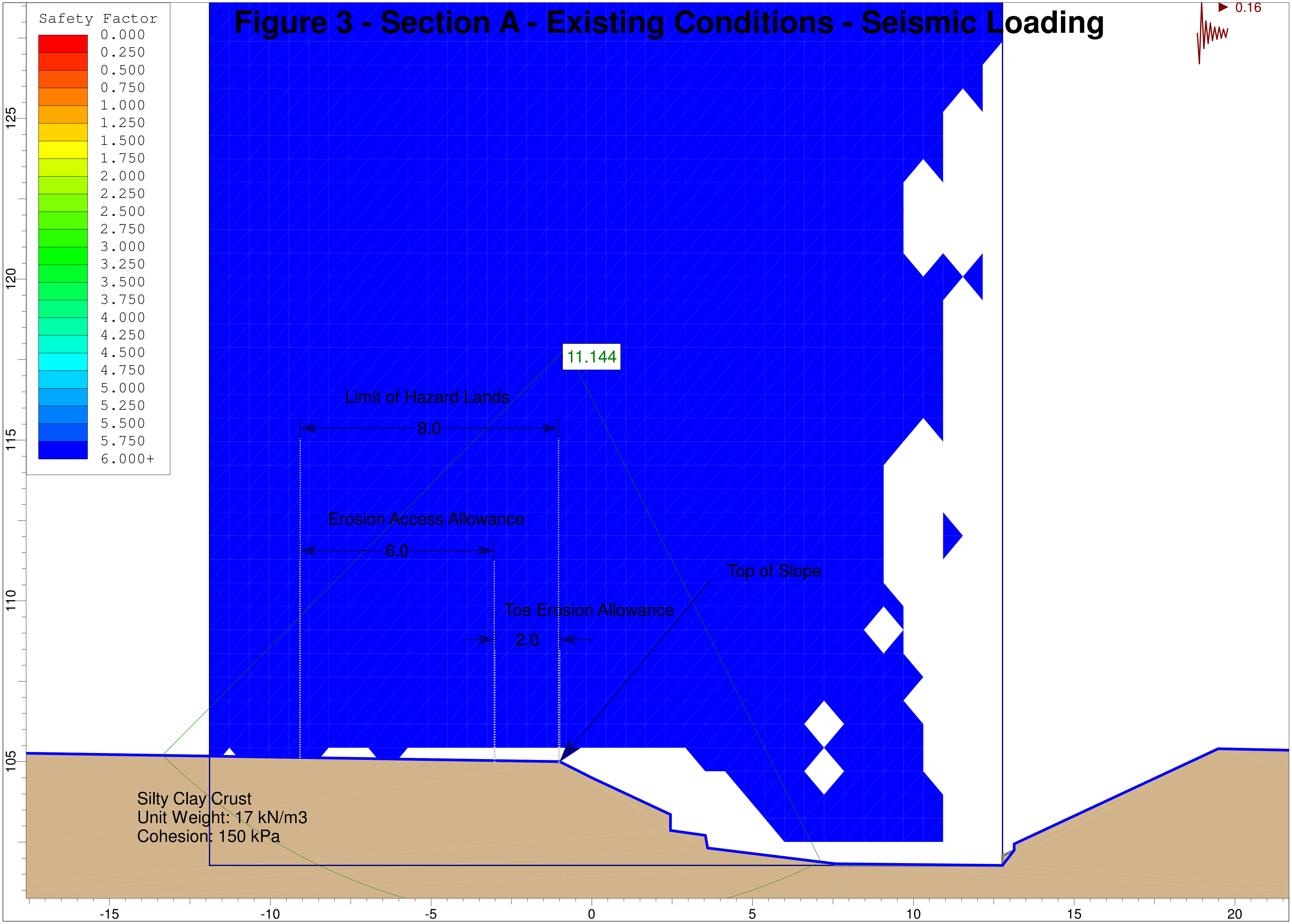


Figure 3 - Section A - Existing Conditions - Seismic Loading
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
11.144
Limit of Hazard Lands
8.0
Erosion Access Allowance
6.0
Top of Slope
Toe Erosion Allowance
2.0
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 150 kPa
125
120
115
110
105
-15
-10
-5
0
5
10
15
20

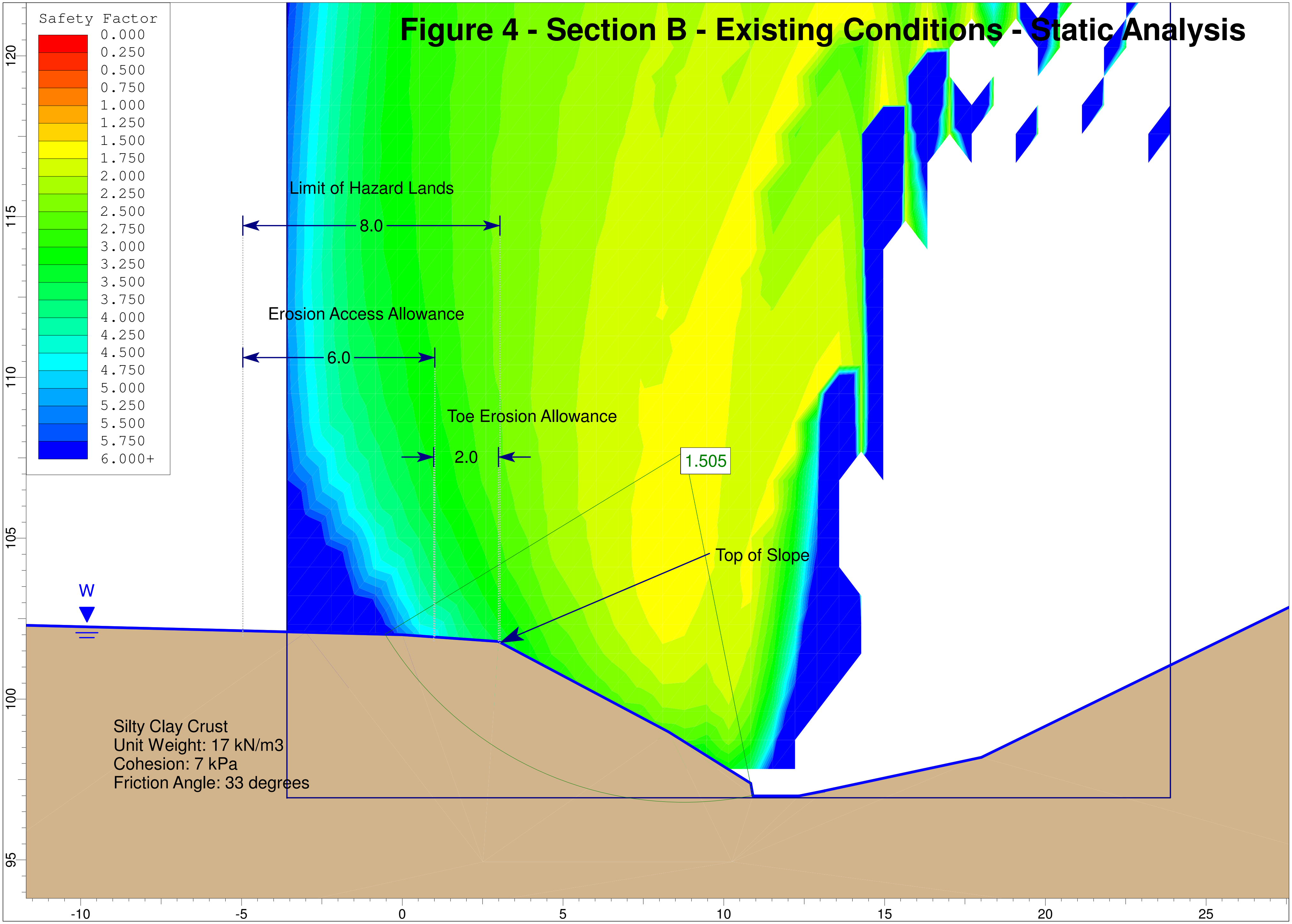


Figure 4 - Section B - Existing Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
8.0
Erosion Access Allowance
6.0
Toe Erosion Allowance
2.0
1.505
Top of Slope
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
120
115
110
105
100
95
-10
-5
0
5
10
15
20
25

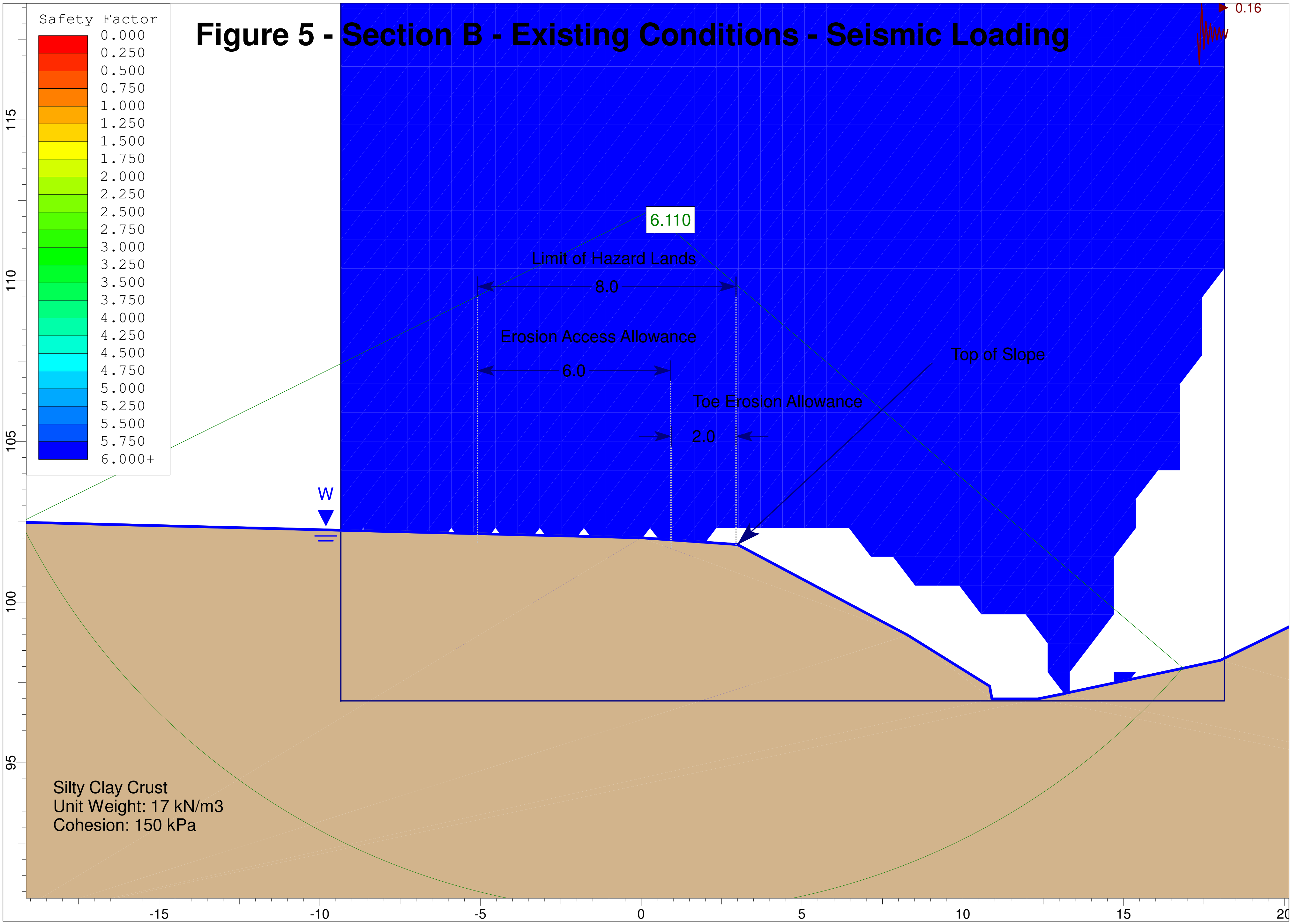
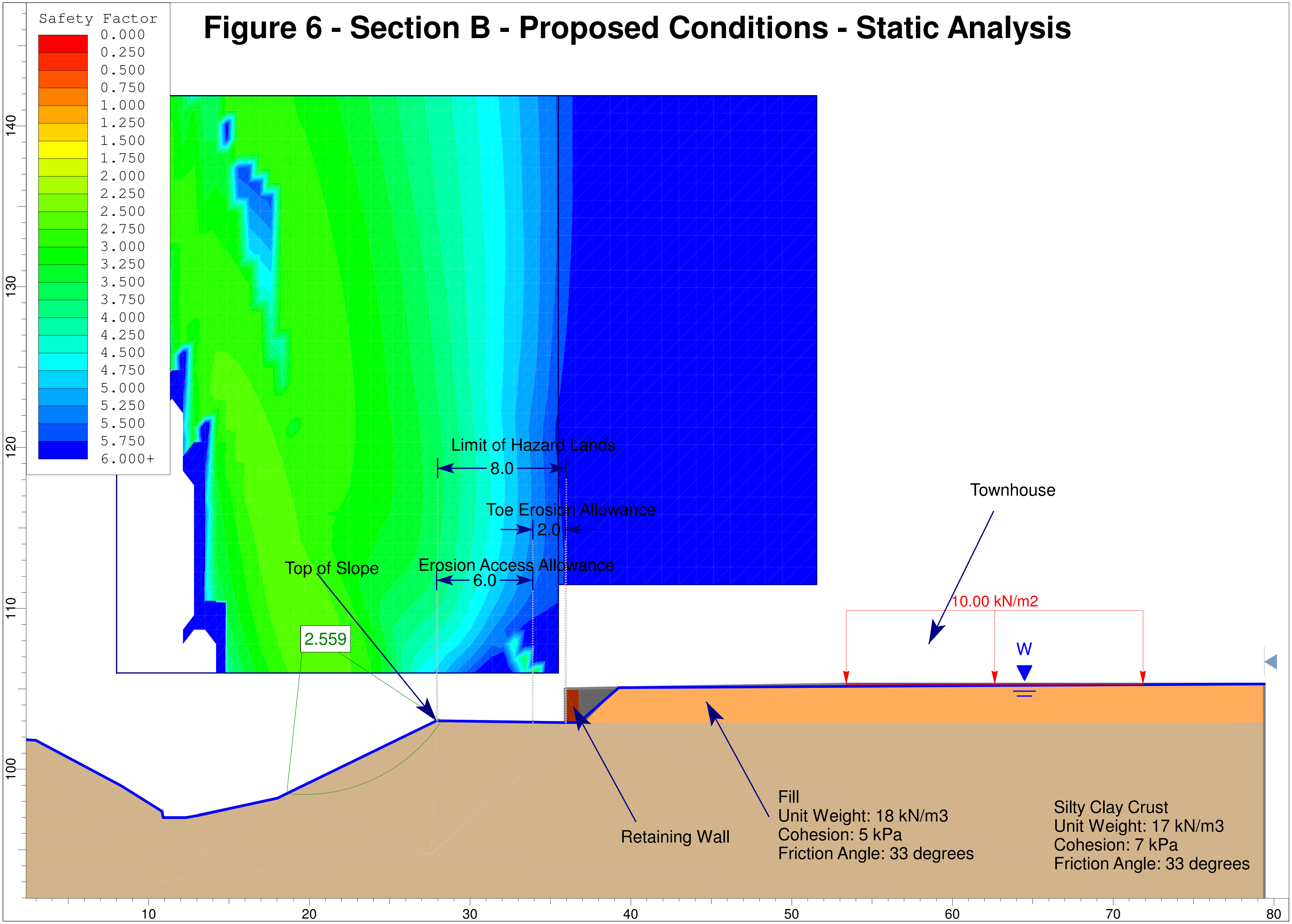
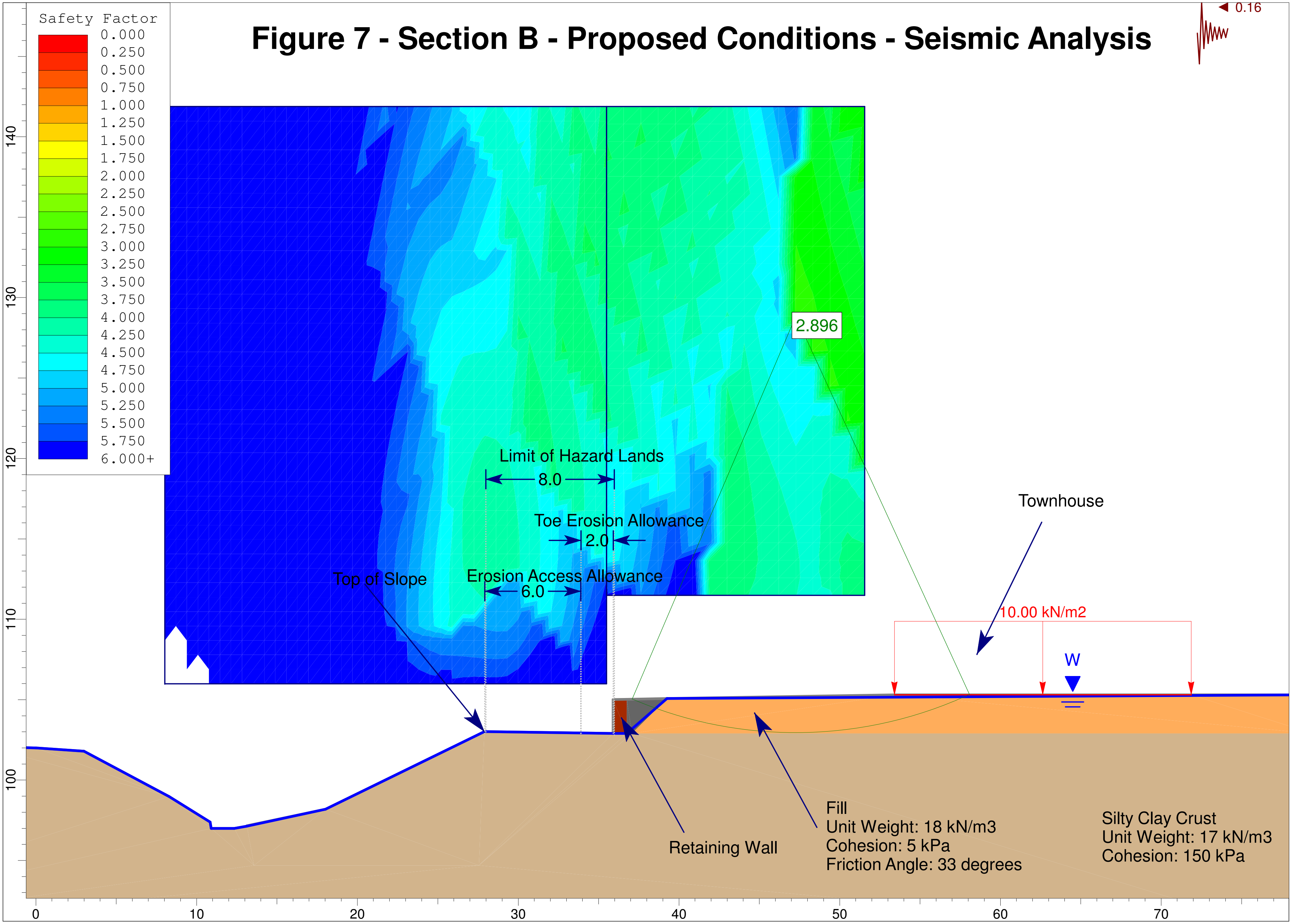


Figure 5 - Section B - Existing Conditions - Seismic Loading
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
6.110
Limit of Hazard Lands
8.0
Erosion Access Allowance
6.0
Top of Slope
Toe Erosion Allowance
2.0
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 150 kPa
115
110
105
100
95
-15
-10
-5
0
5
10
15
20

# Figure 6 - Section B - Proposed Conditions - Static Analysis



# Figure 7 - Section B - Proposed Conditions - Seismic Analysis



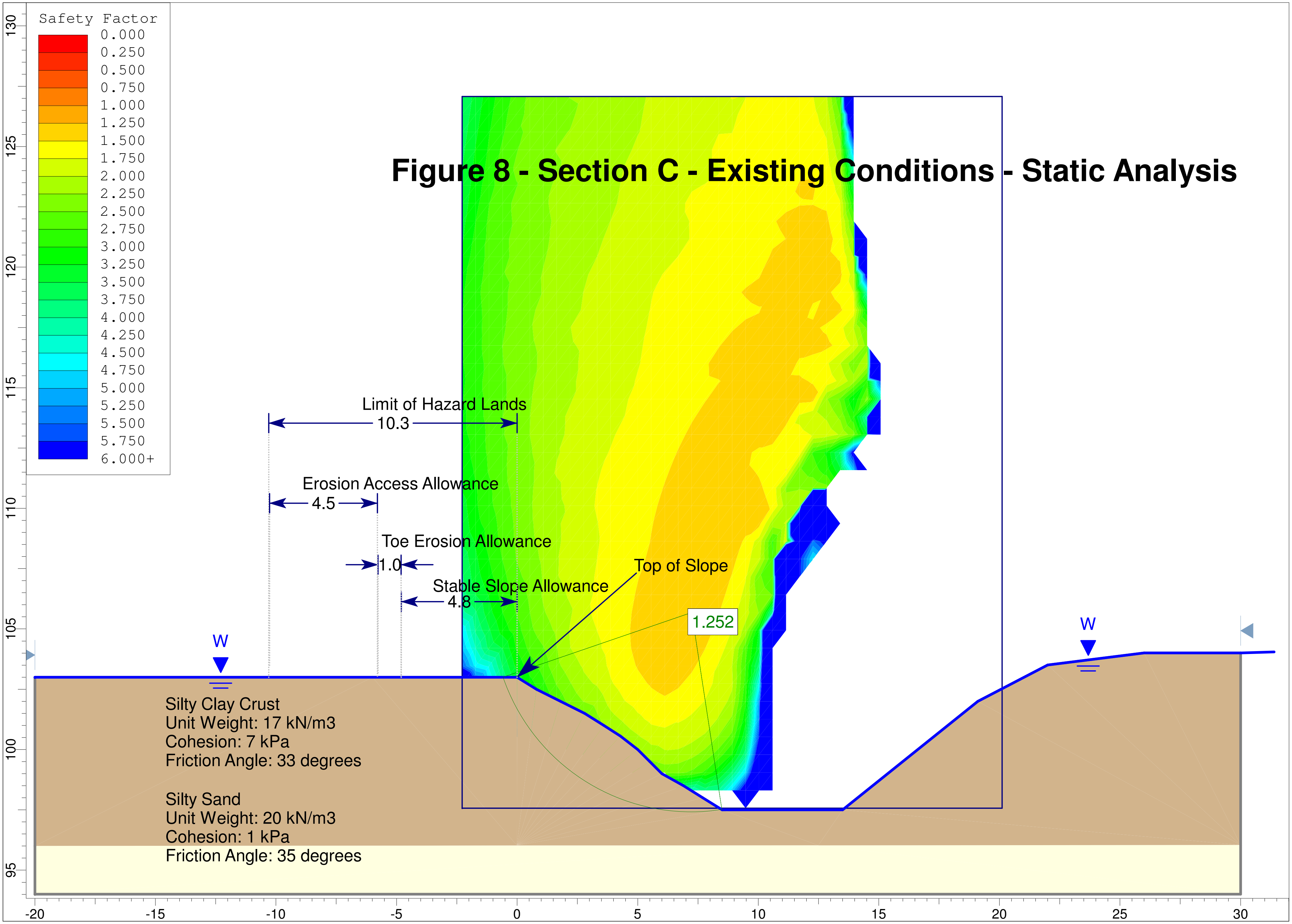


Figure 8 - Section C - Existing Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
10.3
Erosion Access Allowance
4.5
Toe Erosion Allowance
1.0
Stable Slope Allowance
4.8
Top of Slope
1.252
W
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 35 degrees
130
125
120
115
110
105
100
95
-20
-15
-10
-5
0
5
10
15
20
25
30

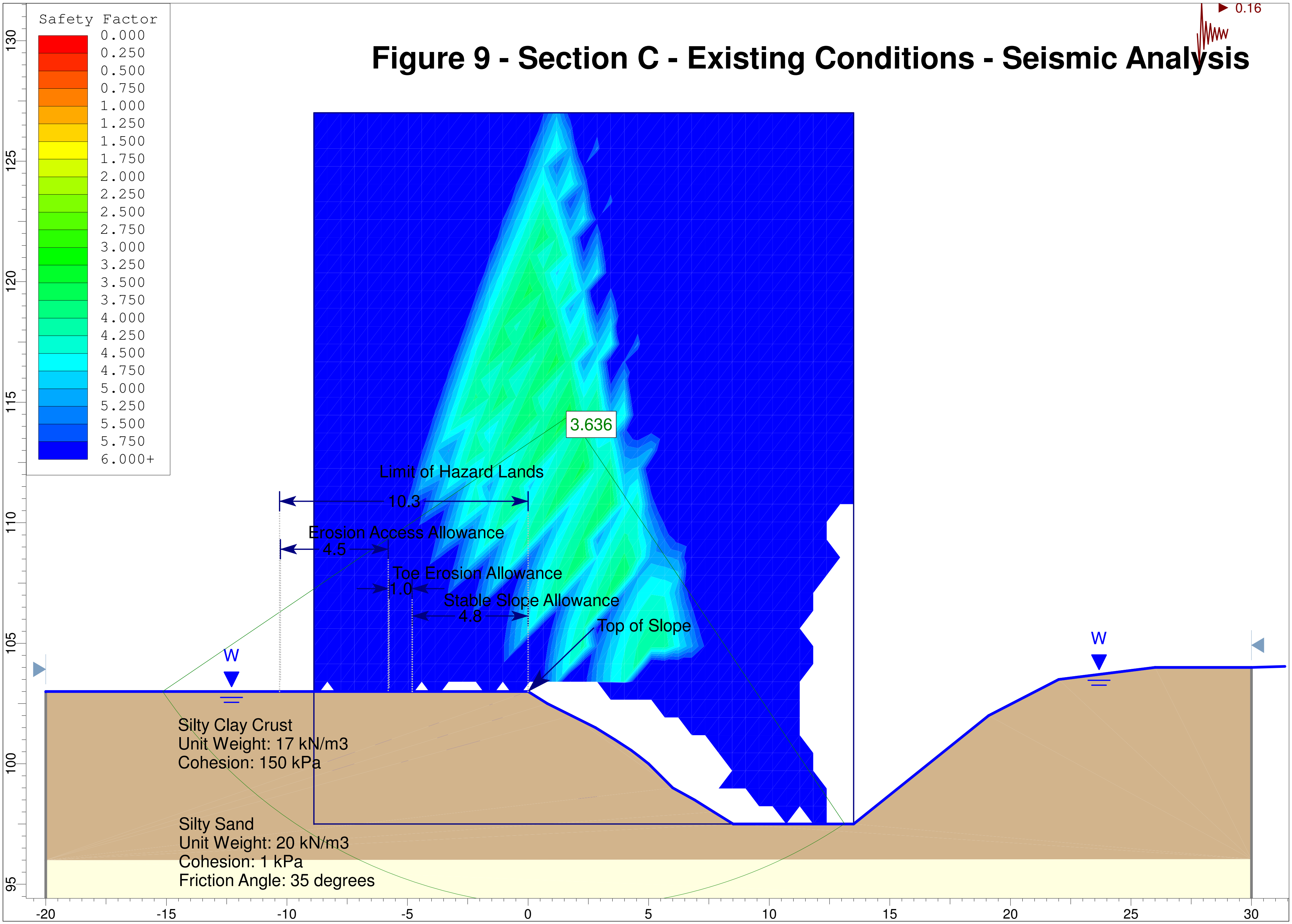
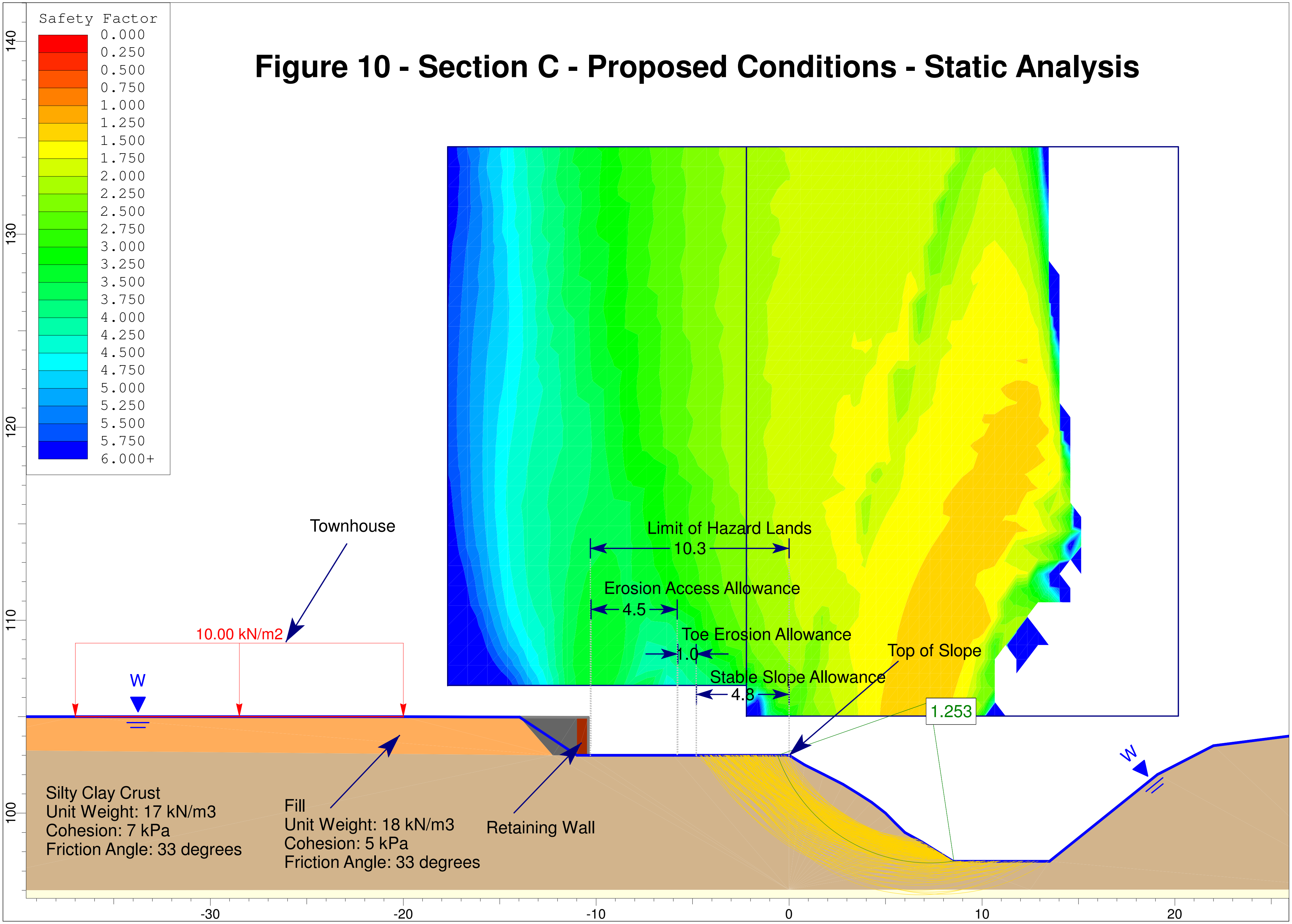


Figure 9 - Section C - Existing Conditions - Seismic Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
3.636
Limit of Hazard Lands
10.3
Erosion Access Allowance
4.5
Toe Erosion Allowance
1.0
Stable Slope Allowance
4.8
Top of Slope
W
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 150 kPa
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 35 degrees
130
125
120
115
110
105
100
95
-20
-15
-10
-5
0
5
10
15
20
25
30

# Figure 10 - Section C - Proposed Conditions - Static Analysis



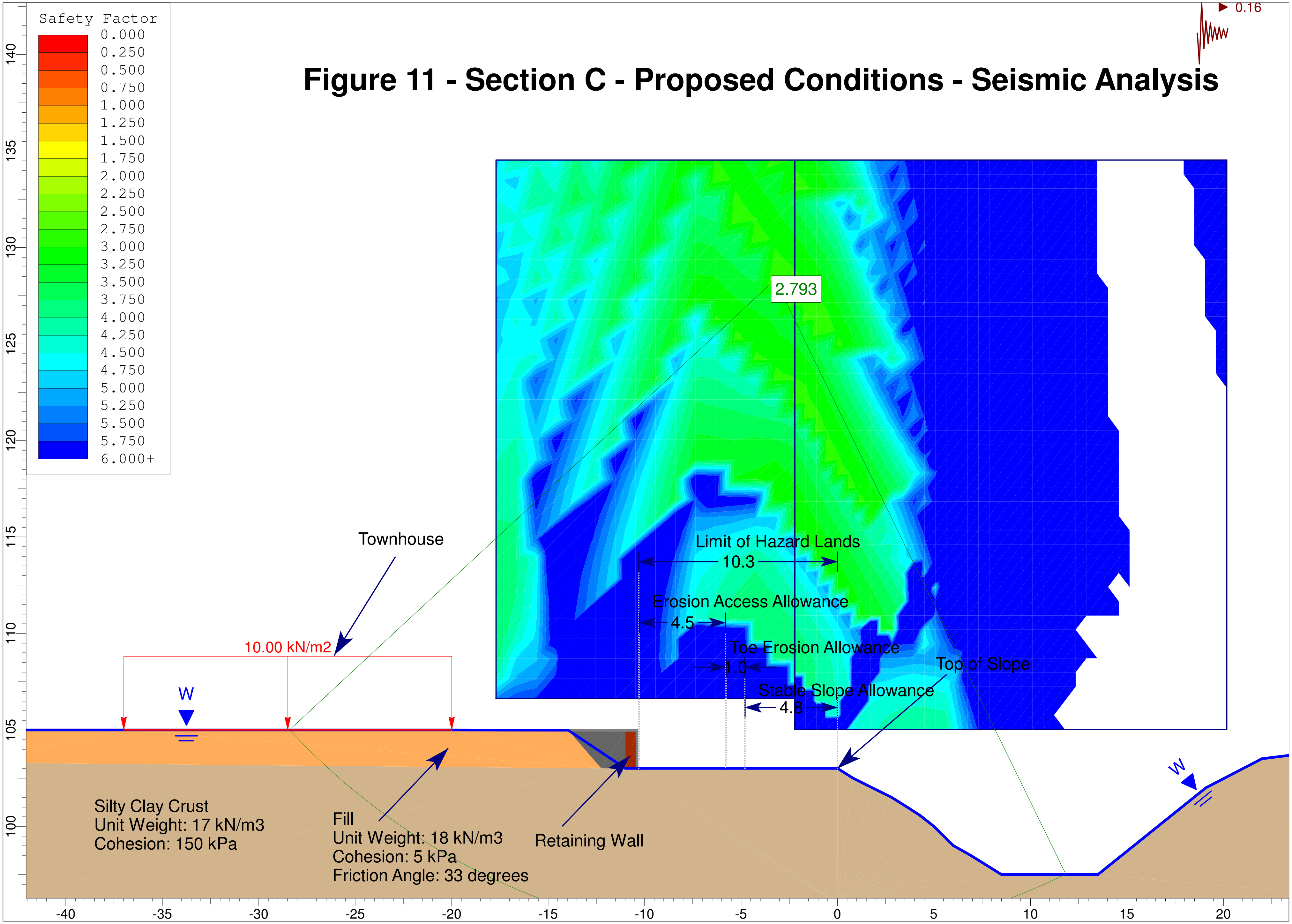


Figure 11 - Section C - Proposed Conditions - Seismic Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
2.793
Townhouse
10.00 kN/m2
W
Limit of Hazard Lands
10.3
Erosion Access Allowance
4.5
Toe Erosion Allowance
1.0
Stable Slope Allowance
4.8
Top of Slope
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 150 kPa
Fill
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Retaining Wall
140
135
130
125
120
115
110
105
100
-40
-35
-30
-25
-20
-15
-10
-5
0
5
10
15
20

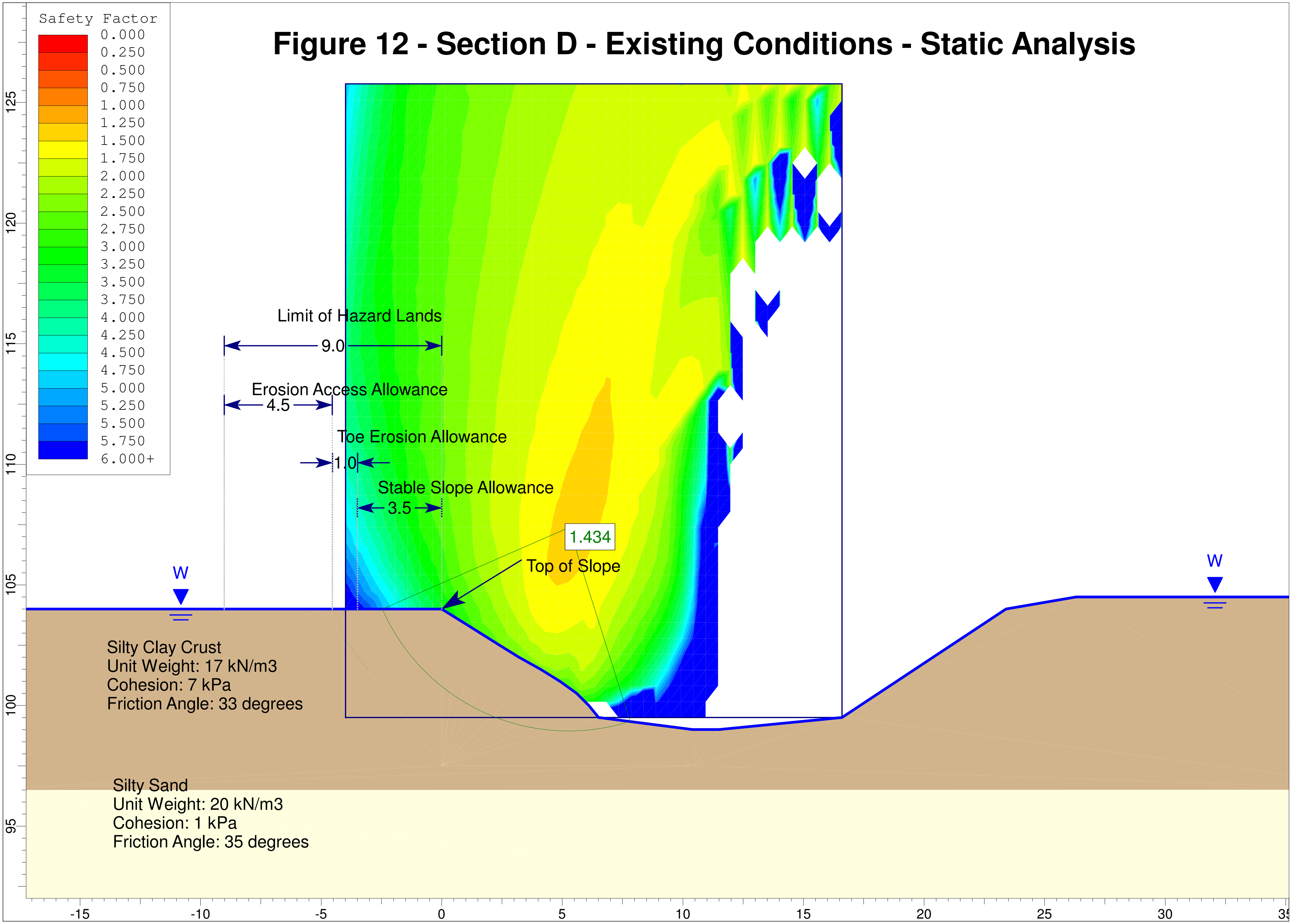
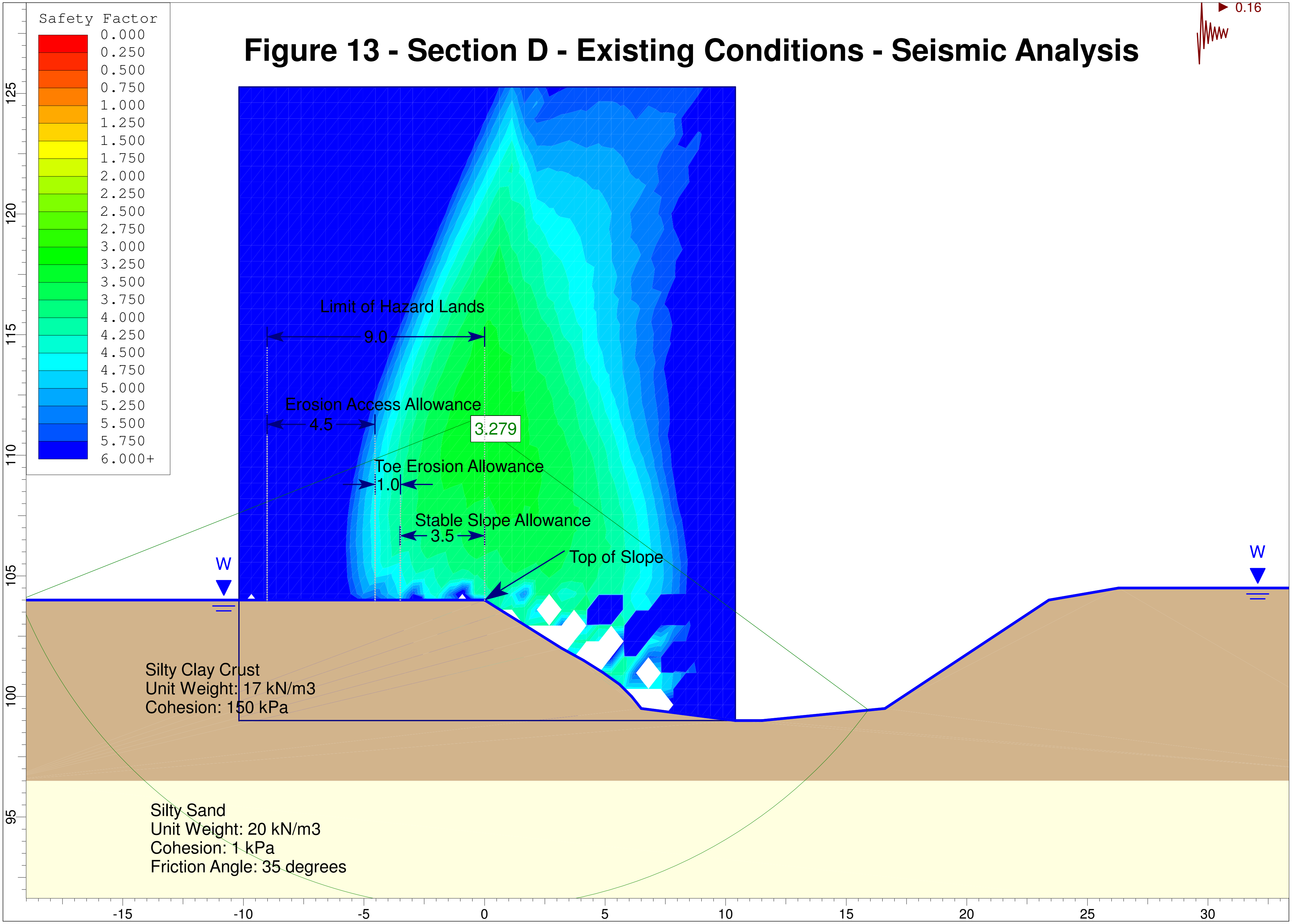


Figure 12 - Section D - Existing Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
9.0
Erosion Access Allowance
4.5
Toe Erosion Allowance
1.0
Stable Slope Allowance
3.5
1.434
Top of Slope
W
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 35 degrees
125
120
115
110
105
100
95
-15
-10
-5
0
5
10
15
20
25
30
35

# Figure 13 - Section D - Existing Conditions - Seismic Analysis



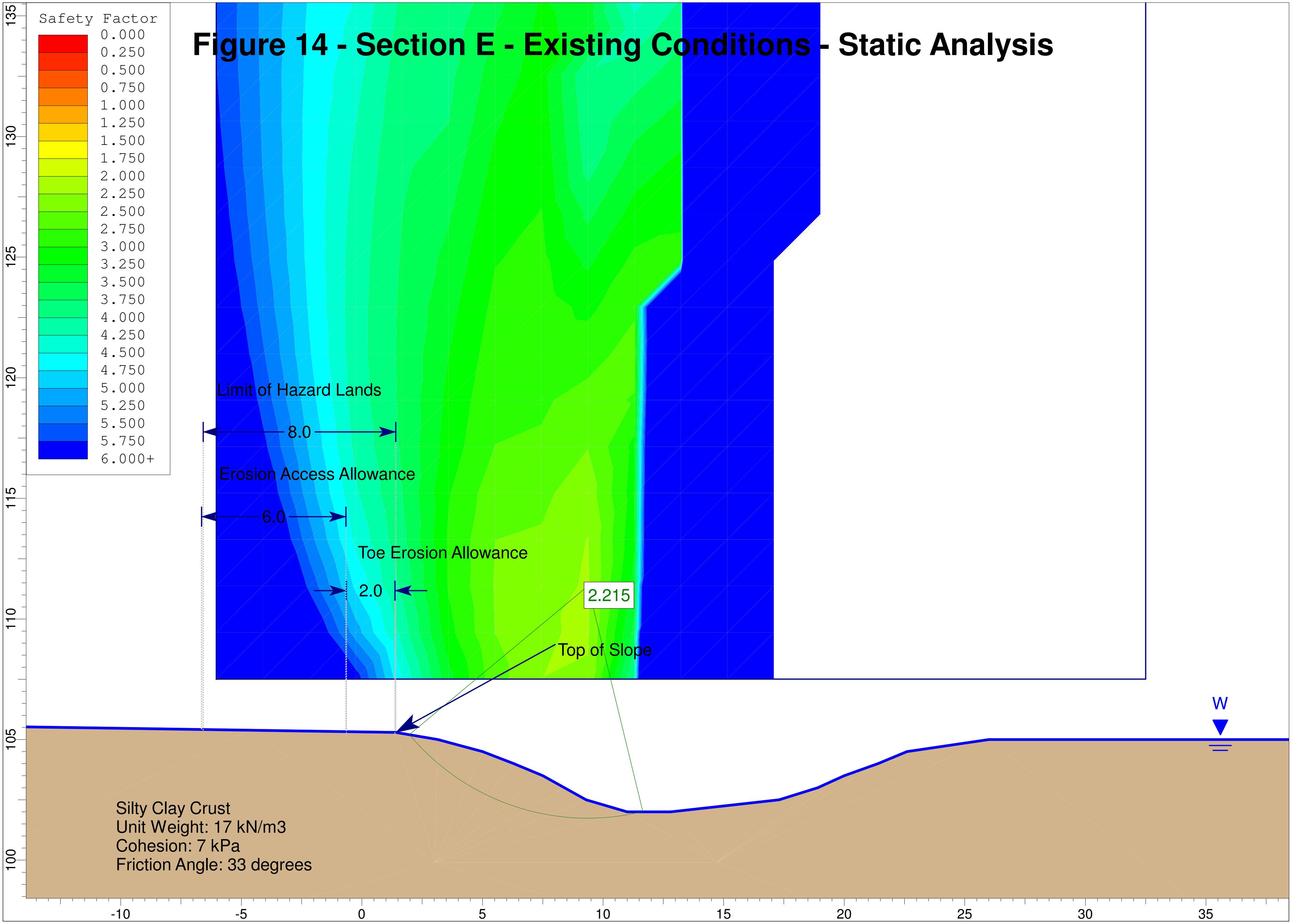


Figure 14 - Section E - Existing Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
8.0
Erosion Access Allowance
6.0
Toe Erosion Allowance
2.0
2.215
Top of Slope
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
135
130
125
120
115
110
105
100
-10
-5
0
5
10
15
20
25
30
35

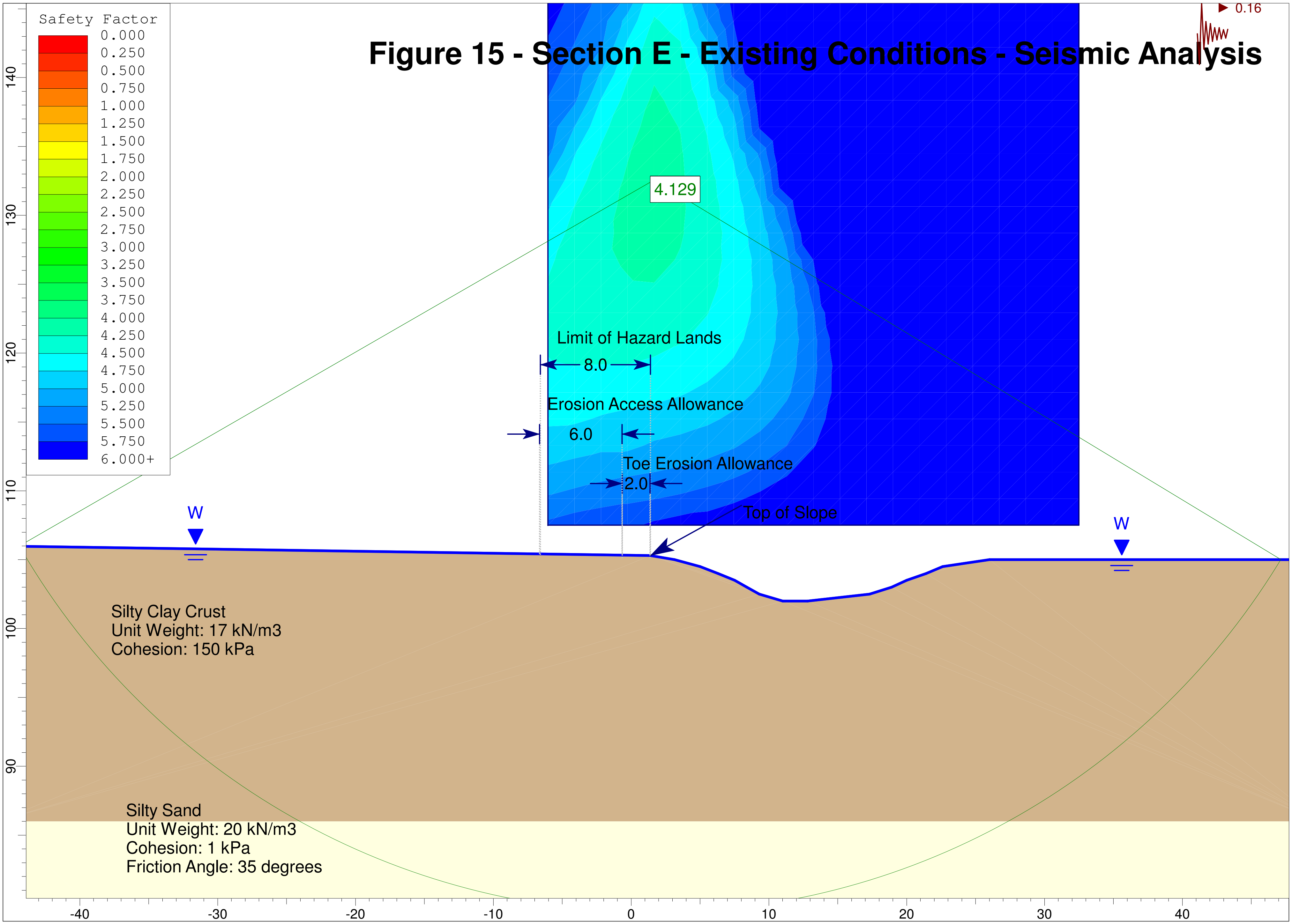
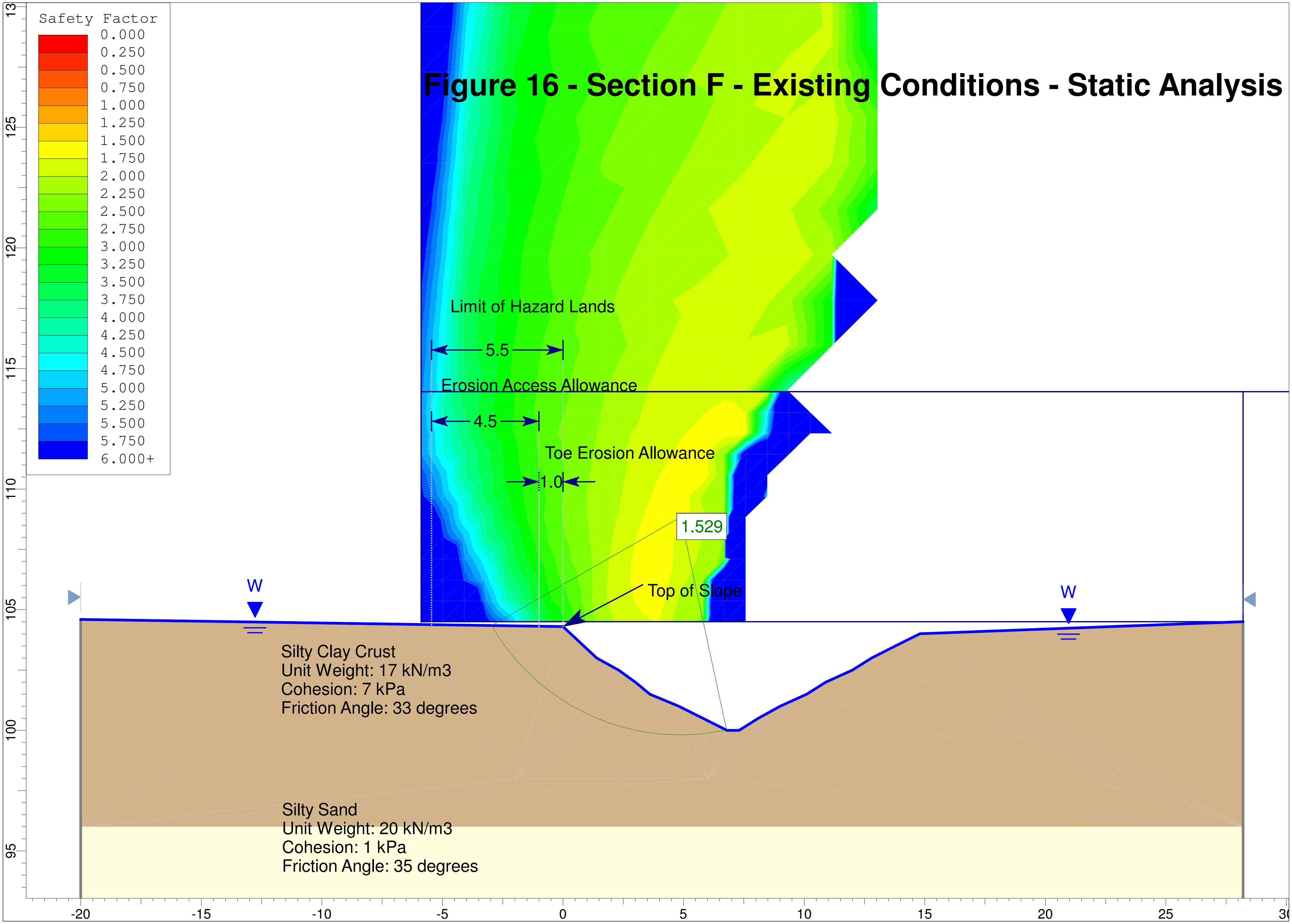
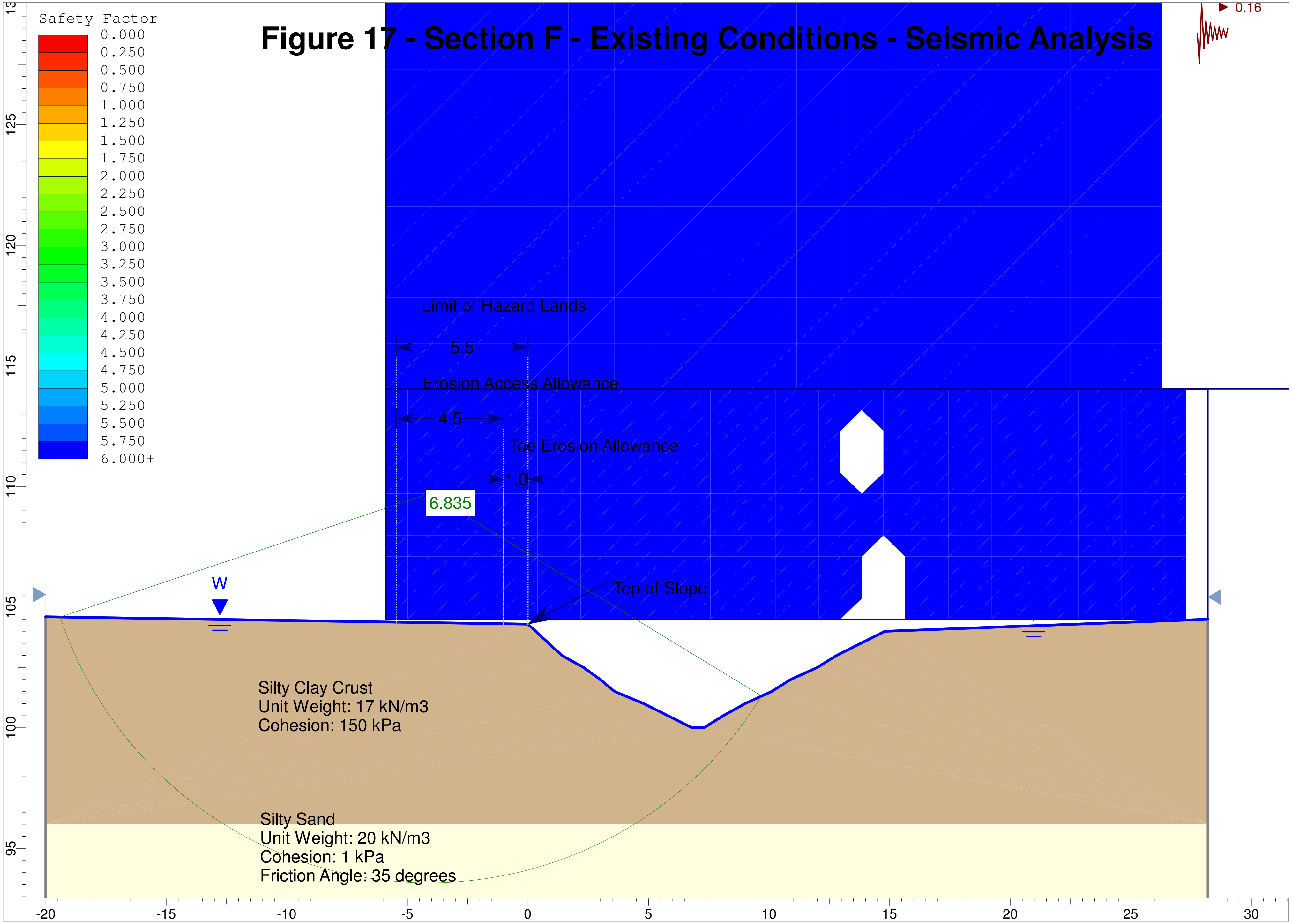


Figure 15 - Section E - Existing Conditions - Seismic Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
4.129
Limit of Hazard Lands
8.0
Erosion Access Allowance
6.0
Toe Erosion Allowance
2.0
Top of Slope
W
W
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 150 kPa
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 35 degrees
140
130
120
110
100
90
-40
-30
-20
-10
0
10
20
30
40

# Figure 16 - Section F - Existing Conditions - Static Analysis



# Figure 17 - Section F - Existing Conditions - Seismic Analysis



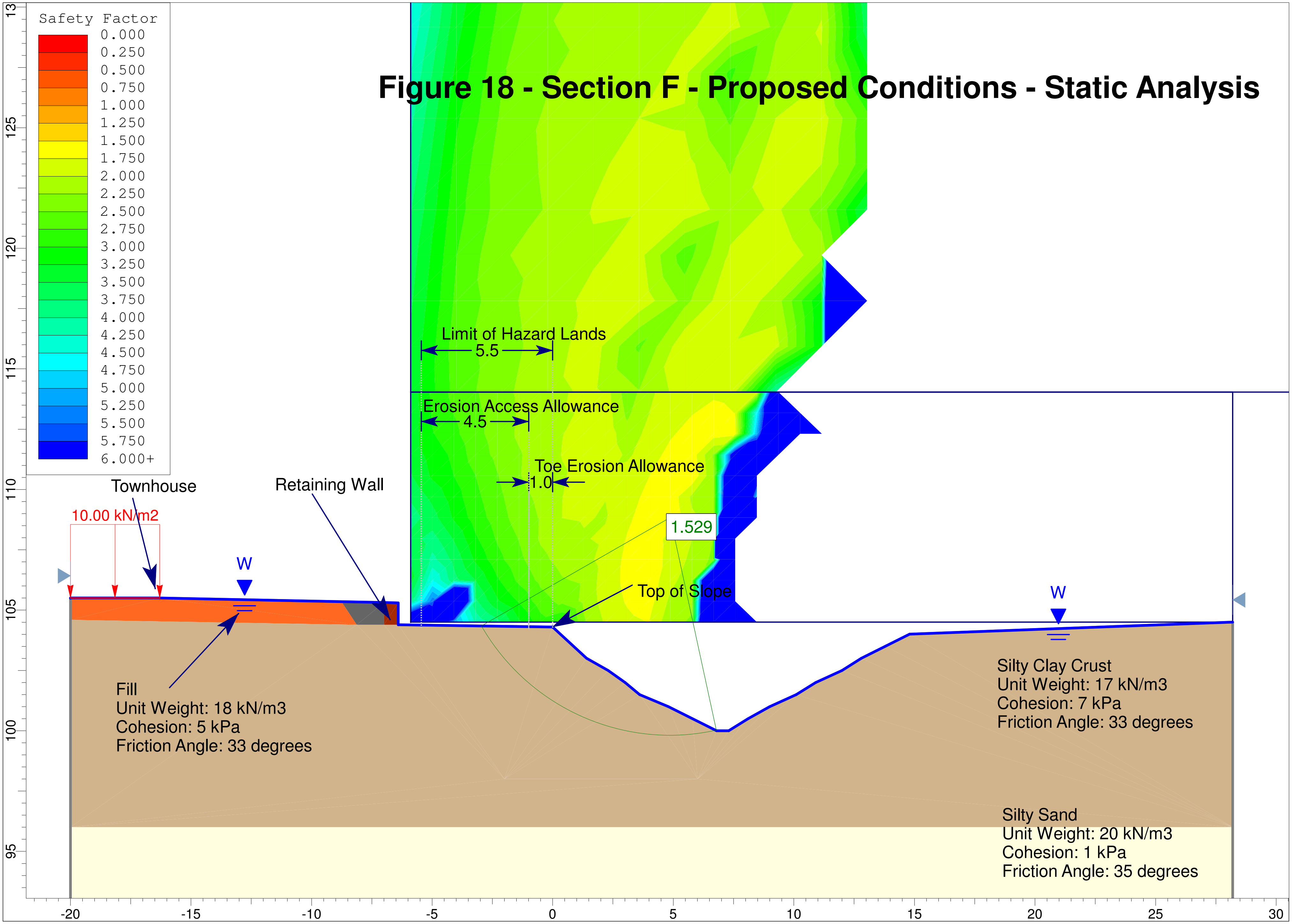


Figure 18 - Section F - Proposed Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
5.5
Erosion Access Allowance
4.5
Toe Erosion Allowance
1.0
1.529
Top of Slope
Townhouse
Retaining Wall
10.00 kN/m2
W
W
Fill
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 35 degrees
130
125
120
115
110
105
100
95
-20
-15
-10
-5
0
5
10
15
20
25
30

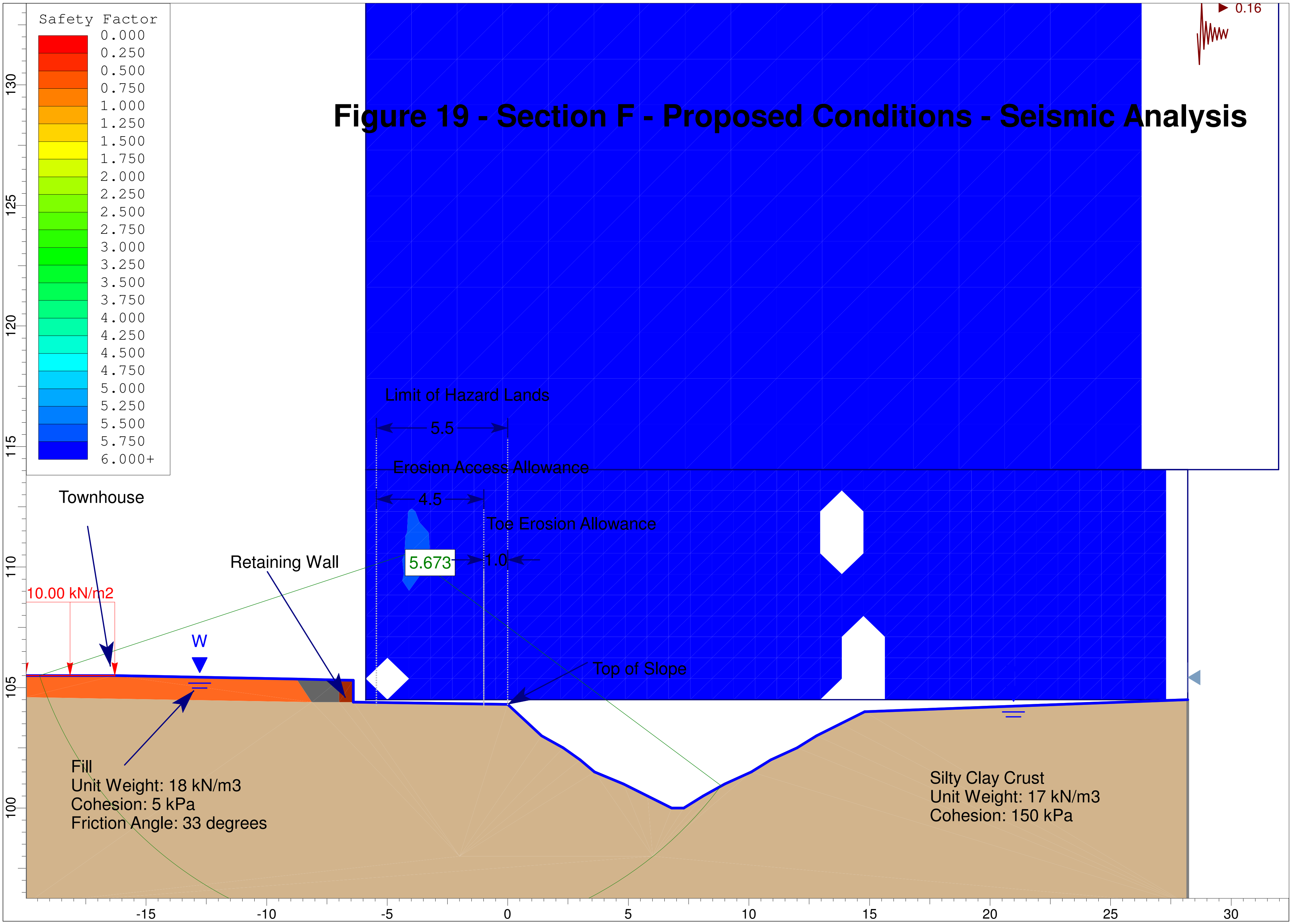


Figure 19 - Section F - Proposed Conditions - Seismic Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
Limit of Hazard Lands
5.5
Erosion Access Allowance
4.5
Toe Erosion Allowance
5.673
1.0
Townhouse
Retaining Wall
10.00 kN/m2
W
Top of Slope
Fill
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 150 kPa
130
125
120
115
110
105
100
-15
-10
-5
0
5
10
15
20
25
30

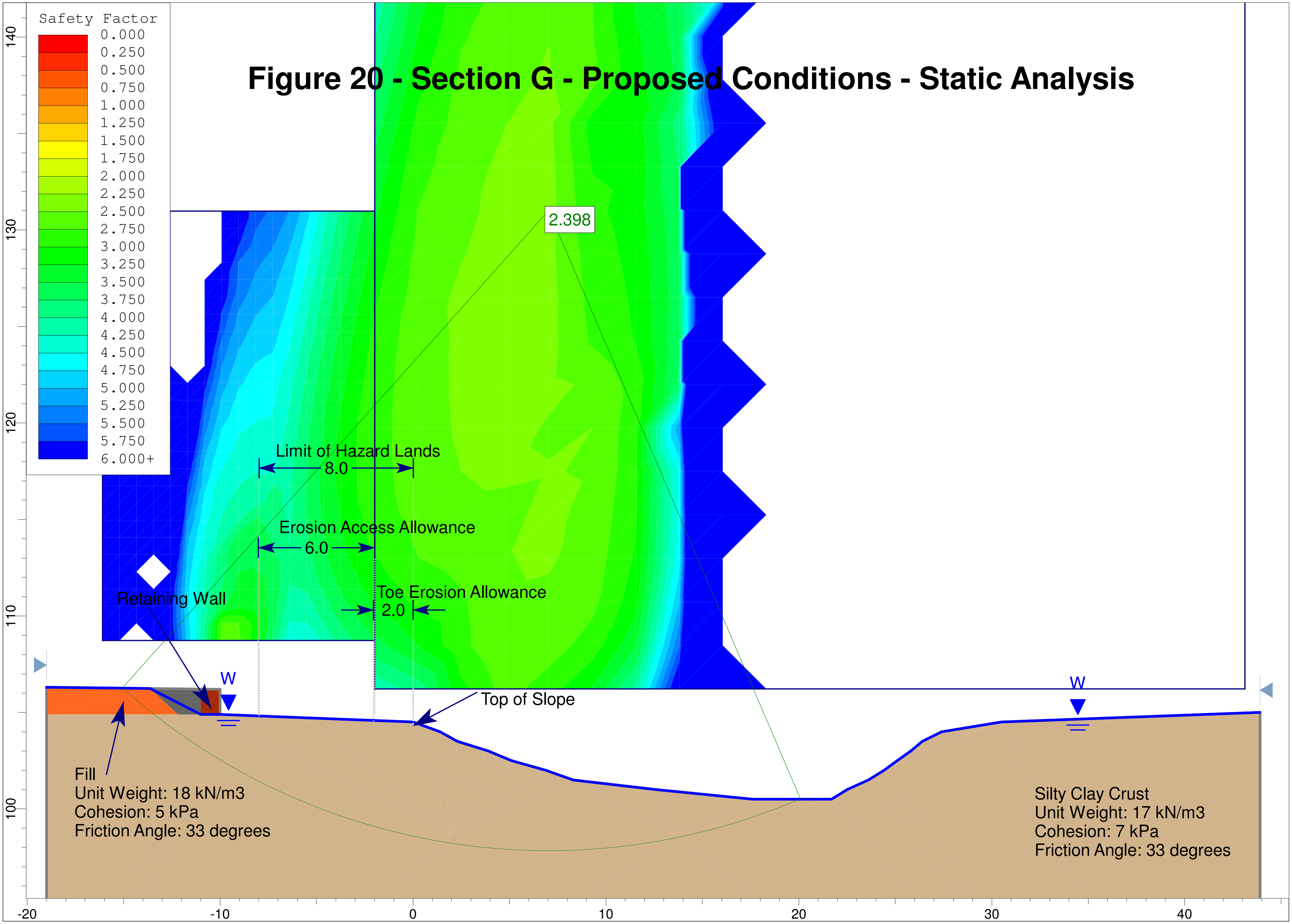


Figure 20 - Section G - Proposed Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
2.398
Limit of Hazard Lands
8.0
Erosion Access Allowance
6.0
Toe Erosion Allowance
2.0
Retaining Wall
W
Top of Slope
W
Fill
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
140
130
120
110
100
-20
-10
0
10
20
30
40

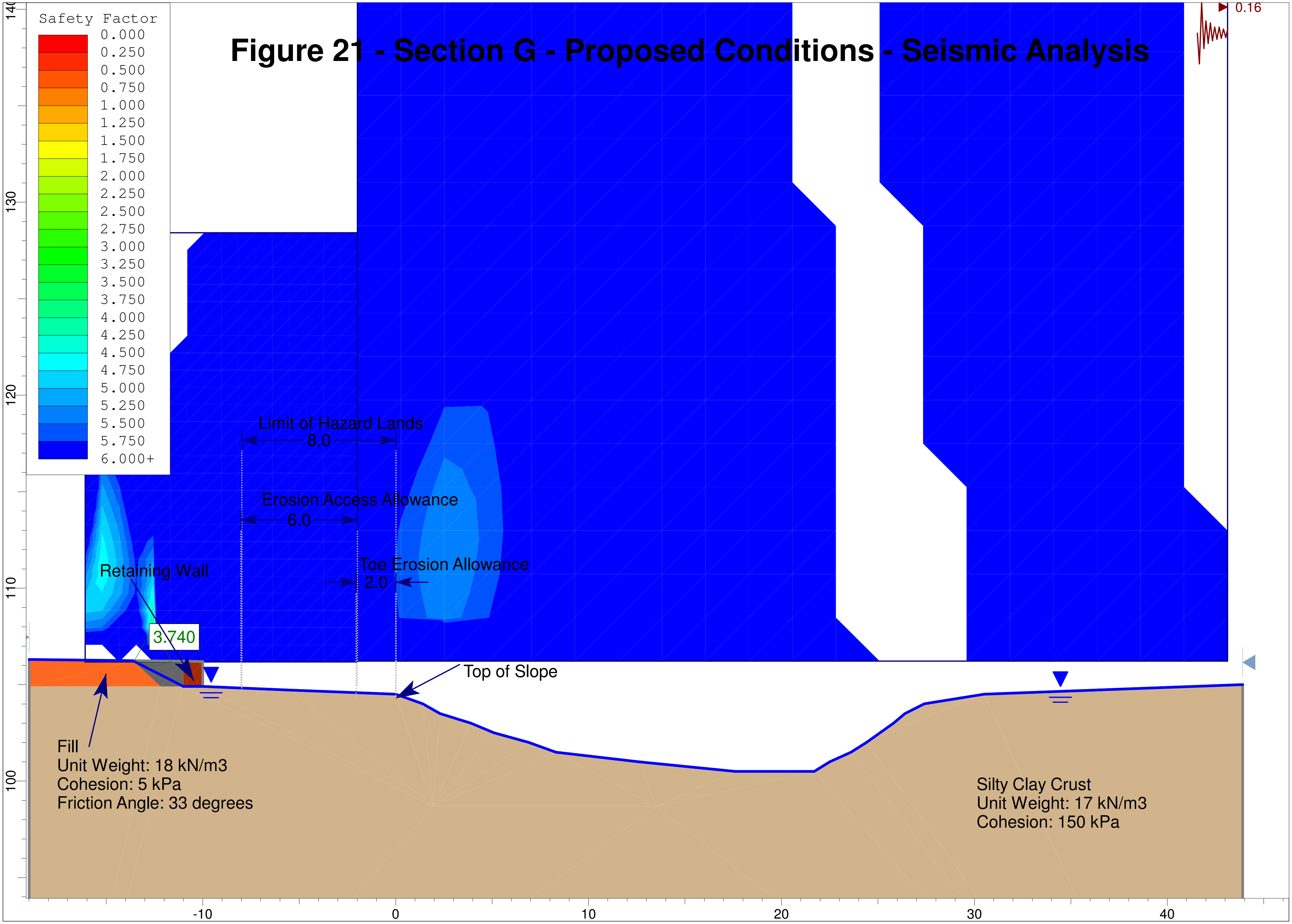


Figure 21 - Section G - Proposed Conditions - Seismic Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
Limit of Hazard Lands
8.0
Erosion Access Allowance
6.0
Toe Erosion Allowance
2.0
Retaining Wall
3.740
Top of Slope
Fill
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 150 kPa
140
130
120
110
100
-10
0
10
20
30
40

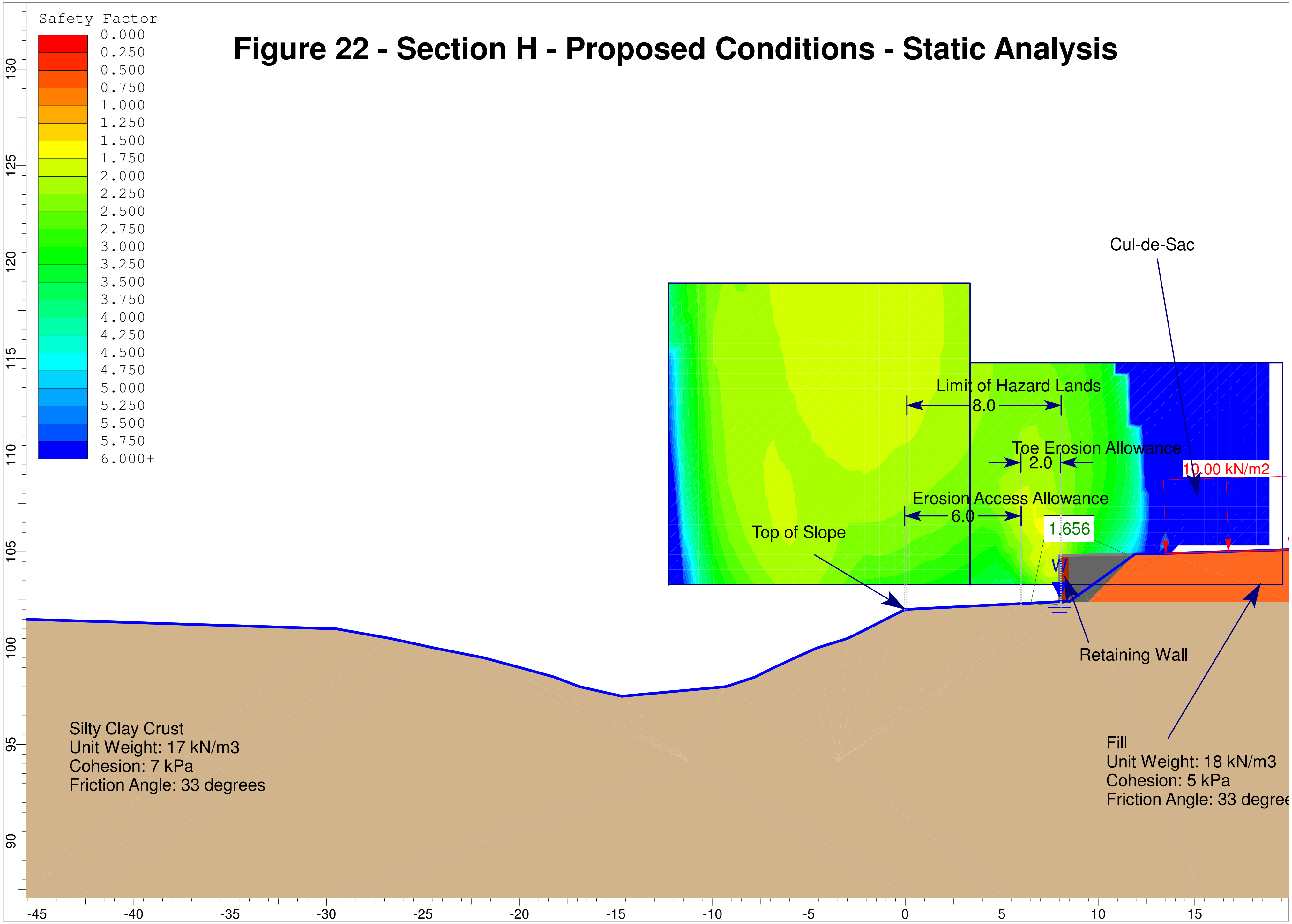
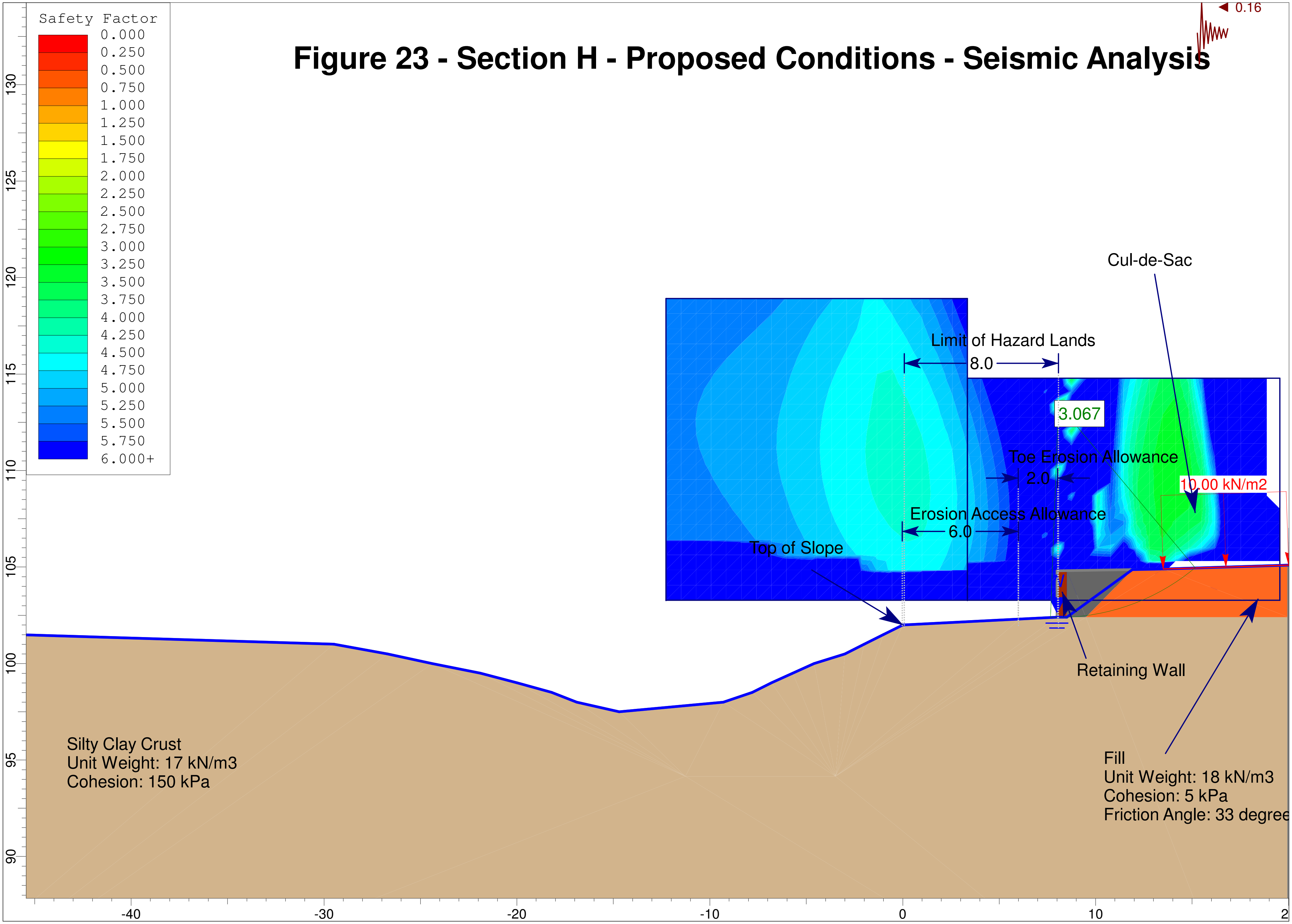
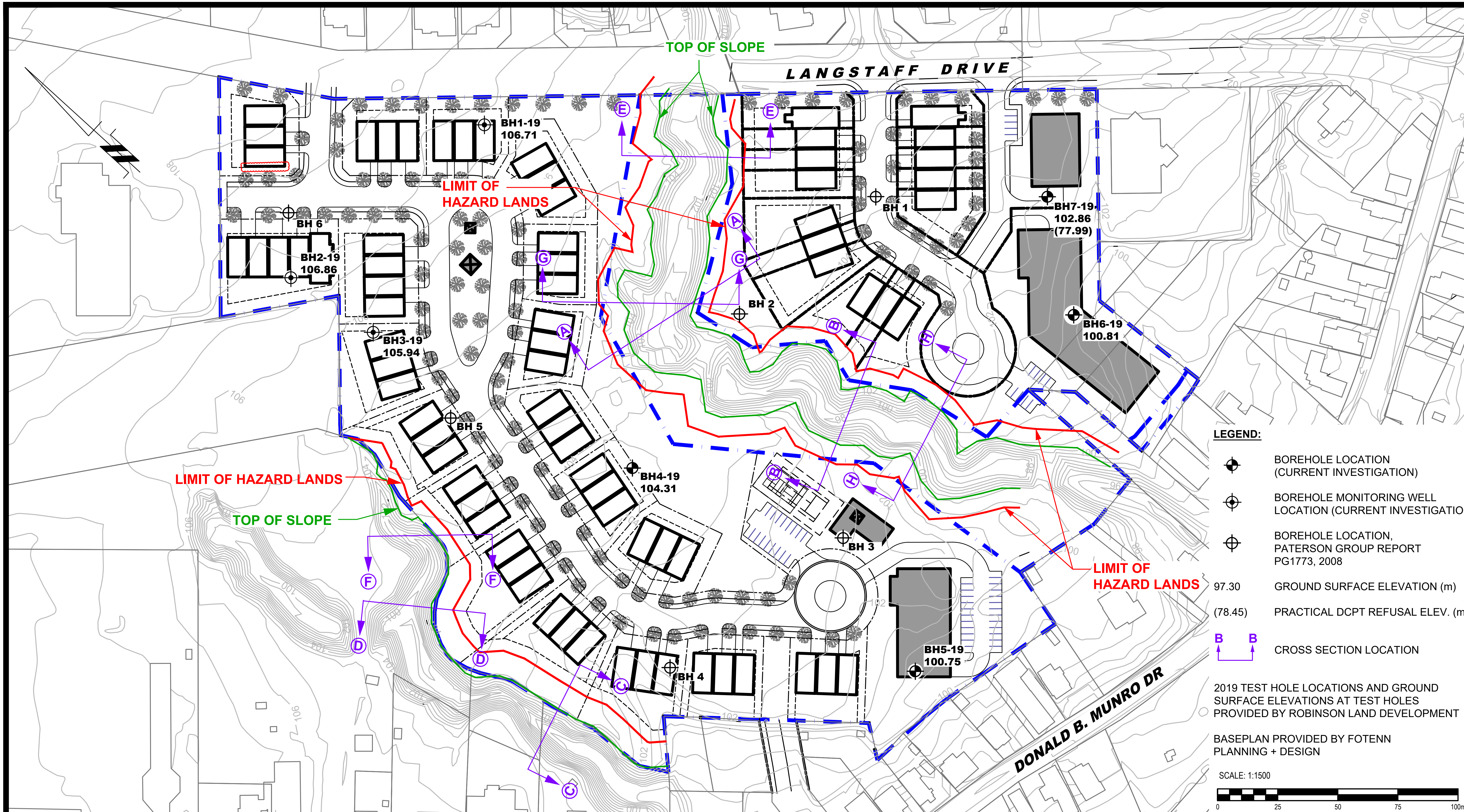


Figure 22 - Section H - Proposed Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Cul-de-Sac
Limit of Hazard Lands
8.0
Toe Erosion Allowance
2.0
10.00 kN/m2
Erosion Access Allowance
6.0
1.656
Top of Slope
W
Retaining Wall
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
Fill
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degree
130
125
120
115
110
105
100
95
90
-45
-40
-35
-30
-25
-20
-15
-10
-5
0
5
10
15

# Figure 23 - Section H - Proposed Conditions - Seismic Analysis



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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 5 | ADDITIONAL CROSS SECTIONS ADDED | 15/01/2020 | SD |
| 4 | UPDATED CONCEPTUAL PLAN | 23/10/2019 | SD |
| 3 | REVISED LIMIT OF HAZARD LANDS | 11/10/2019 | SD |
| 2 | UPDATED CONCEPTUAL PLAN | 30/07/2019 | SD |
| 1 | RAVINE AND TOP OF SLOPE TOPOGRAPHY ADDED TO PLAN | 06/06/2019 | RG |

INVERNESS HOMES
GEOTECHNICAL INVESTIGATION - PROPOSED RESIDENTIAL DEVELOPMENT
147 LANGSTAFF DRIVE
CARP, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

| | |
|---|---|
| Scale: 1:1500 | Date: 08/2019 |
| Drawn by: RCG | Report No.: PG4918-LET.01 |
| Checked by: SD | Dwg. No.: **PG4918-1** |
| Approved by: AJT | Revision No.: 5 |

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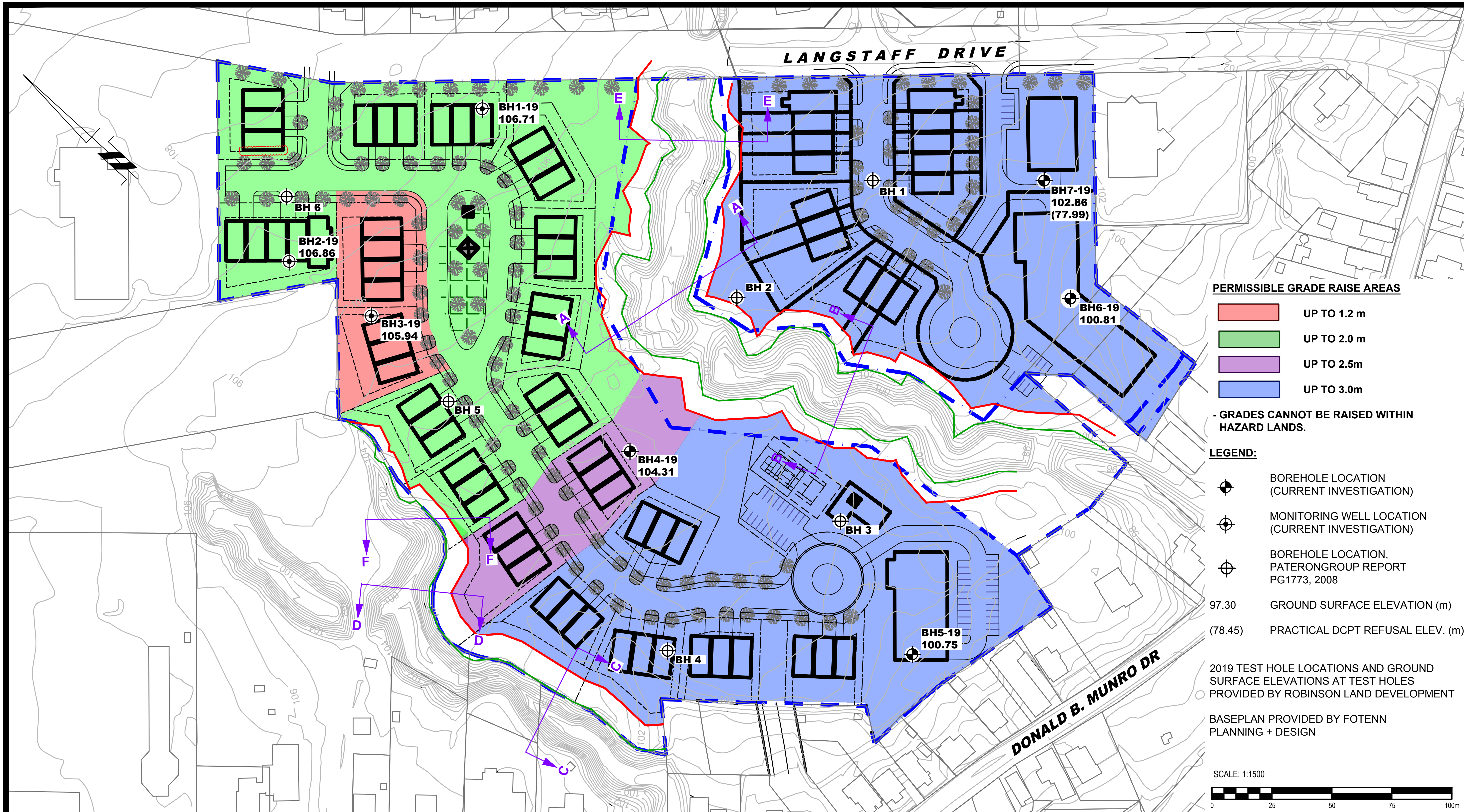


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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 4 | UPDATED CONCEPTUAL PLAN | 23/10/2019 | SD |
| 3 | REVISED LIMIT OF HAZARD LANDS | 11/10/2019 | SD |
| 2 | UPDATED CONCEPTUAL PLAN | 30/07/2019 | SD |
| 1 | RAVINE AND TOP OF SLOPE TOPOGRAPHY ADDED TO PLAN | 06/06/2019 | RG |

INVERNESS HOMES
GEOTECHNICAL INVESTIGATION - PROPOSED RESIDENTIAL DEVELOPMENT
147 LANGSTAFF DRIVE
CARP, ONTARIO

Title:
# PERMISSIBLE GRADE RAISE PLAN

| | |
|---|---|
| Scale: 1:1500 | Date: 08/2019 |
| Drawn by: RCG | Report No.: PG4918-1 |
| Checked by: SD | PG4918-2 |
| Approved by: DJG | Revision No.: 4 |