Geotechnical Engineering

Environmental Engineering

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Preliminary Geotechnical Investigation

Proposed Mixed Use Development
1987 Robertson Road
Ottawa, Ontario

Prepared For

The Properties Group

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Report: PG5715-1

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by The Properties Group to conduct a preliminary geotechnical investigation for the proposed mixed use development to be located at 1987 Robertson Road in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

☐ Provide preliminary 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. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Project

Detailed design plans were not available at the time of preparing this report. It is our understanding based on the latest site plans that the proposed mixed use development will consist of 5 seven-storey buildings, 1 twelve-storey building, 1 sixteen-storey building, 1 twenty-storey building, 1 twenty-four-storey building and 1 twenty-eight-storey building. Details of underground parking and basement levels were not known at the time of preparation of this report. Access lanes, parking areas, parkland and landscaped areas are also anticipated at the subject site. It is further anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was carried out on March 16, 17 and 18, 2021. At that time, a total of seven (7) boreholes were advanced to a maximum depth of 10.1 m. A previous investigation was completed by Paterson on December 21, 2007 which consisted of two (2) boreholes advanced to a maximum depth of 3.1 m within the subject site. The borehole locations were determined by Paterson personnel to provide general coverage of the subject site taking into consideration site features and underground services. The locations of the boreholes are shown on Drawing PG5715-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes either directly from the auger flights or using a 50 mm diameter split-spoon sampler. Rock cores were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed securely in cardboard core boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 1 and BH 3. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Monitoring wells were installed in BH 4, BH 6 and BH 7 and piezometers were installed in all other boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ 3.0 m of slotted 51 mm diameter PVC screen at base the base of the boreholes.
- ☐ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No.3 silica sand backfill within annular space around screen.
- ☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

Sample Storage

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

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground features and services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The ground surface elevation at each borehole location was referenced to a geodetic datum. The borehole locations and ground surface elevation at each borehole location are presented on Drawing PG5715-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples and rock cores recovered from the subject site 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 potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.6.

4.0 Observations

4.1 Surface Conditions

Subject Site

The subject site is currently occupied by an equipment rental business and consists of an associated one-storey warehouse building, asphalt paved and gravel surfaced access lanes and parking, and grass covered areas. The site is bordered to the north by a rail corridor and further by agricultural land, to the east by a commercial building campus, to the south by a residential trailer park, and to the west by Stillwater Creek and further by a residential trailer park.

The ground surface across the site gradually slopes downward from south to north between approximate geodetic elevations of 89.0 to 87.5 m.

Stillwater Creek

Generally, Stillwater Creek runs approximately north-south along western portions of the subject site. The slope bordering Stillwater Creek was reviewed in the field by Paterson personnel as part of our slope stability assessment. Detailed observations at the time of our field reconnaissance are presented in Section 6.7 - Slope Stability Assessment.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of a 0.4 to 1.8 m thick layer of fill and/or topsoil. The fill was generally observed to consist of brown silty sand to silty clay with crushed stone and some organics.

A deposit of very stiff to stiff brown silty clay was encountered underlying the above-noted fill and topsoil layer extending to depths of approximately 1.8 to 6.9 m. The brown silty clay was further underlain by a layer of grey silty clay in BH 1, BH 2, BH 3 and BH 7 extending to depths of up to 9.8 m.

A 0.6 to 1.3 m thick glacial till deposit was encountered underlying the deposit of silty clay in BH 1, BH 2 and BH 4 and below the fill layer encountered in BH 6. The glacial till generally consisted of silty clay to silty sand with gravel, cobbles, and boulders.

Practical refusal to augering or DCPT was encountered in all boreholes with the exception of BH 7 at depths of 1.0 to 13.0 m.

In BH 8 and BH 9 from the 2007 field investigation, a 0.6 to 3.0 m thick layer of glacial till was encountered. At that time, practical refusal to augering was encountered at depths of 0.7 to 3.1 m.

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.

Bedrock

A good to excellent quality sandstone bedrock was encountered in BH 4 and BH 6 underlying the glacial till deposit at approximate depths of 1.0 to 1.9 m.

Based on available geological mapping, the majority of the subject site is located in an area where the bedrock consists of sandstone of the Nepean formation and the north portion of the site consists of dolomite of the Oxford formation, with a drift thickness of 2 to 10 m.

4.3 Groundwater

Groundwater levels were recorded in the monitoring wells and piezometers installed at the borehole locations on March 24, 2021. The groundwater level readings noted at that time are presented in Table 1. It should be noted that the groundwater level readings can be influenced by surface water perching within a backfilled borehole column, which can lead to higher than normal groundwater level readings. The long-term groundwater level can also be estimated by field observations of the recovered soil samples, such as moisture levels, undrained shear strength and colouring of the soil samples. Based on these observations and the color of the recovered soil samples, the long-term groundwater table can be anticipated at an elevation of 81.5 to 82.5 m throughout the majority of the subject site. The groundwater level can be considered to be below the bedrock surface throughout the south-east portion of the subject site. However, it should be noted that groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	87.47	0.31	87.16	March 24, 2021
BH 2	87.52	0.21	87.31	March 24, 2021
BH 3	88.69	0.21	88.48	March 24, 2021
BH 4	88.85	1.37	87.48	March 24, 2021
BH 5	89.12	NA	NA	March 24, 2021
BH 6	89.04	1.28	87.76	March 24, 2021
BH 7	88.82	1.93	86.89	March 24, 2021

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed development. Detailed plans for founding depths and underground levels were not available at the time of preparation of this report. Since design details of the proposed mixed-use buildings are not known at this time, geotechnical design information provided in this report may only be considered preliminary. Once design details have been developed for the subject site, development-specific recommendations may be provided at that time. Preliminary recommendations have been provided herein for future consideration. Further, due to the size of the subject site and the nature of the proposed buildings, a supplemental geotechnical field investigation will be required to provide specific design details.

For preliminary design purposes, it is expected that the proposed mid-rise buildings may be founded on conventional shallow spread footings placed on an undisturbed stiff silty clay or compact glacial till bearing surface, or a surface sounded bedrock bearing surface. The proposed high-rise buildings may be founded on conventional shallow spread footings placed on a surface sounded bedrock bearing surface.

However, for cases where loads exerted by proposed mid-rise buildings founded on a silty clay or glacial till bearing surface exceed the bearing resistance values provided herein, or where proposed high rise buildings are expected to be founded within the overburden soils, it is recommended that the proposed buildings be supported on end-bearing piles extending to the bedrock surface or a raft foundation.

Depending on founding depths for the buildings, bedrock removal may be required to complete underground levels. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

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

5.2 Site Grading and Preparation

Stripping Depth

Asphalt, topsoil, and any deleterious fill, such as those containing organic materials, should be removed from within the perimeter of the proposed buildings and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the perimeter of the proposed buildings. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the proposed building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath exterior 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 their respective 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.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock where necessary. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the effects on the existing services, buildings and other structures should be addressed. A pre-blast or construction survey located in the proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. Where bedrock is of lower quality, the excavation face should be free of any loose rock. An area specific review should be completed by the geotechnical consultant at the time of construction to determine if rock bolting or other remedial measures are required to provide a safe excavation face for areas where low quality bedrock is encountered.

Vibration Considerations

Construction operations could cause vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipments could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a temporary shoring system with soldier piles or sheet piling would require these pieces of equipment. Vibrations, caused by blasting or construction operations, could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit: the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

5.3 Preliminary Foundation Design

Bearing Resistance Values

Spread Footing Foundations - Commercial and Low to Mid-Rise Buildings

Foundations for the proposed low to mid-rise buildings, portions of underground parking levels (if considered) extending beyond the overlaying high-rise buildings and other light-loaded ancillary structures may consist of conventional spread footing foundations.

For preliminary design purposes, strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, very stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit state (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit state (ULS) of **225 kPa**.

Conventional spread footings placed on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit state (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**.

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 dry conditions, prior to the placement of concrete for footings.

Footings placed on a clean, surface sounded sandstone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

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 and/or glacial till above the groundwater table 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 of the same or higher capacity as the bearing medium soil.

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 sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

Strip footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Raft and Deep Foundations - Mid to High-Rise Buildings

Raft Foundation

Should the proposed bearing resistance values for conventional footings be deemed insufficient for support of the proposed mid to high-rise buildings, consideration may be given to foundation support by raft slab foundation structure. However, the geotechnical design of a raft slab is dependant on the number of below grade levels that are to be provided for the proposed buildings and the anticipated founding medium. Therefore, two scenarios have been considered for the purposes of this report (one and two levels of underground parking). Based on this review, a contact pressure of 150 kPa (SLS) for a one basement level scenario with a subgrade modulus of 6.0 MPa/m. A contact pressure of 190 kPa (SLS for a two basement level scenario with a subgrade modulus of 7.0 MPa/m.

Further, discussions and recommendations regarding the design of raft foundations can be provided in a supplemental geotechnical report for the subject site, as based on the results of a supplemental investigation and further review of detailed grading and site plans for the subject site. As a preliminary recommendation, where a raft slab is utilized, it is recommended that a minimum 50 mm thick lean concrete mud slab be placed on an undisturbed silty clay and/or glacial till subgrade shortly after the excavation and preparation of the bearing medium. The main purpose of the raft slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft slab 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. The raft slab should incorporate a waterproofing membrane system along with the perimeter foundation walls if the basement slab is expected to be below the long term groundwater level.

Pile Foundation

If the raft slab bearing resistance values provided are insufficient for the proposed high rise buildings, a deep foundation system driven to refusal in the bedrock will be recommended for foundation support of the proposed high-rise buildings. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area.

It should also be noted that end-bearing piles are only considered suitable if sufficient space for embedment below the foundation is available for end-fixity and lateral load resistance. End-bearing caissons would instead be considered if sufficient embedment cannot be accomplished. Additional foundation alternatives may also be provided at that time as based on the results of a supplemental investigation. However, as previously noted detailed design information may be provided once additional details are known for the proposed development. Buildings founded on piles driven to refusal in the bedrock will have negligible post-construction settlement.

End-Bearing Piles

Applicable pile resistance values at ultimate limit states (ULS) are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored at ULS values. Note that these are all geotechnical axial resistance values. The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance	Geotechnical Uplift Resistance
		Factored at ULS (kN)	Factored at ULS (kN) (assumed 12 m pile)
245	9	1350	200
245	11	1425	200
245	13	1500	200

Caissons

End bearing cast-in-place caissons can be used where supplemental axial resistance is required for structural design for the proposed building. The caisson should be installed by driving a temporary steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to in fill the caissons. The caissons are to be structurally reinforced over their entire length.

Two conditions for drilled shafts are applicable for this site. The first alternative is a caisson installed on the sound bedrock, augering through the weathered bedrock (end bearing). The compressive resistance for such piles is directly related to the compressive strength of the bedrock. It is recommended that the entire capacity be derived from the end bearing capacity.

The second alternative is a concrete caisson socketed into bedrock. The axial capacity is increased by the shear capacity of the concrete/rock interface. Furthermore, the tensile resistance of the caisson is increased by the rock capacity. It should be noted that the rock socket should be reinforced.

Table 3 below presents the estimated capacity for different typical caisson sizes for a rock bearing caisson and rock socketed caisson extending 3 m into sound bedrock.

Table 3 - Caisson Pile Capacities

Caisson Diameter		Axial Capacity (kN)		Factored Capacity Tension at ULS (kN)	
inch	mm	End Bearing	Rock Socket	End Bearing	Rock Socket
36	900	10000	14500	920	2700
42	1000	15000	19000	1050	3450
48	1200	19000	24500	1200	4500
54	1375	24000	31000	1350	5300
60	1500	30000	38000	1500	6000

notes:
- 3 m rock socket in sound bedrock
- Reinforced caisson and rock socket when applicable
- 0.4 geotechnical factor applied to the shaft capacity

Permissible Grade Raise

A permissible grade raise restriction of **2 m** is recommended for the subject site. It should be noted that the permissible grade raise provided is subject to change based on the results of the supplemental geotechnical investigation. If greater permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements of the soils surrounding the buildings.

5.4 Preliminary Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for foundations founded upon a silty clay bearing medium and as **Class C** for foundations founded upon a glacial till or bedrock bearing medium for foundation considered at the subject site.

Higher site classes such as Class A or Class B may be provided for buildings founded upon or within 3 m of the bedrock surface. However, they would have to be confirmed by site specific shear wave velocity testing. Such testing may be considered once more detailed plans are available for the proposed development. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Slab on Grade and Basement Slab

With the removal of all topsoil and deleterious materials, within the footprint of the proposed buildings, the native soil or existing fill as approved by the geotechnical consultant will be considered to be an acceptable subgrade surface on which to commence backfilling for basement floor slab.

If a raft slab is utilized, a granular layer of OPSS Granular A will 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.

For the buildings founded on footings or piles, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear 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 at least 98% of its SPMDD.

For buildings of slab-on-grade construction, it is recommended that the upper 300 mm of sub-slab fill consists of OPSS Granular A crushed stone.

A sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains 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.

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.

5.6 Preliminary Pavement Structure

Although detailed design plans were not available at the time of preparation of this report, the following pavement structures may be considered for planning purposes of the proposed development.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 5 - Recommended Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** HL3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL8 or 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 to a competent layer 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, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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, noting that excessive compaction can result in subgrade softening.

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 and Waterproofing

Buildings proposed throughout the development of the subject site whose basement levels are founded below the long-term groundwater table should be provided a groundwater suppression system. The groundwater suppression system would consist of installing a waterproofing membrane over a drainage geocomposite installed on the exterior portion of the foundation wall. The waterproofing membrane is recommended to extend between the bottom of the foundation and up to a minimum of 1 m above the long-term groundwater level. A groundwater suppression system would also be recommended for structures located below the buildings foundations (ie.- elevator shafts, sump pits, etc...).

Due to the preliminary nature of the development, the requirement for groundwater suppression systems will be assessed once the number of proposed basement levels the future mid and high-rise buildings will be provided is known. Details pertaining to the groundwater suppression system may also be provided at that time.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Backfill material below sidewalk or asphalt paved subgrade areas or other settlement sensitive structures should consist of free draining, non-frost susceptible material placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover is required unless placed in conjunction with adequate foundation insulation.

Exterior unheated foundations, 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.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structures are backfilled.

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 the groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

Temporary Shoring

The design and approval of the temporary 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 system 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 could consist of a soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, neighboring buildings, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

The earth pressures acting on the temporary shoring system may be calculated with the following parameters.

Table 6 - 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
Dry Unit Weight (γ), kN/m^3	20
Effective 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/bedrock 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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The bedding should be increased to a minimum thickness of 300 mm where bedrock is encountered at the subgrade level. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the 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 crushed stone, 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 99% of the SPMDD.

Generally, the brown silty clay should be possible to place above the cover material if the excavation and backfilling operations are completed in dry weather conditions. Wet silty clay materials will be difficult for placement, as the high water content are impractical for the desired compaction without an extensive drying period. All stones greater than 300 mm in their largest dimension should be removed prior to reuse of site-generated backfill materials.

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 SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

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.

Infiltration levels are anticipated to be low through the excavation face, and the groundwater infiltration is anticipated to be controllable with open sumps and pumps. 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 allocated for completion of the PTTW application package and issuance of the permit by the MECP.

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

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or underfloor drainage system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 50,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed.

Impacts on Neighboring Structures

Detailed plans of the development were not available at the time of preparation of this report, details regarding impacts on neighboring structures can be provided based on specific design details for the proposed development.

Generally, the design of the foundation with a groundwater infiltration control system in place will not impact neighboring structures based on the subsurface profiles.

6.6 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a non aggressive to slightly aggressive corrosive environment.

6.7 Slope Stability Assessment

A steep ravine is observed running in a north-south direction across the west portion of the site. A segment of Stillwater Creek runs within the valley corridor of the ravine slopes adjacent to the subject site. The slope condition was reviewed by Paterson field personnel as part of the geotechnical investigation. Four (4) slope cross-sections were studied as the worst case scenarios, where the watercourse has meandered in close proximity of the toe of the upper slope. A 10 to 12 m high stable slope inclined generally 2H:1V with limited areas shaped to a 1H:1V profile. The watercourse was confined within the approximately 2 to 4 m wide watercourse banks and the water flow rate was noted to be low.

Generally, the overall slope face was observed to be grass covered with some mature trees, minor toe erosion was observed along the edges of the meanders at some locations. Significant in-filling was observed at the top of the slope and down the slope face. Photographs taken during our site visit to assess the slope condition can be found in Appendix 2.

Based on historical aerial images of the slope face obtained from GeoOttawa, the natural course of the creek has been altered due to fill placement within the subject site. When aerial images of the creek from 1958 and 2011, shown in Figures 2 to 4 in Appendix 2, are compared the natural course of the creek was observed to have shifted to the west and the meander shapes were altered. In-filling at the site has forced the water course to re-establish further west.

A slope stability analysis was carried out to determine the required geotechnical setback from the top of the bank based on a factor of safety of 1.5. Toe erosion and erosion access allowances were also considered in the determination of limits of hazard lands setback line and are discussed on the following pages. If limits of hazard lands need to be further reduced, erosional protection, such as rip rap or alternative means, would need to be provided and is subject to the approval of the conservation authority with jurisdiction of this watercourse.

Slope Stability Analysis

The analysis of the stability of the upper slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

Subsoil conditions at the cross-sections were inferred based on nearby boreholes. For a conservative review of the groundwater conditions, the silty clay deposit was noted to be fully saturated for our analysis and exiting at the toe of the slope. The results are shown in Figures 5, 7, 9 and 11 in Appendix 2. The results indicate a slope with a factor of safety of 1.53 at Section A and slopes with factors of safety less than 1.5 beyond the top of slope at Section B, C and D. Based on these results, a stable slope setback varying between 9 and 15 m from the top of the slope are required to achieve a factor of safety of 1.5 for the limit of the hazard lands in the area of Sections B, C and D.

Seismic Loading Analysis

An analysis considering seismic loading and the groundwater at ground surface was also completed. A horizontal acceleration of 0.16g was considered for all slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the analyses including seismic loading are shown in Figures 6, 8, 10 and 12 in Appendix 2. The results indicate a slope with a factor of safety of 1.36 at Section A and 1.30 at Section D and slopes with factors of safety less than 1.1 beyond the top of slope at Section B and C. Based on these results, a stable slope setback varying between 1 and 5 m from the top of the slope is required to achieve a factor of safety of 1.1 for the limit of the hazard lands. However, it should be noted that the stable slope setback associated with our seismic loading analysis is superceded by the required stable slope setback required for static conditions.

Erosion and Access Allowances

Based on the soil profiles encountered at the borehole locations, silty sand fill, firm to very stiff silty clay and/or glacial till are anticipated to be subject to erosion activity by the watercourse within the valley corridor. Based on the anticipated soils, a toe erosion allowance of 5 m should be applied from the watercourse edge and an access allowance of 6 m is required from the top of slope or geotechnical setback (where applicable). In areas where the watercourse edge has meandered to within 5 m of the toe of the existing slope, the toe erosion and access allowances should be applied in addition to geotechnical setback limit from the top of slope.

The existing vegetation on the slope faces should not be removed as it contributes to the stability of the slope and reduces erosion.

7.0 Recommendations

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

☐ Supplemental investigation to be provided once final development design has been established.

☐ 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.

☐ Periodic observation of the condition of the vertical bedrock face during excavation.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Our recommendations should be reviewed when the drawings and specifications are complete.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request immediate notification in order to reassess our recommendations.

The recommendations provided should only be used by the design professionals associated with this project. The recommendations are not intended for contractors bidding on or constructing the project. The later should evaluate the factual information provided in the report. The contractor should also determine the suitability and completeness for the intended construction schedule and methods. Additional testing may be required for the contractors' purpose.

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 The Properties Group 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.

Nicole Patey, B.Eng.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
May 21, 2021
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Report Distribution:

- ☐ The Properties Group (e-mail copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Mixed-Use Development - 1987 Robertson Rd.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME 55 Power Auger
DATE March 18, 2021
FILE NO. PG5715
HOLE NO. BH 1-21

SOIL DESCRIPTION

GROUND SURFACE

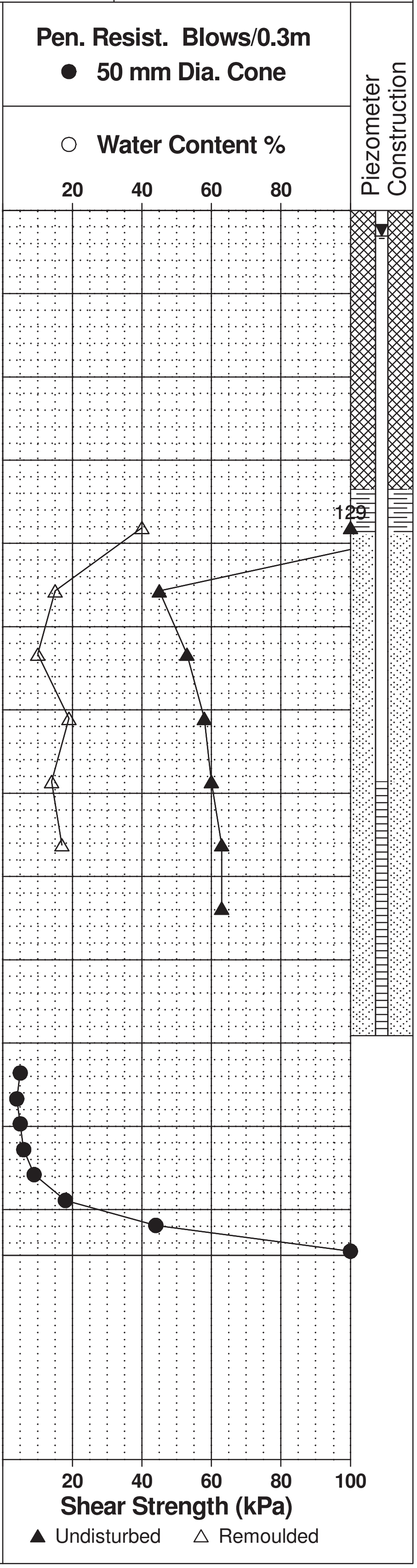
SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
TOPSOIL	0.05	AU			
FILL: Brown silty clay with sand, gravel, some topsoil	0.61	AU	1		
		SS	2	33	5
Very stiff to stiff, brown **SILTY CLAY** some sand		SS	3	100	3
- no sand by 1.9m depth		SS	4	100	5
	4.57	SS	5	100	4
Firm to stiff, grey **SILTY CLAY**	8.99				
GLACIAL TILL: Loose grey silty clay with sand, gravel, cobbles and boulders	10.06	SS	6	75	3
Dynamic Cone Penetration Test commenced at 10.06 m depth.	12.55				

End of Borehole

Practical DCPT refusal at 12.55 m depth.

(GWL @ 0.31m - March 24, 2021)

DEPTH (m)	ELEV. (m)
0	87.47
1	86.47
2	85.47
3	84.47
4	83.47
5	82.47
6	81.47
7	80.47
8	79.47
9	78.47
10	77.47
11	76.47
12	75.47



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Mixed-Use Development - 1987 Robertson Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 17, 2021

FILE NO. PG5715

HOLE NO. BH 2-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel, trace clay and cobbles	0.10
TOPSOIL	0.46
Very stiff to stiff, brown **SILTY CLAY** trace sand	4.57
Firm, grey **SILTY CLAY**	6.86
GLACIAL TILL: Dense to very dense grey silty clay with sand, gravel, cobbles and boulders	8.00

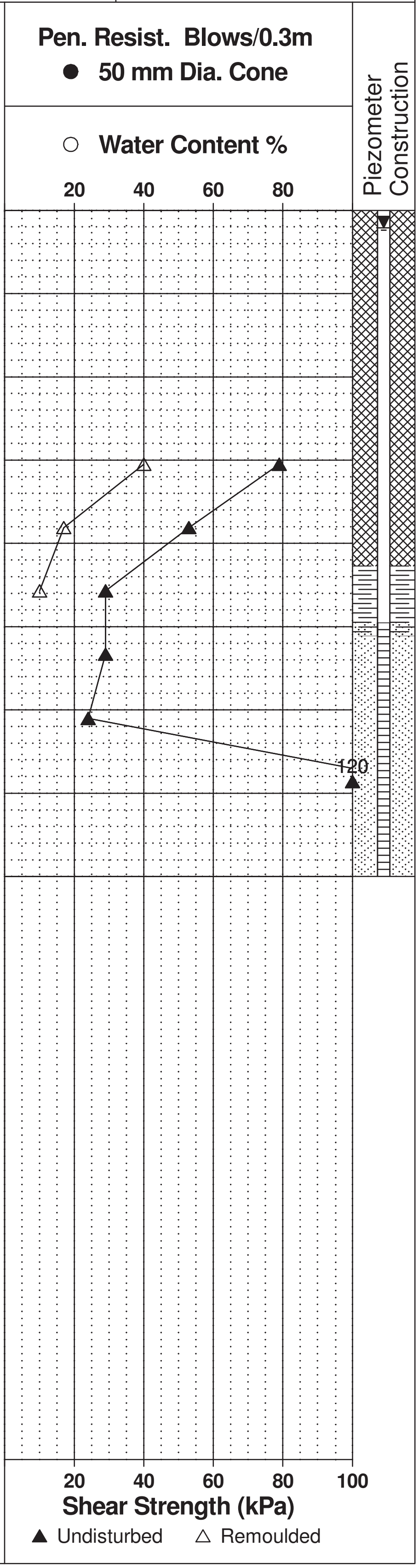
End of Borehole

Practical refusal to augering at 8.00m depth

(GWL @ 0.21m - March 24, 2021)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
AU	2		
SS	3	75	7
SS	4	100	6
SS	5	100	5
SS	6	0	+50

DEPTH (m)	ELEV. (m)
0	87.52
1	86.52
2	85.52
3	84.52
4	83.52
5	82.52
6	81.52
7	80.52
8	79.52



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Mixed-Use Development - 1987 Robertson Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

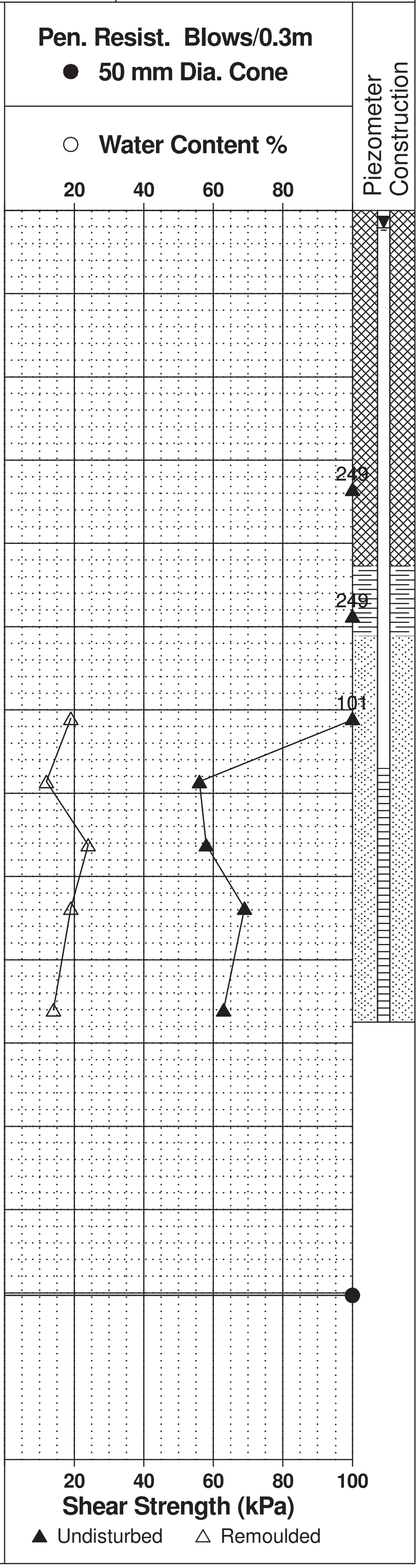
DATE March 17, 2021

FILE NO. PG5715

HOLE NO. BH 3-21

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with crushed stone, some organics, trace clay	AU	1		
	SS	2	25	11
1.83	SS	3	67	10
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	4	67	5
	SS	5	100	7
- trace sand and gravel by 3.5 m depth	SS	6	100	7
	SS	7	100	7
	SS	8	100	3
6.71				
Stiff, grey **SILTY CLAY**				
9.75	SS	9	100	W
Dynamic Cone Penetration Test commenced at 9.75 m depth. Cone pushed to 13.03 m depth.				
13.03				
End of Borehole				
Practical DCPT refusal at 13.03m depth				
(GWL @ 0.21m - March 24, 2021)				

DEPTH (m)	ELEV. (m)
0	88.69
1	87.69
2	86.69
3	85.69
4	84.69
5	83.69
6	82.69
7	81.69
8	80.69
9	79.69
10	78.69
11	77.69
12	76.69
13	75.69



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Mixed-Use Development - 1987 Robertson Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 16, 2021

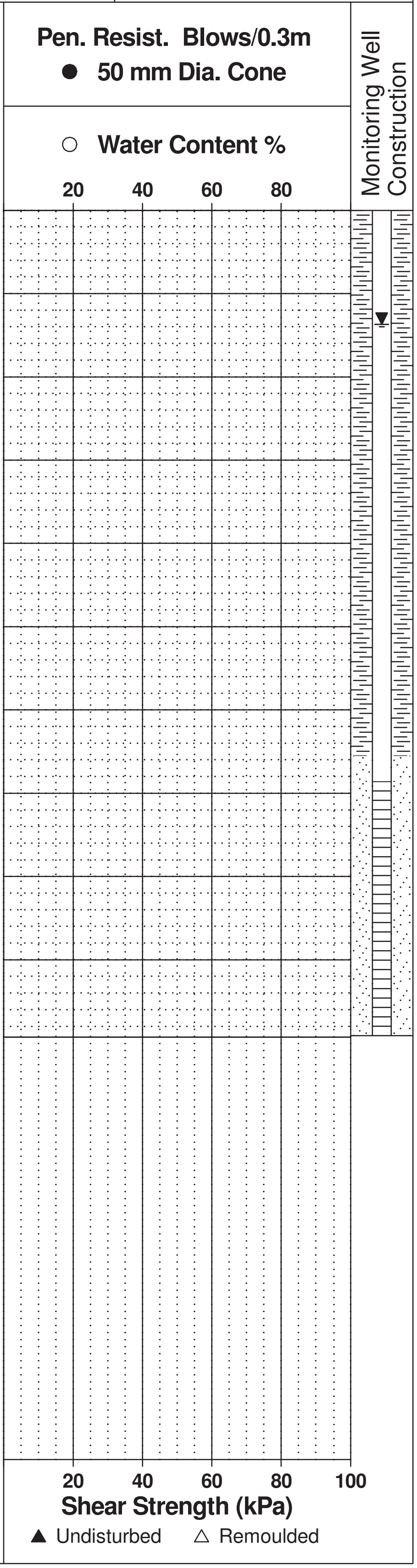
FILE NO. PG5715

HOLE NO. BH 4-21

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
FILL: Crushed stone with brown silty sand	0.15
FILL: Brown silty clay some sand trace gravel	0.61
GLACIAL TILL: Dense to very dense grey silty clay with sand, gravel, cobbles and boulders	1.93
BEDROCK Good to excellent quality, grey quartz sandstone	9.93
End of Borehole	
(GWL @ 1.37m - March 24, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
AU	2		
SS	3	42	7
SS	4	100	+50
RC	1	100	100
RC	2	100	100
RC	3	100	88
RC	4	100	71
RC	5	100	90
RC	6	100	95

DEPTH (m)	ELEV. (m)
0	88.85
1	87.85
2	86.85
3	85.85
4	84.85
5	83.85
6	82.85
7	81.85
8	80.85
9	79.85



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Mixed-Use Development - 1987 Robertson Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 17, 2021

FILE NO. PG5715

HOLE NO. BH 5-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.12
FILL: Brown silty sand with crushed stone — 0.30		AU	1				
TOPSOIL — 0.64		AU	2				
		AU	3				
Very stiff brown **SILTY CLAY**		SS	4	50	13	1	88.12
— 1.80		SS	5	100	+50		
End of Borehole							

Practical refusal to augering at 1.80 m depth

(Piezometer blocked - March 24, 2021)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Mixed-Use Development - 1987 Robertson Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

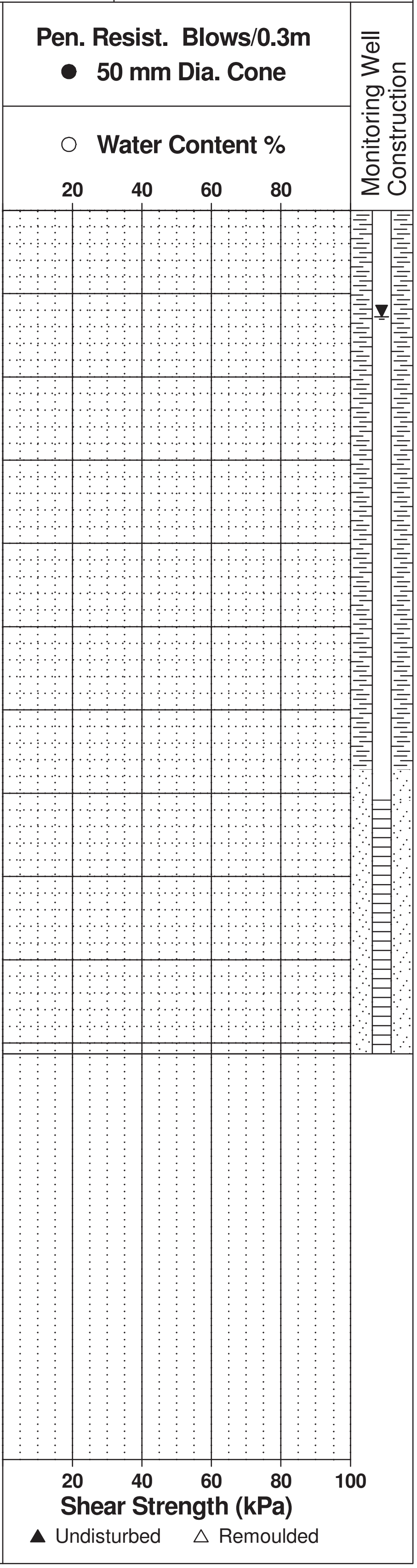
DATE March 18, 2021

FILE NO. PG5715

HOLE NO. BH 6-21

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with crushed stone, some clay, trace organics	0.43	AU			
		AU	1		
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders	0.99	SS	2	50	+50
BEDROCK: Good to excellent quality, grey quartz sandstone		RC	1	100	80
		RC	2	90	65
		RC	3	98	87
		RC	4	100	80
		RC	5	100	78
		RC	6	100	92
	10.13				
End of Borehole (GWL @ 1.28m - March 24, 2021)					

DEPTH (m)	ELEV. (m)
0	89.04
1	88.04
2	87.04
3	86.04
4	85.04
5	84.04
6	83.04
7	82.04
8	81.04
9	80.04
10	79.04



SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Mixed-Use Development - 1987 Robertson Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

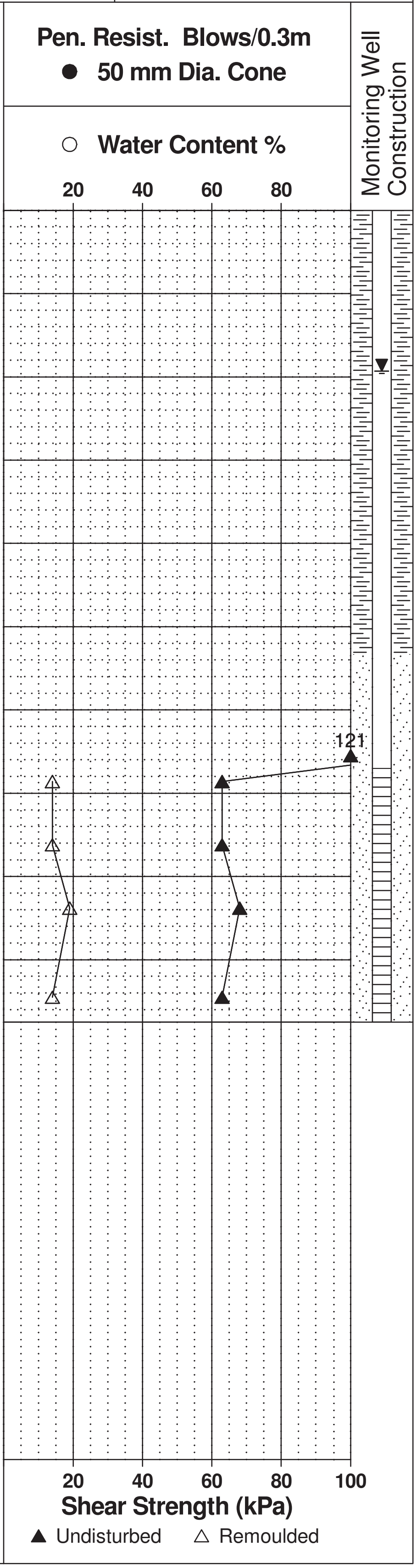
DATE March 16, 2021

FILE NO. PG5715

HOLE NO. BH 7-21

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with crushed stone, some clay, trace organics	AU	1		
	SS	2	33	14
1.83	SS	3	50	8
Very stiff to stiff, brown **SILTY CLAY,** some silt seams	SS	4	67	12
	SS	5	100	9
	SS	6	100	9
	SS	7	100	7
	SS	8	58	7
	SS	9	100	3
6.86				
Stiff, grey **SILTY CLAY**				
9.75	SS	10	100	W
End of Borehole				
(GWL @ 1.93m - March 24, 2021)				

DEPTH (m)	ELEV. (m)
0	88.82
1	87.82
2	86.82
3	85.82
4	84.82
5	83.82
6	82.82
7	81.82
8	80.82
9	79.82



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

SOIL PROFILE & TEST DATA

Preliminary Geotechnical Investigation
Proposed Development, Bellwood Trailer Park
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 21 DEC 07

FILE NO. PG1593

HOLE NO. BH 8

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	86.80		
TOPSOIL 0.13		AU	1						
GLACIAL TILL: Brown silty sand with gravel and cobbles 0.69									
End of Borehole Practical refusal to augering @ 0.69m depth (BH dry upon completion)									

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 & TEST DATA

Preliminary Geotechnical Investigation
Proposed Development, Bellwood Trailer Park
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

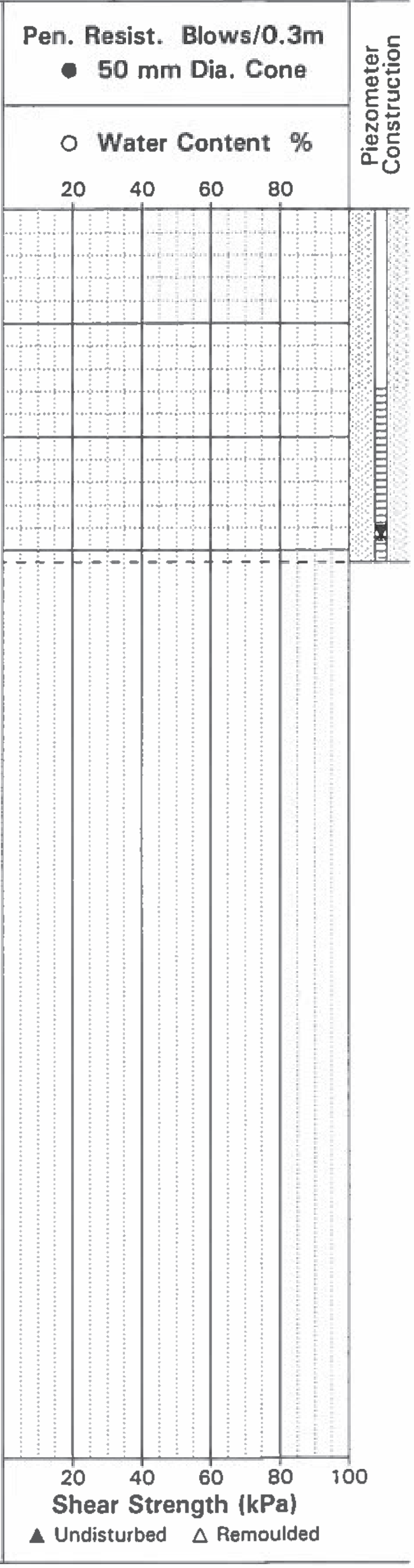
BORINGS BY CME 55 Power Auger

DATE 21 DEC 07

FILE NO. PG1593

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.83
TOPSOIL 0.10							
GLACIAL TILL: Brown silty clay with sand, gravel and cobbles		AU	1				
						1	86.83
		SS	2	100	15		
						2	85.83
3.10		SS	3	0	50+	3	84.83
End of Borehole Practical refusal to augering @ 3.10m depth (GWL @ 2.89m-Jan. 2/08)							



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

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

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

CONSOLIDATION TEST

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

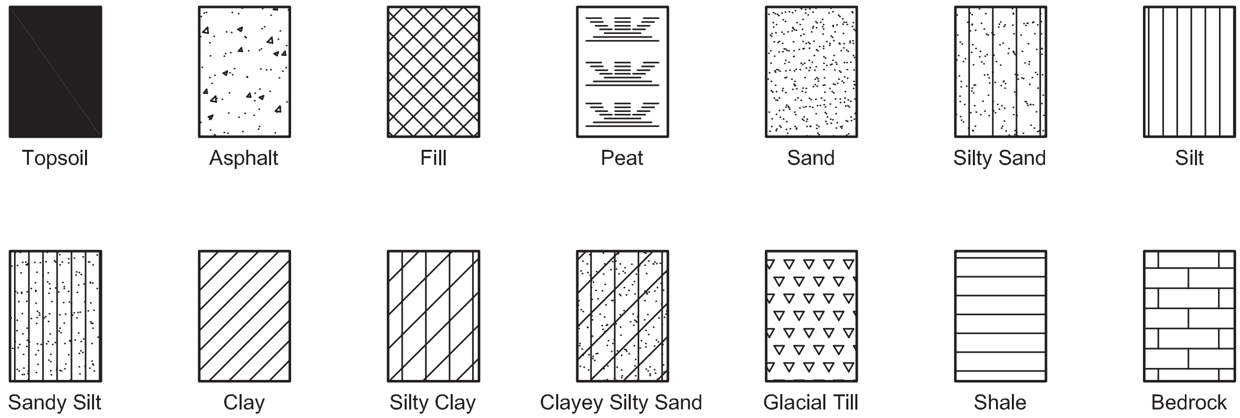
OC Ratio Overconsolidaton ratio $= p'_c / p'_o$
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

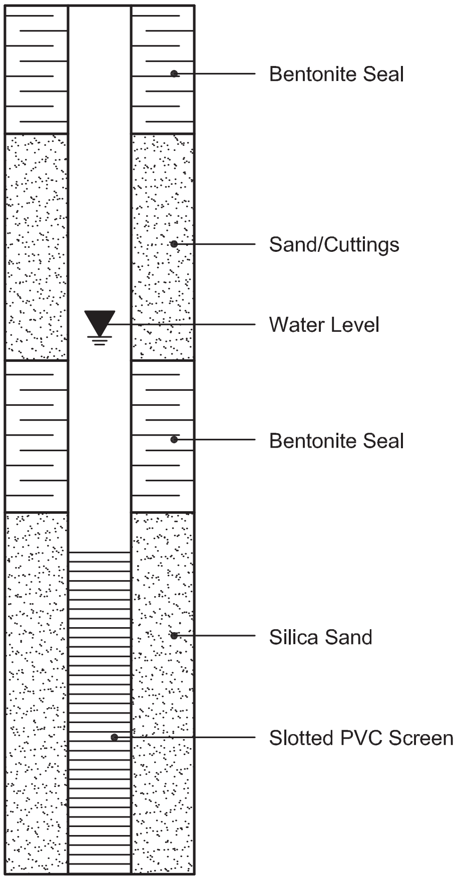
SYMBOLS AND TERMS (continued)

STRATA PLOT

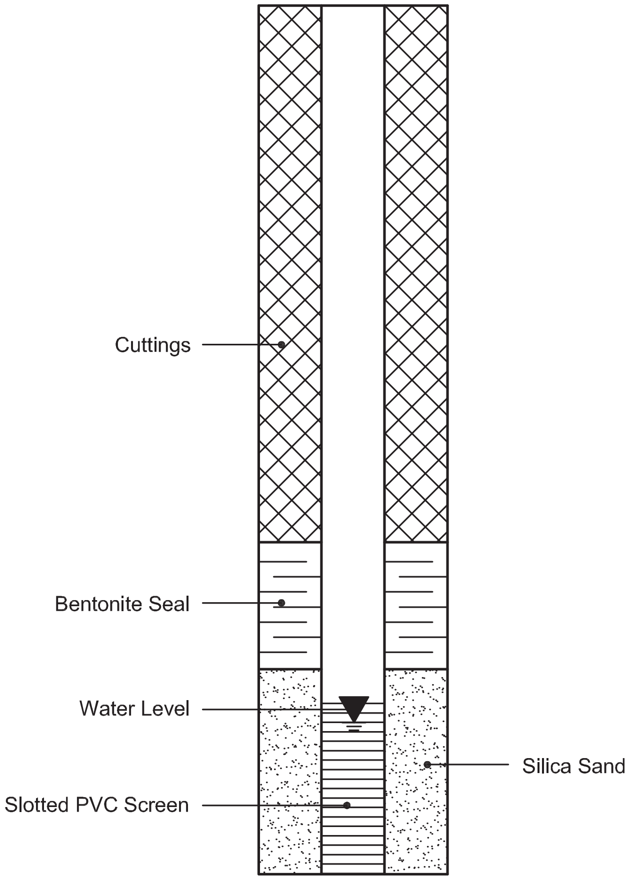


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29744

Report Date: 24-Mar-2021

Order Date: 18-Mar-2021

Project Description: PG5715

	Client ID:	BH3-21 SS4	-	-	-
	Sample Date:	17-Mar-21 09:00	-	-	-
	Sample ID:	2112531-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	72.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.42	-	-	-
Resistivity	0.10 Ohm.m	43.7	-	-	-
Anions					
Chloride	5 ug/g dry	61	-	-	-
Sulphate	5 ug/g dry	22	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 4 - AERIAL IMAGES

PHOTOS 1 TO 4 - PHOTOGRAPHS FROM SITE VISIT

FIGURES 5 TO 12 - SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG5715-1 - TEST HOLE LOCATION PLAN

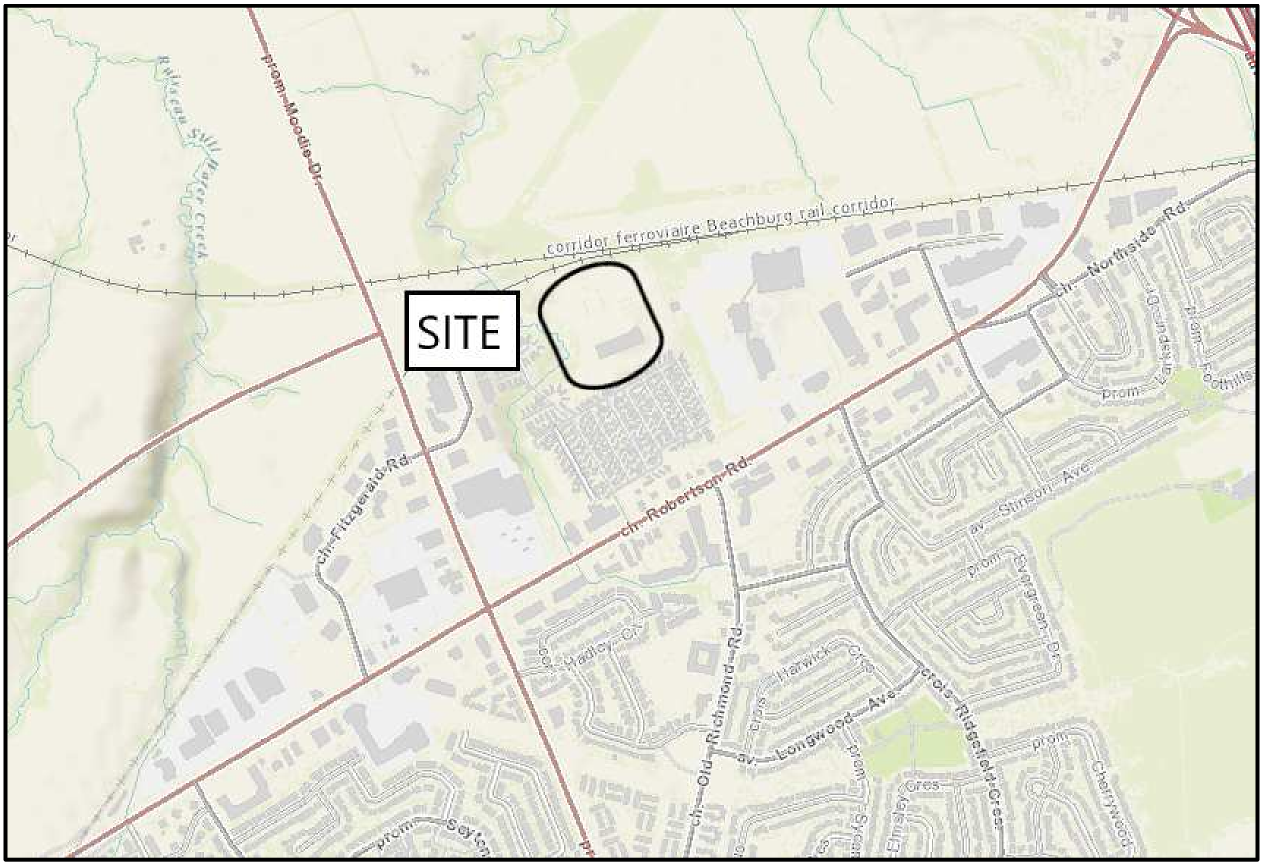


FIGURE 1

KEY PLAN

FIGURE 2

1958 AERIAL IMAGE

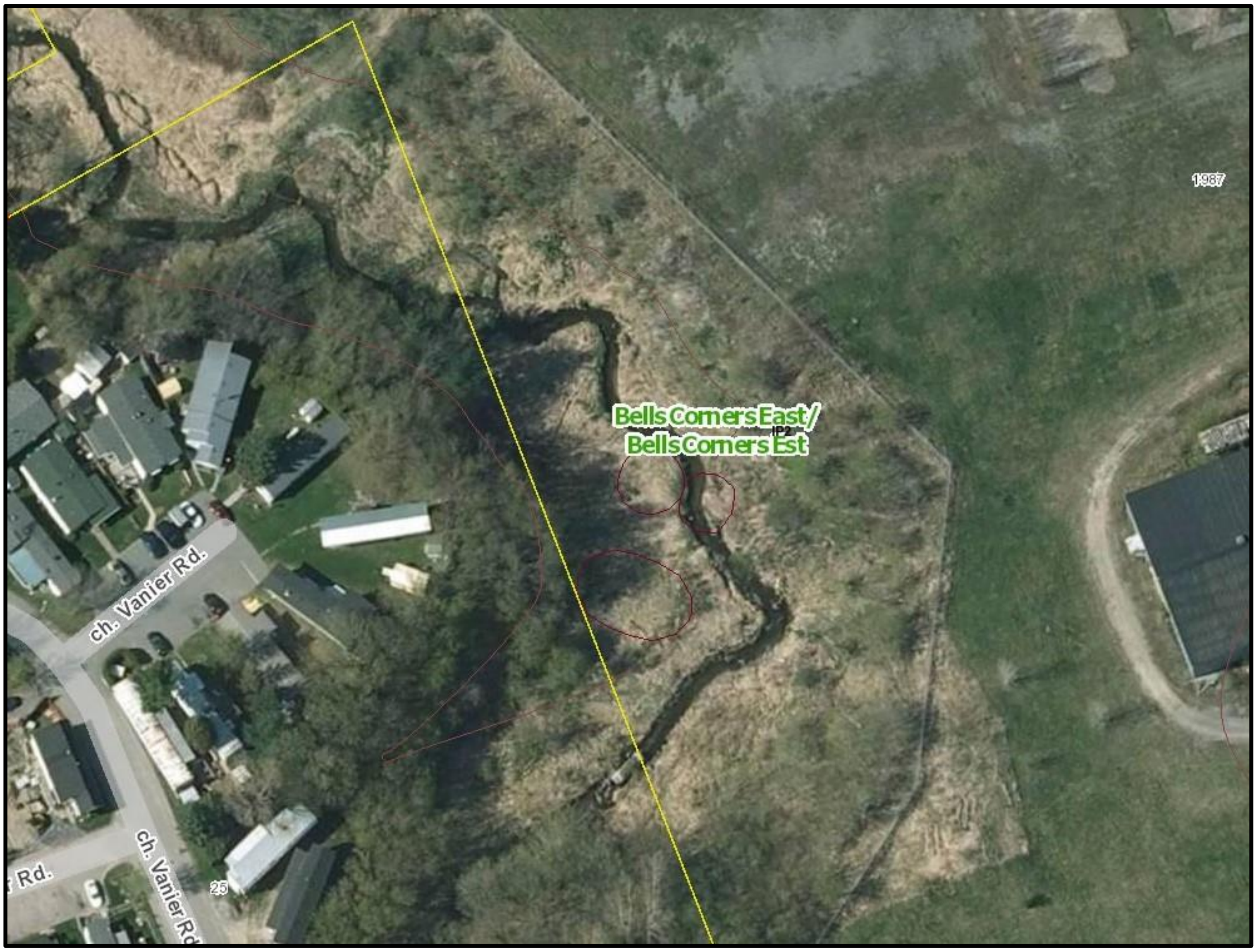


FIGURE 3

2011 AERIAL IMAGE

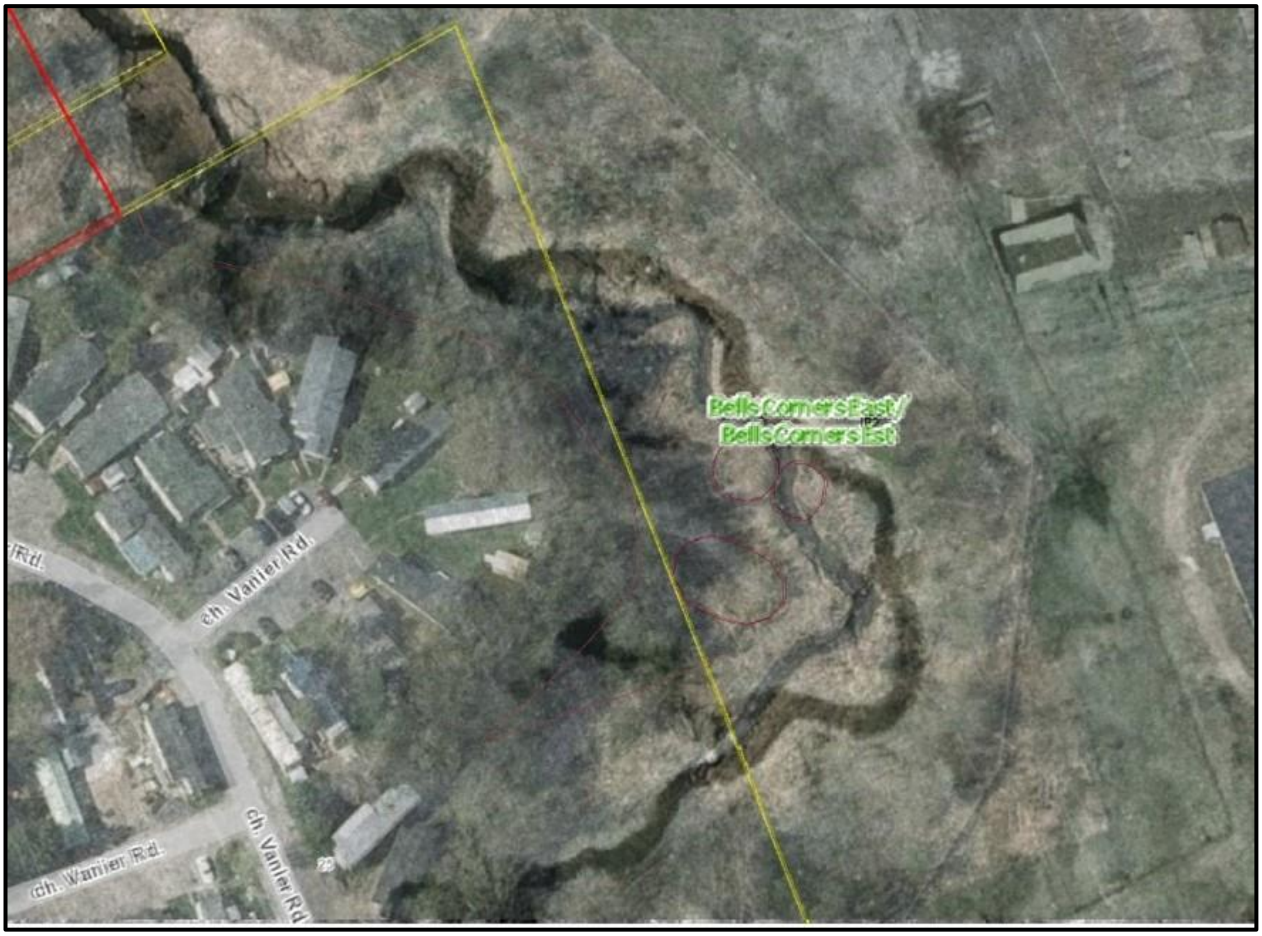


FIGURE 4

OVERLAY OF 1958 & 2011 AERIAL IMAGES

Photo 1: Photograph of Stillwater Creek and toe of slope taken at the west portion of the site towards the north illustrating grass covered side slopes, no toe erosion was observed.

Photo 2: Photograph of Stillwater Creek and toe of slope taken at the west portion of the site towards the north illustrating grass covered side slopes, minor toe erosion was observed.

Photo 3: Photograph from the creek looking east towards the top of slope illustrating fill on the slope.

Photo 4: Photograph from the top of slope looking west towards the creek illustrating fill on the slope.

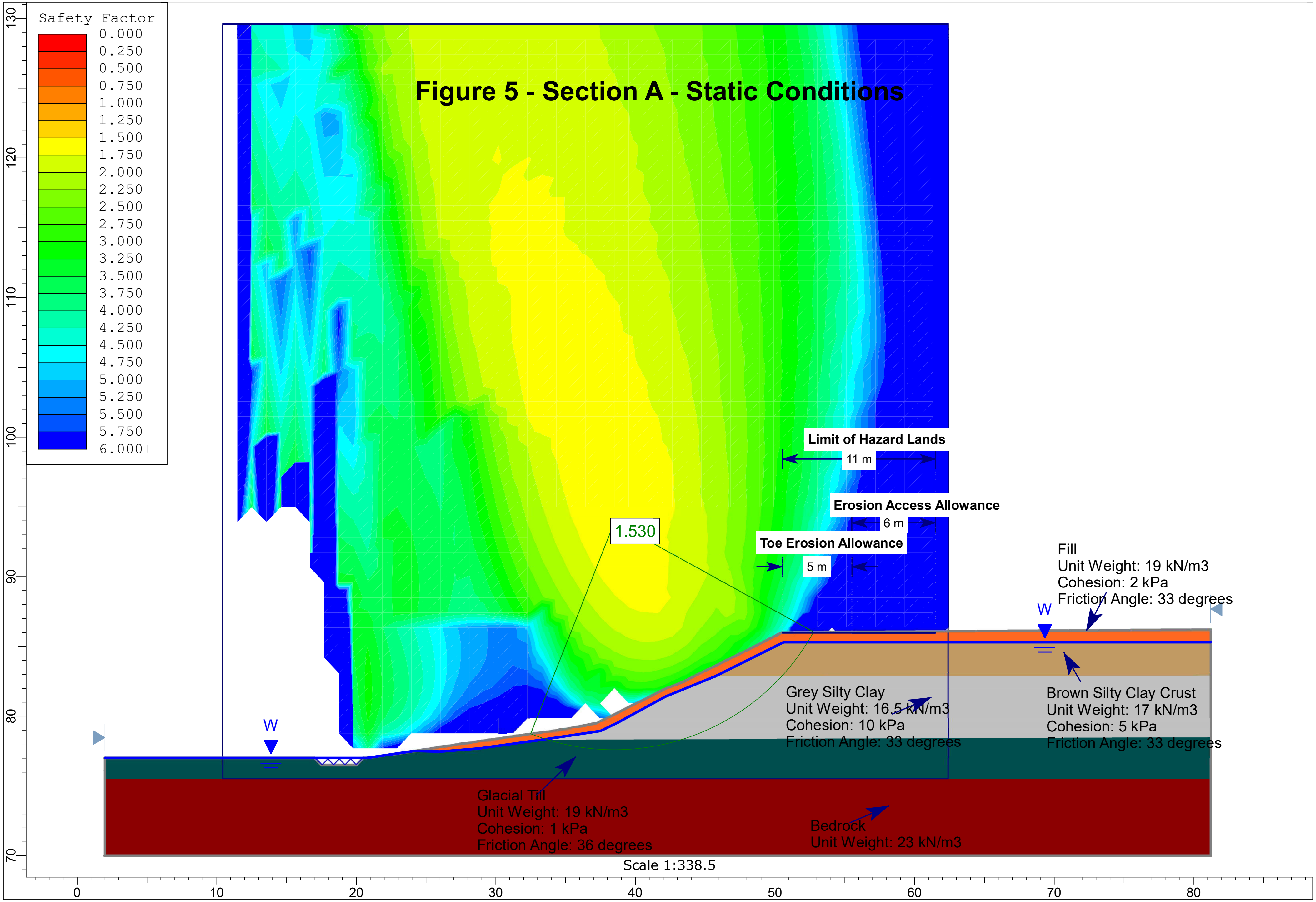
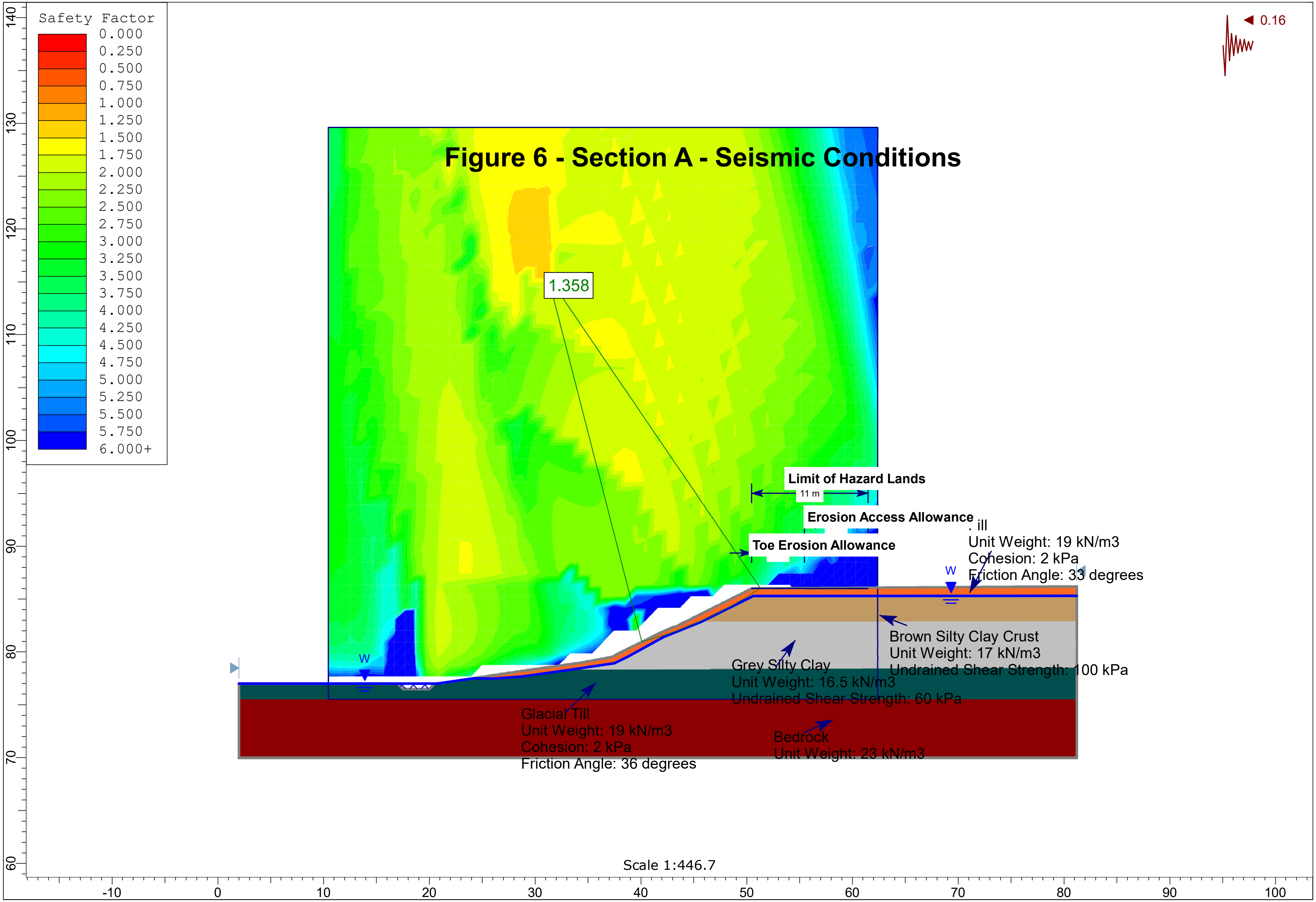


Figure 5 - Section A - Static Conditions

Figure 6 - Section A - Seismic Conditions



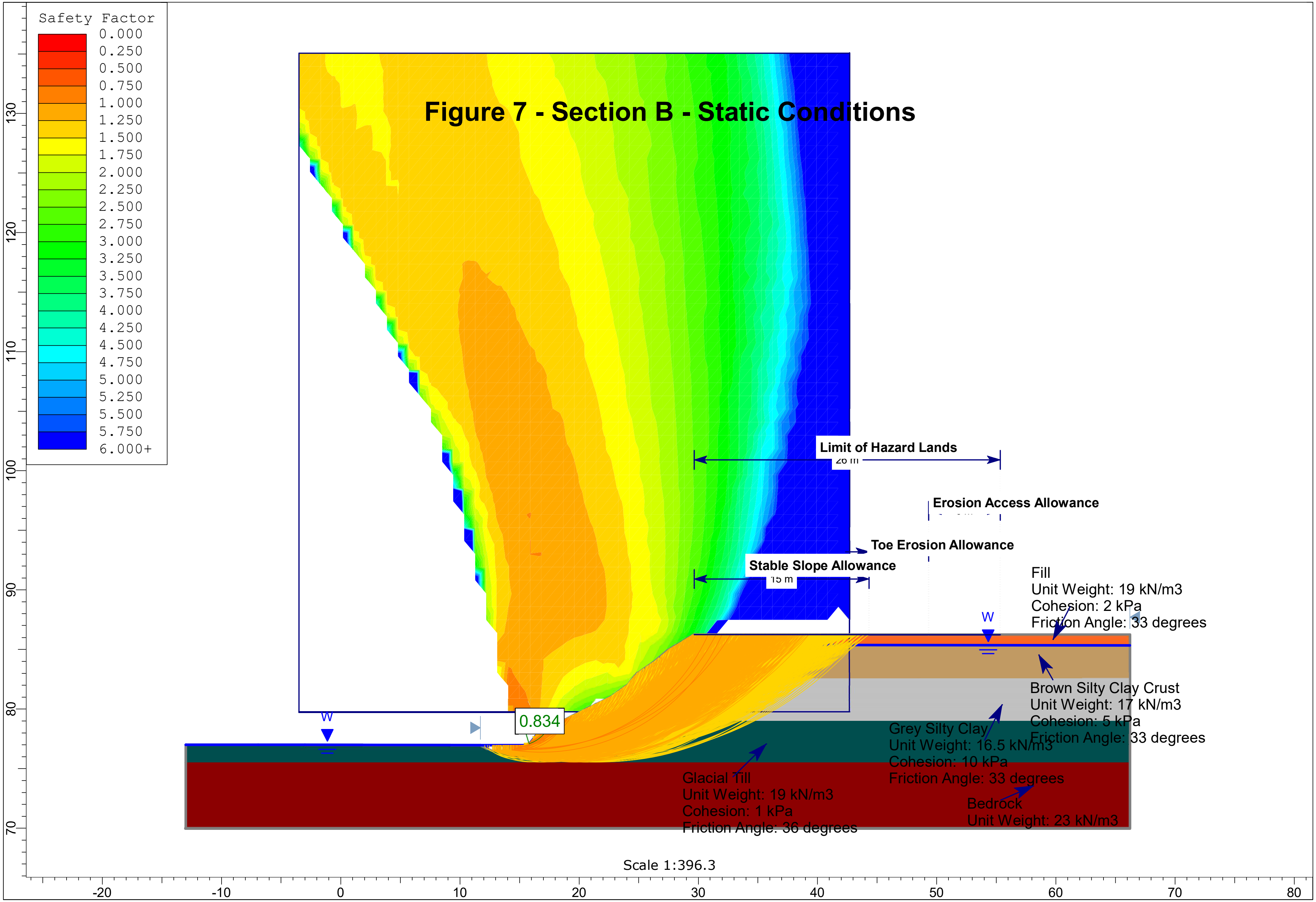


Figure 7 - Section B - Static Conditions

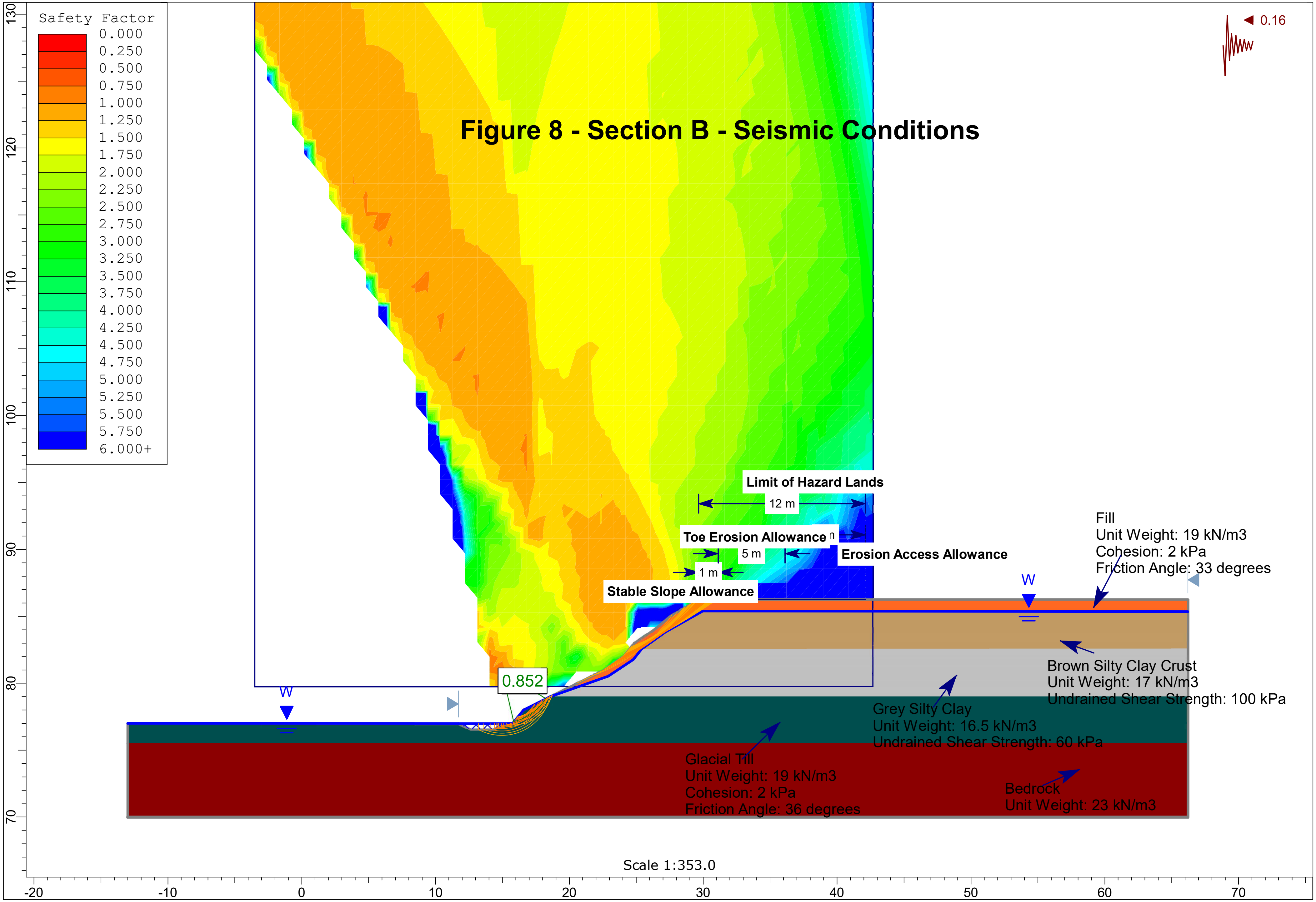


Figure 8 - Section B - Seismic Conditions

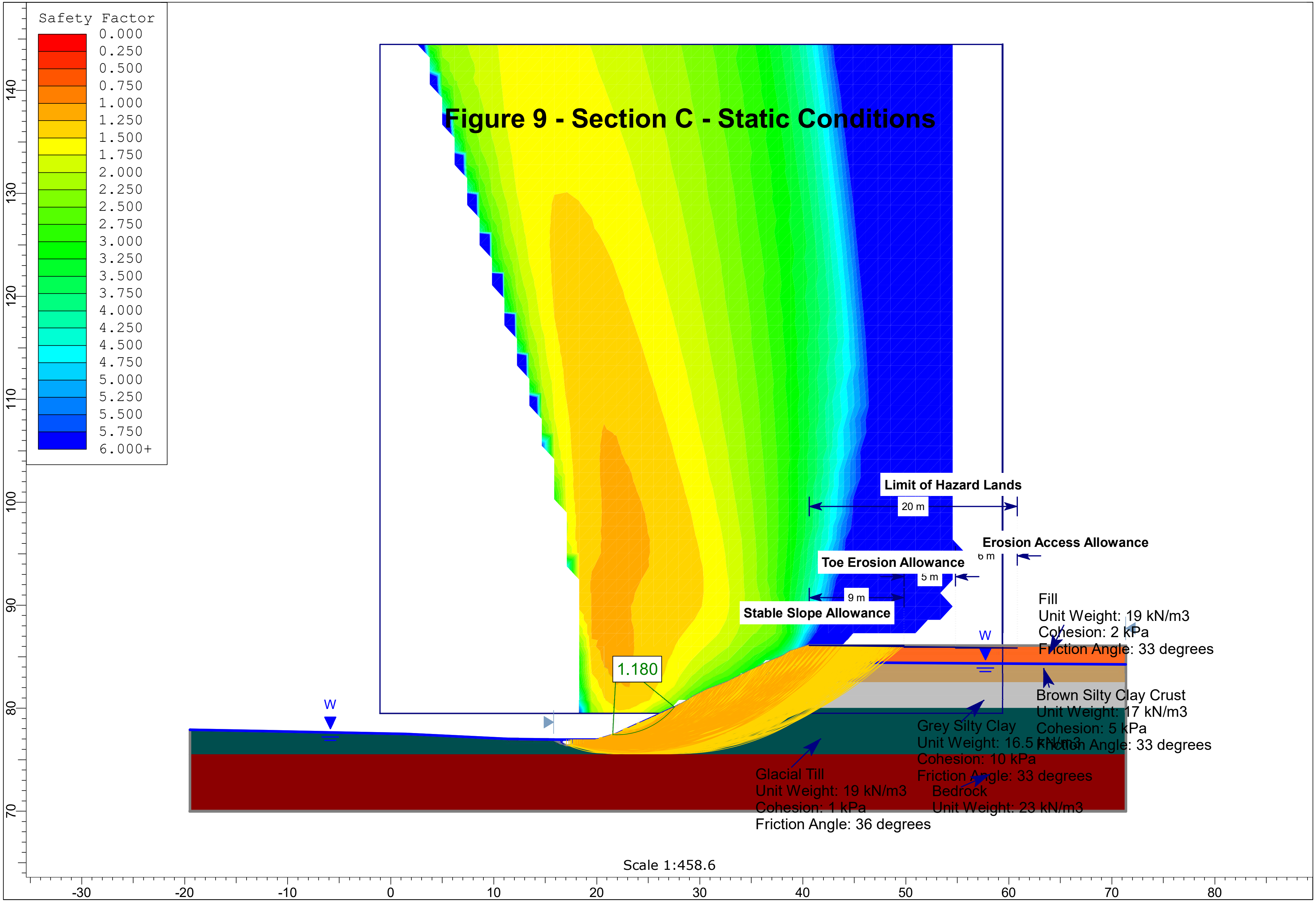


Figure 9 - Section C - Static Conditions

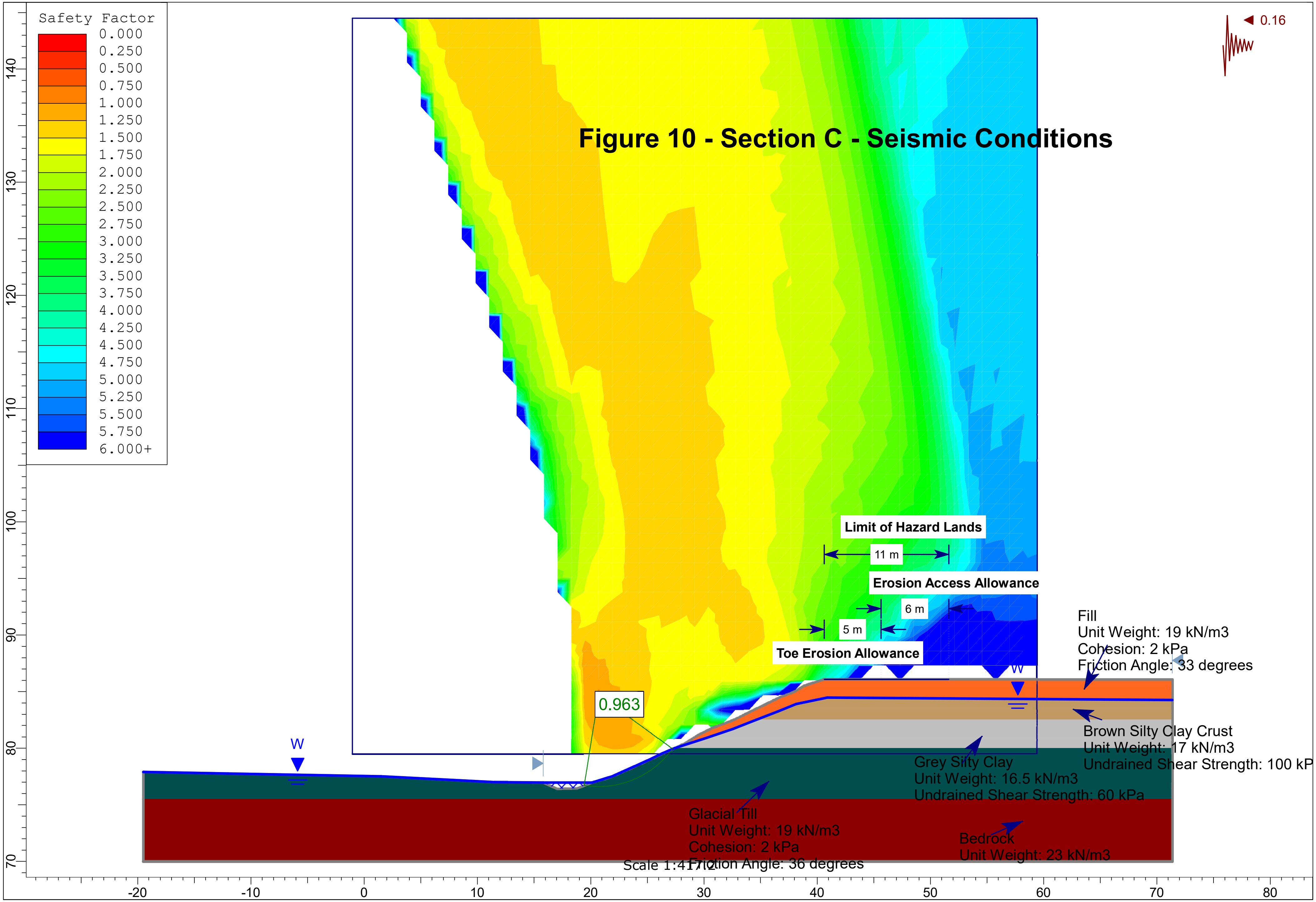


Figure 10 - Section C - Seismic Conditions

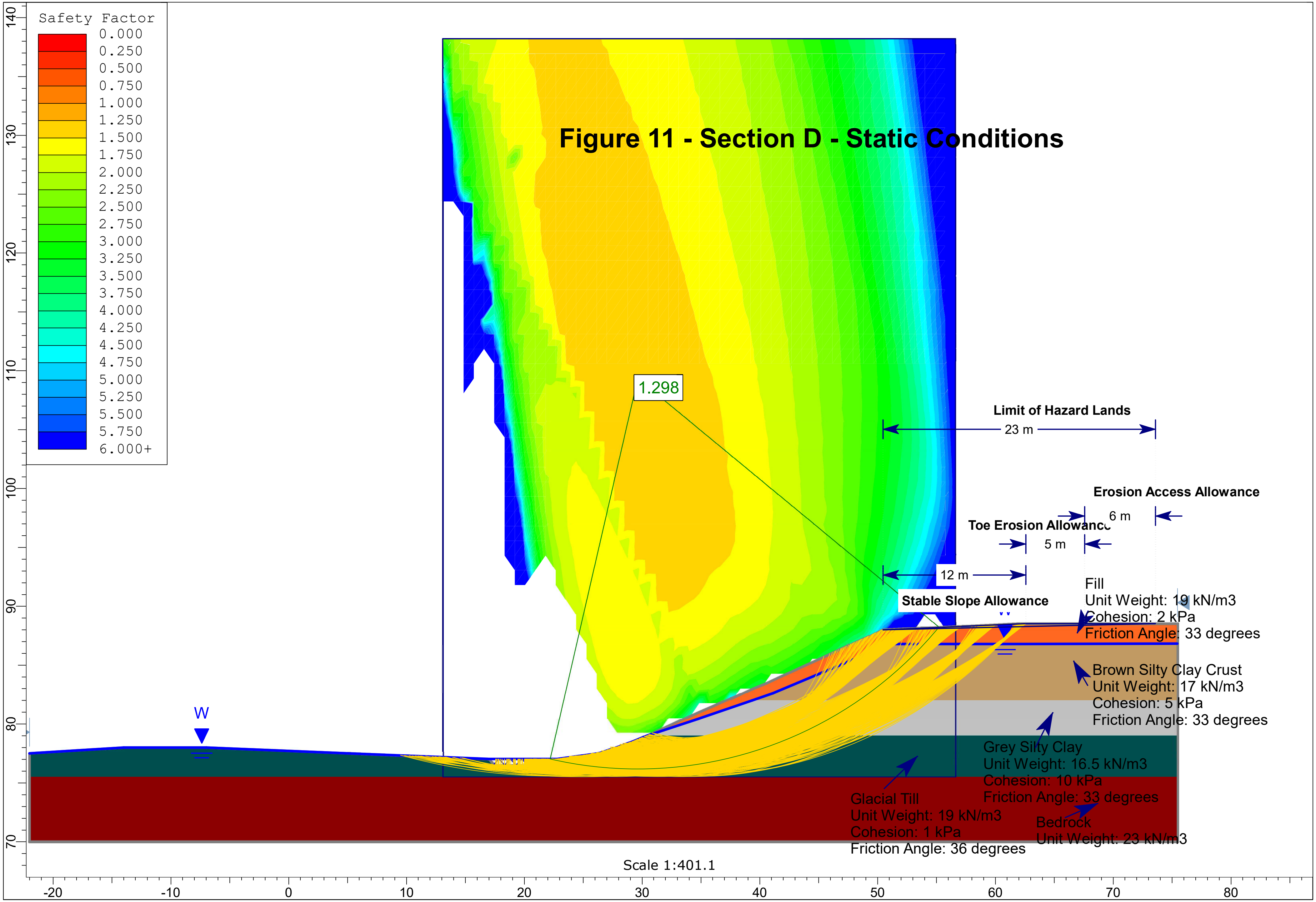


Figure 11 - Section D - Static Conditions

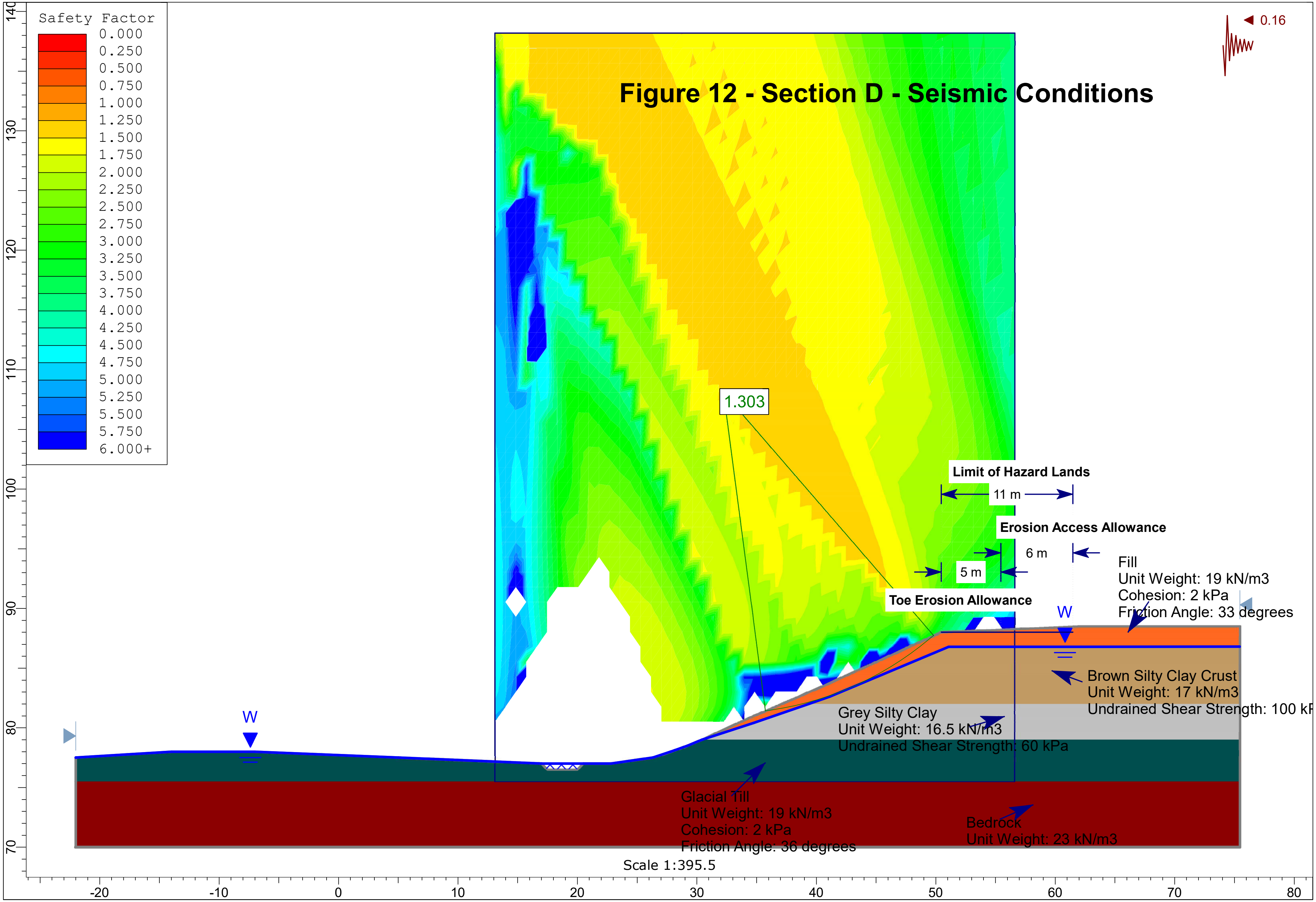
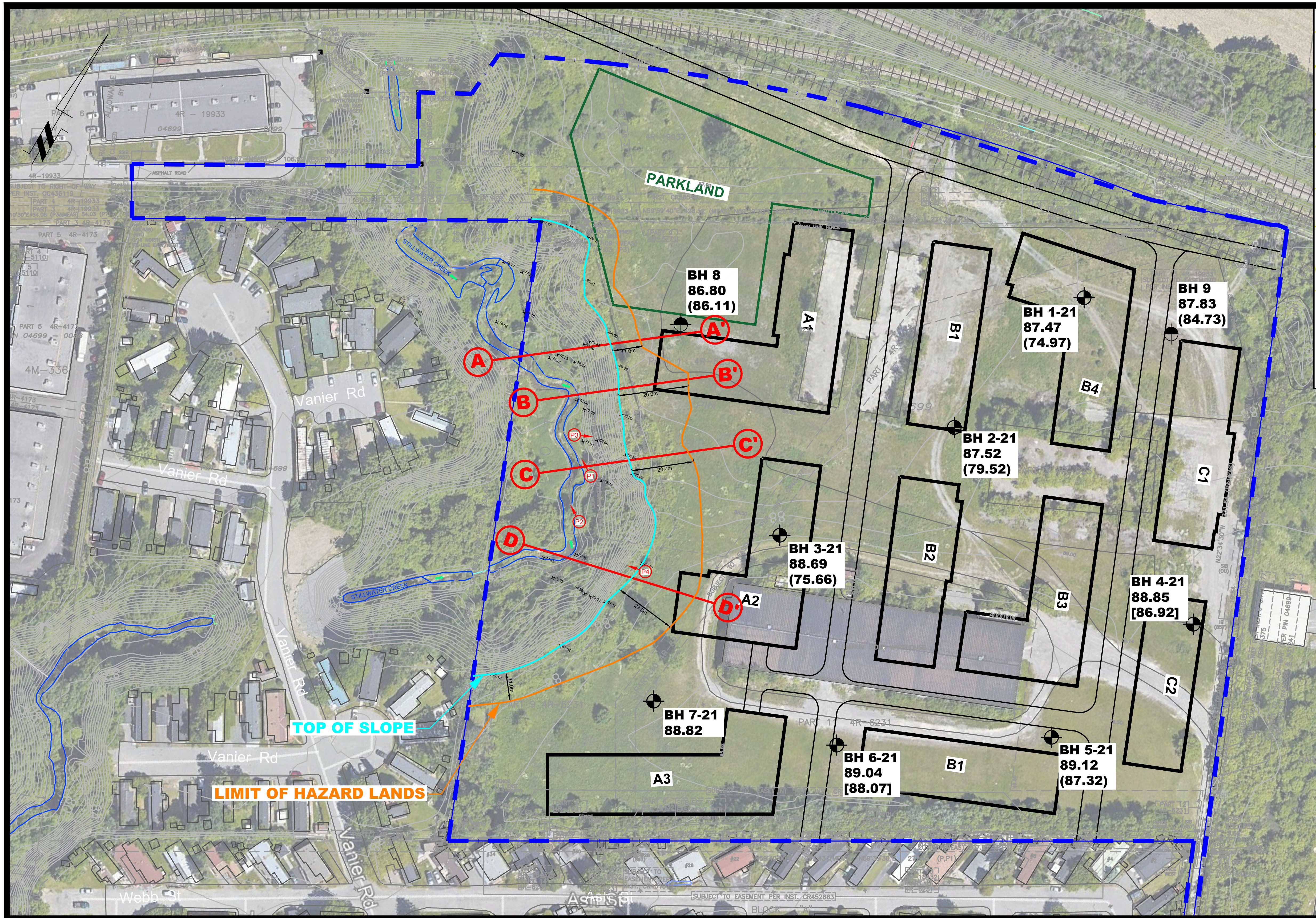
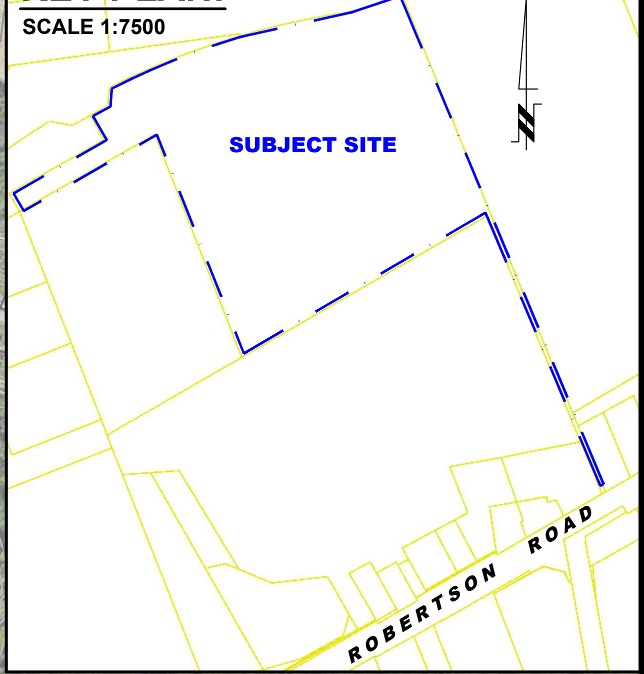


Figure 12 - Section D - Seismic Conditions



KEY PLAN:

SCALE 1:7500



LEGEND:

- ⊕ BOREHOLE LOCATION
- ⊕ BOREHOLE LOCATION, PREVIOUS INVESTIGATION (PATERSON, 2007)
- 88.85 GROUND SURFACE ELEVATION (m)
- (87.32) PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)
- [86.92] BEDROCK SURFACE ELEVATION (m)
- Ⓐ—Ⓐ' SLOPE STABILITY CROSS-SECTION
- Ⓟ2 PHOTO LOCATION

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:1500

0 25 50 75 100m

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Ottawa, Ontario K2E 7J5
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NO.	REVISIONS	DATE	INITIAL

THE PROPERTIES GROUP

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MIXED USE DEVELOPMENT
1987 ROBERTSON ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1500
Drawn by: NFRV
Checked by: NP
Approved by: DJG
Date: 04/2021
Report No.: PG5715-1
Dwg. No.: PG5715-1
Revision No.:

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