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# Geotechnical Investigation

Proposed Commercial and Residential Development
5 Orchard Drive
Ottawa, Ontario

Prepared For

Campanale Homes

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Report PG4428-1

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# Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned by Campanale Homes to conduct a geotechnical investigation for the proposed commercial and residential development to be located at 5 Orchard Drive in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

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

☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. 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

It is understood that the proposed development will consist of mixed-use buildings on the northern half of the site, and residential detached and semi-detached dwellings on the southern half of the site. Associated paved access lanes and landscaped areas are also expected as part of the development.

# 3.0 Methods of Investigation

## 3.1 Field Investigation

### Field Program

The field program for this investigation was carried out on November 4, 2015. At that time, a total of 15 boreholes were advanced to a maximum 5.4 m depth. The borehole locations are illustrated on Drawing PG4428-1 - 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 personnel from our geotechnical division under the direction of a senior engineer. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

### Sampling and In Situ Testing

Soil samples were recovered with a 50 mm diameter split-spoon sample or from the auger flights. The split-spoon samples were classified on site and placed in sealed plastic bags. All samples were transported to the our laboratory for examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS, and AU, 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.

Rock samples were recovered from borehole BH 15 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson’s laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data Sheets in Appendix 1.

The recovery value and 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 subsurface conditions observed at the borehole and test pits were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

### Groundwater

Flexible standpipes were installed in all boreholes, except BH 13, BH 14 and BH 15 where groundwater monitoring wells were installed, to monitor the groundwater levels subsequent to the completion of the sampling program.

## 3.2 Field Survey

The test holes were located in the field by Annis O'Sullivan Vollebekk. It is understood that the elevations are referenced to a geodetic datum. The ground surface elevation and location of the test holes are presented on Drawing PG4428-1 - Test Hole Location Plan in Appendix 2.

## 3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

# 4.0 Observations

## 4.1 Surface Conditions

Generally, the ground surface across the subject site is grass and tree covered, and slopes gradually downward to the east. Several piles of construction debris and fill were observed within the west portion of the site. A section of Poole Creek runs along the western property boundary. Based on our cursory review, the subject section of Poole Creek is approximately 4 to 5 m wide confined by 2 to 3 m high side banks. Boulders and cobbles were observed along the toe of slope at water level.

## 4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consists of a topsoil layer underlain by loose to compact brown silty sand layer and/or glacial till. The glacial till was observed to consist of brown silty sand with gravel, cobbles and boulders. Practical refusal to augering was encountered at all borehole locations completed during our current investigation.

Grey limestone bedrock was encountered below the silty sand and/or glacial till, and was cored at borehole BH 15 to a depth of 2.2 m below the bedrock surface. Based on the Rock Quality Designation (RQD) values of the recovered rock core, the limestone bedrock can be classified as good to excellent quality.

Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets provided in Appendix 1.

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone and dolomite of the Gull River formation. The overburden thickness is anticipated to be between 3 to 10 m depth.

## 4.3 Groundwater

The groundwater levels measured within the piezometers and monitoring wells installed at the borehole locations are presented in Table 1. It is important to note that groundwater readings within piezometers can be influenced by water perched within the borehole backfill material, which can lead to higher than normal groundwater level readings. Long-term groundwater levels can also be estimated based on field observations of the recovered soil samples, such as moisture levels, colouring and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater level can be expected between 3 to 4 m depth. Groundwater levels are subject to seasonal fluctuations and therefore could vary during 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 | 103.70 | 0.97 | 102.73 | November 18, 2015 |
| BH 2A | 103.98 | 1.82 | 102.16 | November 18, 2015 |
| BH 3 | 104.00 | 1.97 | 102.03 | November 18, 2015 |
| BH 4 | 104.17 | 1.43 | 102.74 | November 18, 2015 |
| BH 5 | 104.14 | Dry | - | November 18, 2015 |
| BH 6 | 104.34 | 1.60 | 102.74 | November 18, 2015 |
| BH 7 | 104.93 | 0.78 | 104.15 | November 18, 2015 |
| BH 8 | 104.78 | 0.76 | 104.02 | November 18, 2015 |
| BH 9 | 104.48 | 0.34 | 104.14 | November 18, 2015 |
| BH 10 | 104.63 | 0.41 | 104.19 | November 18, 2015 |
| BH 11 | 104.61 | 0.68 | 103.93 | November 18, 2015 |
| BH 12 | 104.28 | 0.37 | 103.91 | November 18, 2015 |
| BH 13 | 108.32 | 0.75 | 107.57 | November 18, 2015 |
| BH 14 | 108.26 | 0.71 | 107.55 | November 18, 2015 |
| BH 15 | 106.99 | 1.57 | 105.42 | November 18, 2015 |

# 5.0 Discussion

## 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed buildings. It is expected that the proposed structures will be founded by conventional shallow spread footings bearing on an undisturbed, compact silty sand, glacial till, or a clean, limestone bedrock bearing surface.

Bedrock removal may be required dependent on the founding depths of the proposed structures. Hoe ramming is an option where only small quantities of bedrock need to be removed. Where large quantities of bedrock need to be removed, line drilling and controlled blasting is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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 the proposed buildings and other settlement sensitive structures.

### Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

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

As a general guideline, peak particle velocities (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 also an experienced blasting consultant.

### Vibration Considerations

Construction operations are expected to 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, as much as possible, a cooperative environment with the residents.

The following construction equipments could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, 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). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed buildings.

### 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. The fill should be tested and approved prior to delivery to the site. The fill should be placed in lifts to a maximum of 300 mm thick 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 placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill 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 of a backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

## 5.3 Foundation Design

### Shallow Foundations

Footings placed on an undisturbed, compact silty sand bearing surface ca be designed using a factored bearing resistance value at Serviceability Limit States (SLS) of **150 kPa** and a bearing resistance value at Ultimate Limit States (ULS) of **250 kPa**.

Where silty sand bearing surface is found to be in a loose state of compactness, the area should be proof-rolled using a vibratory compactor and approved by the geotechnical consultant prior to placing footings.

Footings placed on an undisturbed, glacial till bearing surface can be designed using a bearing resistance value at SLS of **250 kPa** and a factored bearing resistance value at ULS of **350 kPa**.

Footings placed on a clean, surface sounded limestone bedrock surface can be designed using a bearing resistance value at SLS of **1,000 kPa** and a factored bearing resistance value at ULS of **2,000 kPa**.

The above noted bearing resistance values at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS.

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 an engineered fill, silty sand or glacial till bearing medium when a plane extending a minimum of 1.5H:1V, from the footing perimeter to the founding soil/engineered fill.

Where a building is founded partly on bedrock and partly on soils, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material at the soil/bedrock and bedrock/soil transitions. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m beyond both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

## 5.4 Design for Earthquakes

Based on current information, the foundations for the proposed buildings can be designed using a seismic site response **Class C** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A). A higher site class, such as Class A or B, may be applicable for foundation design for structures founded directly over a bedrock surface. However, a site specific seismic shear wave velocity test is required as per OBC 2012 to confirm the higher site class. The underlying soils are not considered to be susceptible to liquefaction.

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

With the removal of all topsoil and deleterious materials, within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab. The upper 200 mm of sub-slab fill should consist of an OPSS Granular A material for slab-on-grade construction and 19 mm clear crushed stone for basement slabs. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD.

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

## 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 structures. However, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m$^3$.

The foundations are expected be provided with perimeter drainage systems; therefore, the retained soils should be considered drained. However, if undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective unit weight of the retained soil can be taken as 13 $kN/m^3$, where applicable. A hydrostatic pressure should be added to the total static earth pressure for all subsurface units below the watertable when calculating the effective unit weight. The total earth pressure ($P_{AE}$) includes both the static earth pressure component ($P_o$) and the seismic component ($\Delta P_{AE}$).

**Lateral Earth Pressures**

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. Note that 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 seismic earth pressure ($\Delta P_{AE}$) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

$a_c$ = $(1.45-a_{max}/g)a_{max}$
$\gamma$ = 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 total earth pressure ($P_{AE}$) is considered to act at a height, h (m), from the base of the wall, where:

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

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

## 5.7 Pavement Structure

Car-only parking and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3.

**Table 2 - Recommended Pavement Structure - Car-Only Parking Areas**

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

**Table 3 - Recommended Pavement Structure
Access Lanes and Heavy-Truck Parking Areas**

| Thickness (mm) | Material Description |
|---|---|
| 40 | **Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete |
| 50 | **Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete |
| 150 | **BASE** - OPSS Granular A Crushed Stone |
| 450 | **SUBBASE** - OPSS Granular B Type II |
|  | **SUBGRADE** - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil, or fill |

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

## 5.8 Limit of Hazard Lands

A section of Poole Creek is located along the western property boundary of the site. The slope condition was reviewed by Paterson field personnel as part of the geotechnical investigation. Two (2) slope cross-sections were studied as the worst case scenarios, where Poole Creek is located at the slope toe. The cross section locations are presented on Drawing PG4428-1 - Test Hole Location Plan in Appendix 2. Generally, the riverbanks along both sides of Poole Creek are well vegetated and stable. The subject section of Poole Creek is approximately 4 to 5 m wide, approximately 0.3 to 0.6 m depth. The subject section of Poole Creek is confined by a 2 to 3 m high stable slope. The slope is observed to be well vegetated and stable with little to no signs of active erosion.

A slope stability analysis was carried out to determine the required stable slope allowance setback from the top of slope. Toe erosion and 6 m erosion access allowances were also considered in the determination of limits of hazard lands and are discussed on the following pages. **The proposed limit of hazard lands including the stable slope allowance, toe erosion allowance, and 6 m erosion access allowance and top of slope are shown on Drawing PG4428-1 - Test Hole Location Plan in Appendix 2.**

### Slope Stability Assessment

The analysis of slope stability was carried out using SLIDE, a computer program that 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 than 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.

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The cross-sections were analysed taking into account an elevated groundwater level, which represents a worse-case scenario that can reasonably be expected. Subsoil conditions at the cross-sections were inferred based on the findings at borehole locations along the top of slope and general knowledge of the area's geology.

### Stable Slope Allowance

The results of the stability analysis for static conditions at Sections A and B are presented in Figures 2a and 3a in Appendix 2. Sections A and B require a stable slope allowance due to the slope stability factor of safety being less than 1.5. It should be noted that the two cross-sections were analyzed as worst case scenarios for the subject slopes. Based on the soil conditions observed and slope profile along the subject section of Poole Creek, the remainder of the slope has the same slope stability factor of safety as Sections A and B.

The results of the analyses including seismic loading are shown in Figures 2b and 3b for the slope sections. The results indicate that the factor of safety for these sections are less than 1.1 and that a stable slope allowance is required.

The existing vegetation on the slope face should not be removed as it contributes to the stability of the slope and reduces erosion. If the existing vegetation needs to be removed, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed or an erosional control blanket be placed across the exposed slope face.

### Toe Erosion and Erosion Access Allowance

The toe erosion allowance for the subject slope was based on the nature of the soils, the observed current erosional activities and the width and location of the current watercourse. Little to no signs of active erosion were noted along the subject section of Poole Creek. It is considered that a toe erosion allowance of 2 m and an erosion access allowance of 6 m are required from the top of slope.

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage and Backfill

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

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 composite drainage system, 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 be used for backfill material.

## 6.2 Protection of Footings Against Frost Action

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

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

## 6.3 Excavation Side Slopes

The 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 excavation 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 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

At least 150 mm of OPSS Granular A crushed stone should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A or clean sand if a concrete pipe is used. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

Generally, it should be possible to re-use the moist (not wet) site excavated silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions.

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.

**Bedrock/Soil Transition**

In areas where the service subgrade transitions from soil to bedrock, it is recommended that the founding medium be inspected in the filed to determine how steeply the bedrock surface, where encountered, drops off. A transition treatment should be provided where the bedrock slopes at more than 3H:1V. At these locations, the bedrock should be excavated, and extra bedding placed to provide a 3H:1V transition from the bedrock subgrade toward the soil subgrade. This treatment will reduce the propensity for bending stresses to occur in the watermain.

## 6.5 Groundwater Control

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.

It is expected that the flow of groundwater into the excavation will be moderate through the sides of the excavation. However, it is expected that the groundwater inflow will be controllable using open sumps and pumps.

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

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 MOECC review of the PTTW application.

## 6.6 Winter Construction

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

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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 and 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 buildings and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

## 6.7 Landscaping Considerations

Based on our review of the subsurface conditions, tree planting restrictions are not required for the proposed development site.

# 7.0 Recommendations

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

❏ Review detailed grading plan(s) from a geotechnical perspective.

❏ Observation of all bearing surfaces prior to the placement of concrete.

❏ Sampling and testing of the concrete and fill materials used.

❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❏ Observation of all subgrades prior to backfilling.

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

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

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

# 8.0 Statement of Limitations

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

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our 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 Campanale Homes or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

**Paterson Group Inc.**

Scott Dennis

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**Report Distribution:**

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# APPENDIX 1

## SOIL PROFILE AND TEST DATA SHEETS

## SYMBOLS AND TERMS

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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** 4 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH 1

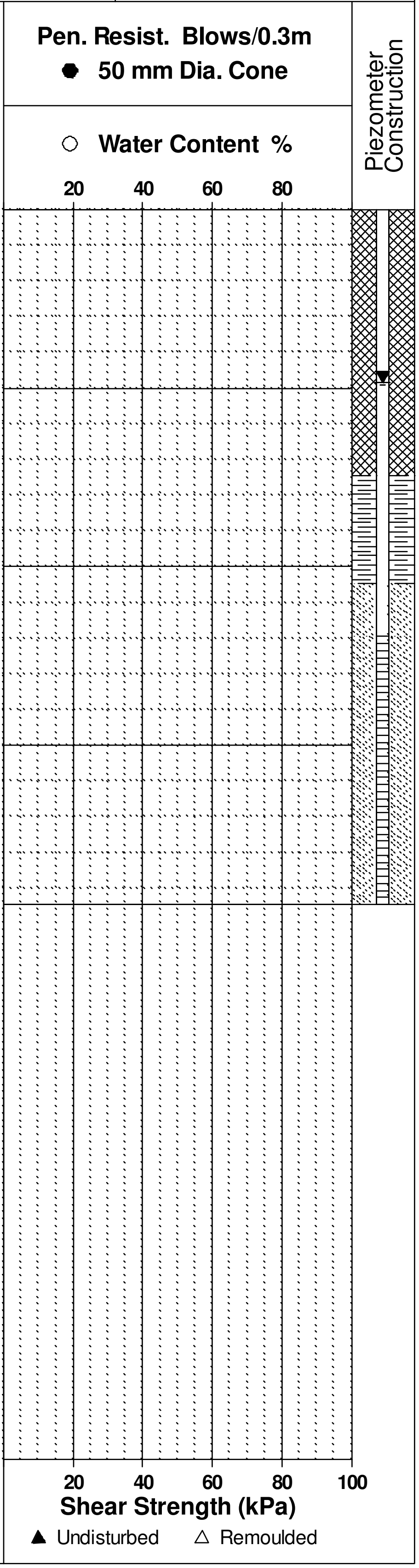
| SOIL DESCRIPTION | SAMPLE | | | | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| | TYPE | NUMBER | % RECOVERY | N VALUE or RQD | | |
| **GROUND SURFACE** | | | | | 0 | 103.70 |
| **TOPSOIL** 0.20 | AU | 1 | | | | |
| Loose to very loose, brown **SILTY SAND** | SS | 2 | 67 | 5 | 1 | 102.70 |
| 1.78 | SS | 3 | 88 | 3 | 2 | 101.70 |
| **GLACIAL TILL:** Very loose to compact, brown silty sand with gravel, trace cobbles | SS | 4 | 67 | 22 | 3 | 100.70 |
| - dense by 3.2m depth | SS | 5 | 79 | 38 | | |
| - grey by 3.6m depth 3.89 | SS | 6 | 100 | 50+ | | |

End of Borehole

Practical refusal to augering at 3.89m depth

(GWL @ 0.97m-Nov. 18, 2015)

Based on field observations, the long-term GWL was encountered @ 3.6m depth



**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** 4 November 2015

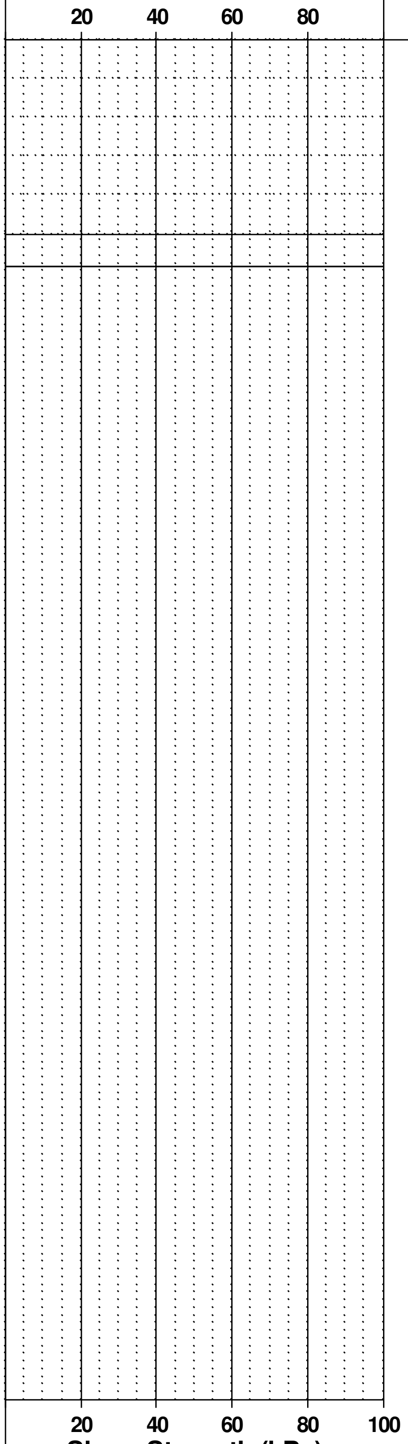
**FILE NO.** PG3544

**HOLE NO.** BH 2

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 103.96 |
| **TOPSOIL** 0.20 | ■ | | | | | | |
| Dense, brown **SILTY SAND** | | AU | 1 | | | | |
| | | SS | 2 | 44 | 50+ | 1 | 102.96 |
| 1.17 | | | | | | | |
| End of Borehole<br>Practical refusal to augering at 1.17m depth<br>(BH dry upon completion) | | | | | | | |

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

○ Water Content %



**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

Piezometer Construction

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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

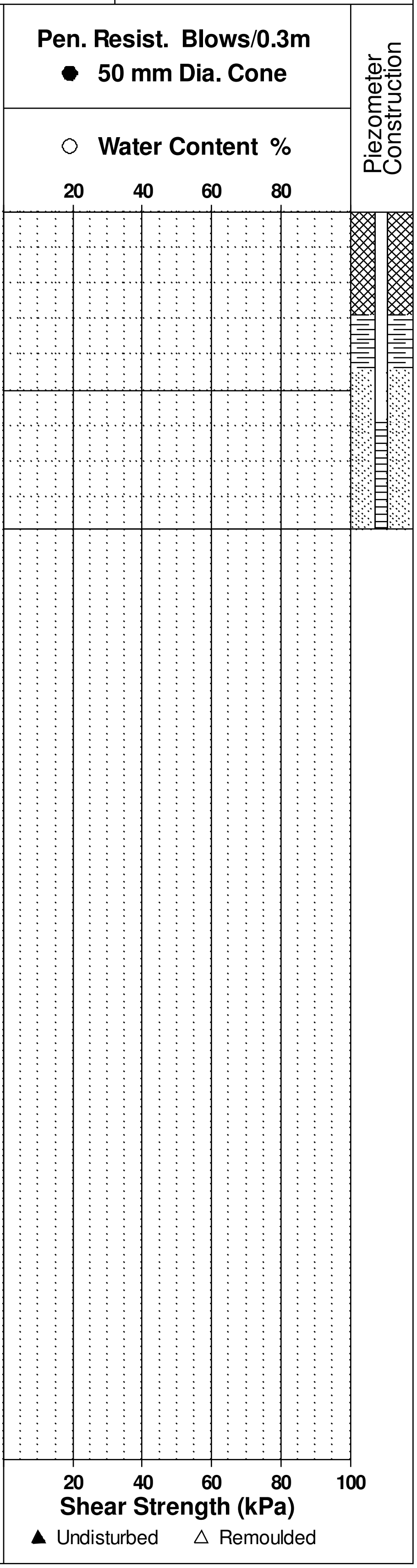
**BORINGS BY** CME 55 Power Auger

**DATE** 4 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH 2A

| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | 0 | 103.98 |
| **TOPSOIL** 0.20 | | | | | | |
| Dense, brown **SILTY SAND** 1.17 | | | | | 1 | 102.98 |
| **GLACIAL TILL:** Dense, brown silty sand, trace cobbles and boulders 1.78 | SS | 1 | 55 | 50+ | | |
| End of Borehole<br>Practical refusal to augering at 1.78m depth<br>(BH dry - Nov. 18, 2015) | | | | | | |



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**FILE NO.** PG3544

**REMARKS**

**HOLE NO.** BH 3

**BORINGS BY** CME 55 Power Auger

**DATE** 4 November 2015

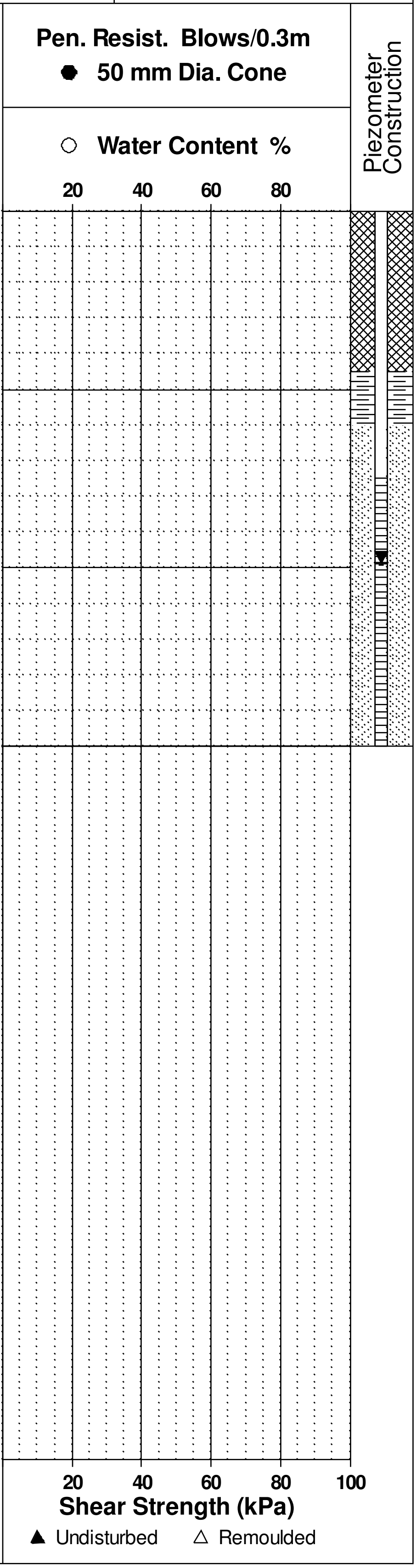
| SOIL DESCRIPTION | STRATA PLOT | SAMPLE | | | | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| | | TYPE | NUMBER | % RECOVERY | N VALUE or RQD | | |
| **GROUND SURFACE** | | | | | | 0 | 104.00 |
| **TOPSOIL** 0.18 | | AU | 1 | | | | |
| Loose to compact, brown **SILTY SAND** 1.65 | | SS | 2 | 83 | 4 | 1 | 103.00 |
| **GLACIAL TILL:** Compact, brown silty sand, some gravel, cobbles and boulders | | SS | 3 | 58 | 15 | 2 | 102.00 |
| | | SS | 4 | 42 | 25 | | |
| 3.00 | | | | | | 3 | 101.00 |

End of Borehole

Practical refusal to augering at 3.00m depth

(GWL @ 1.97m-Nov. 18, 2015)

Based on field observations, the long-term GWL was not encountered



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** 4 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH 4

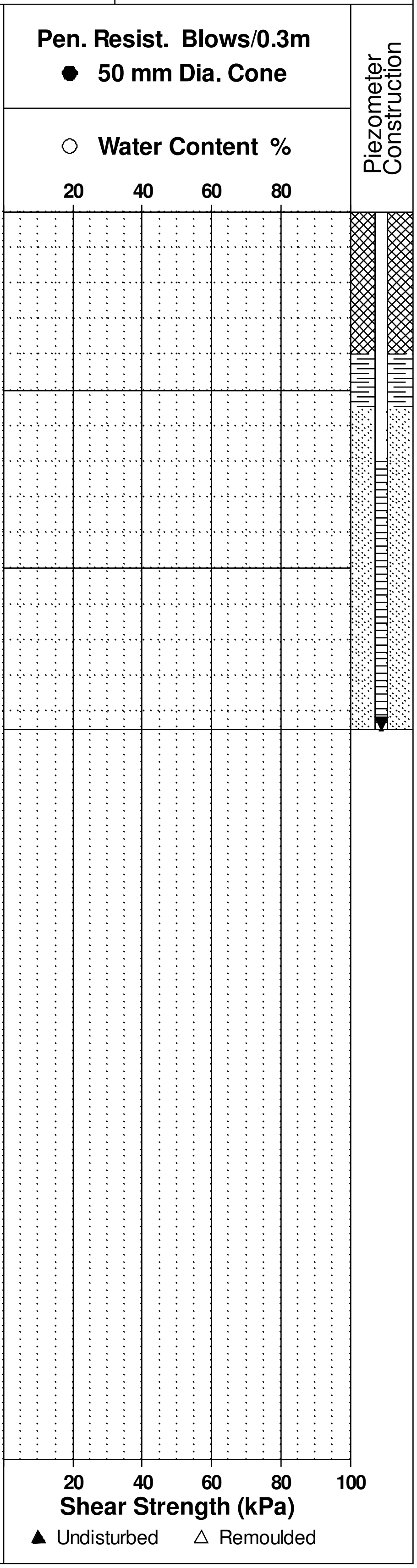
| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 104.17 |
| **TOPSOIL** 0.15 | | | | | | | |
| Dense, brown **SILTY SAND,** trace gravel 0.51 | | AU | 1 | | | | |
| **GLACIAL TILL:** Dense to very dense, brown silty sand, some gravel, cobbles and boulders | | SS | 2 | 83 | 34 | 1 | 103.17 |
| | | SS | 3 | 100 | 35 | 2 | 102.17 |
| 2.90 | | SS | 4 | 65 | 50+ | | |

End of Borehole

Practical refusal to augering at 2.90m depth

(GWL @ 1.43m-Nov. 18, 2015)

Based on field observations, the long-term GWL was not encountered



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## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**FILE NO.** **PG3544**

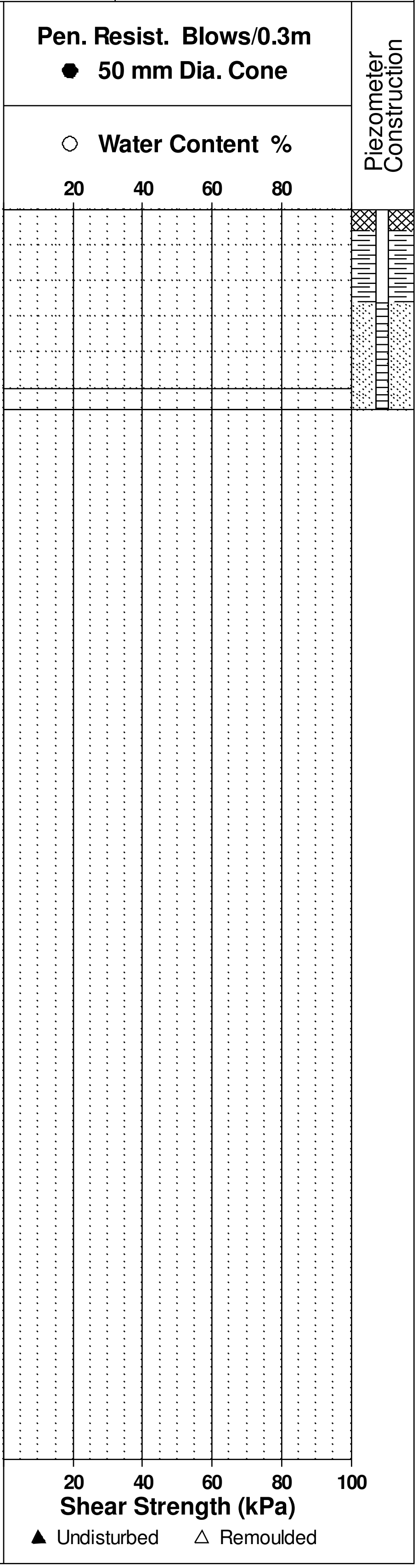
**REMARKS**

**HOLE NO.** **BH 5**

**BORINGS BY** CME 55 Power Auger

**DATE** 4 November 2015

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 104.14 |
| **TOPSOIL** 0.18 | | | | | | | |
| Dense, brown **SILTY SAND,** trace gravel | | AU | 1 | | | | |
| | | SS | 2 | 70 | 50+ | 1 | 103.14 |
| 1.12 | | | | | | | |
| End of Borehole<br>Practical refusal to augering at 1.12m depth<br>(BH dry - Nov. 18, 2015) | | | | | | | |



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.
**REMARKS**
**BORINGS BY** CME 55 Power Auger
**DATE** 4 November 2015
**FILE NO.** PG3544
**HOLE NO.** BH 6

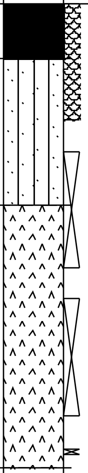
| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | 0 | 104.34 |
| **TOPSOIL** 0.28 | AU | 1 | | | | |
| Compact, brown **SILTY SAND,** trace gravel 1.04 | SS | 2 | 88 | 25 | 1 | 103.34 |
| **GLACIAL TILL:** Dense, brown silty sand with gravel, trace cobbles | SS | 3 | 100 | 31 | 2 | 102.34 |
| 2.39 | SS | 4 | 50 | 50+ | | |

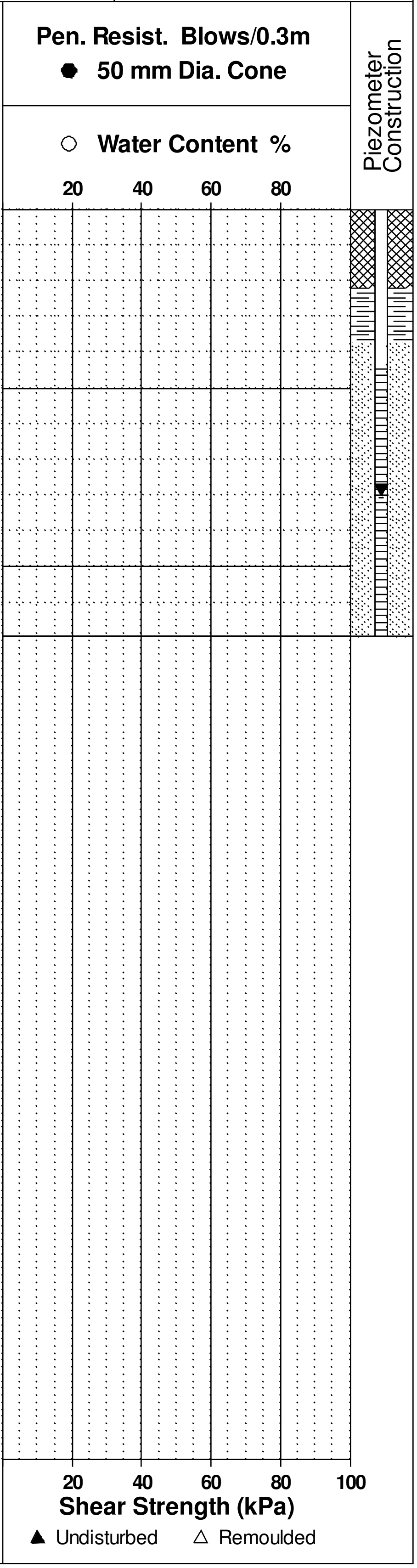
End of Borehole

Practical refusal to augering at 2.39m depth

(GWL @ 1.60m-Nov. 18, 2015)

Based on field observations, the long-term GWL was not encountered





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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** 4 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH 7

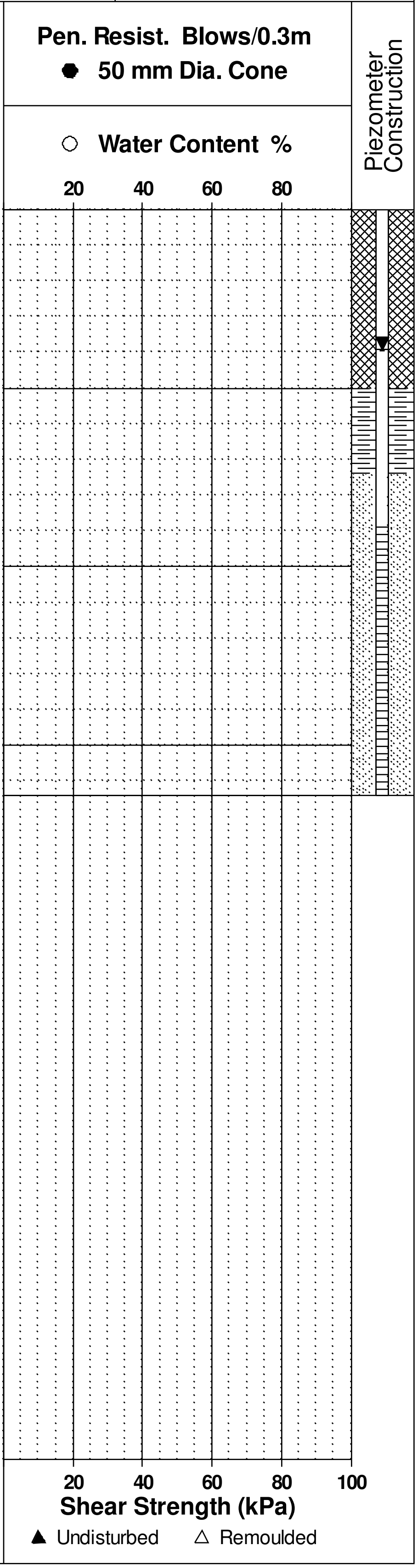
| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | 0 | 104.93 |
| **TOPSOIL** 0.15 | | | | | | |
| Compact, brown **SILTY SAND,** trace gravel 0.60 | AU | 1 | | | | |
| **GLACIAL TILL:** Compact to dense, brown silty sand, some gravel, cobbles and boulders | SS | 2 | 67 | 18 | 1 | 103.93 |
| | SS | 3 | 92 | 69 | 2 | 102.93 |
| | SS | 4 | 100 | 47 | 3 | 101.93 |
| 3.28 | SS | 5 | 67 | 50+ | | |

End of Borehole

Practical refusal to augering at 3.28m depth

(GWL @ 0.78m-Nov. 18, 2015)

Based on field observations, the long-term GWL was not encountered



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## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** 4 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH 8

| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | 0 | 104.78 |
| **TOPSOIL** 0.18 | | | | | | |
| Loose, brown **SILTY SAND,** trace gravel 0.66 | AU | 1 | | | | |
| **GLACIAL TILL:** Loose to dense, brown silty sand, some gravel, cobbles and boulders | SS | 2 | 48 | 8 | 1 | 103.78 |
| 1.73 | SS | 3 | 86 | 50+ | | |

End of Borehole

Practical refusal to augering at 1.73m depth

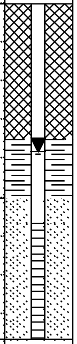
(GWL @ 0.76m-Nov. 18, 2015)

Based on field observations, the long-term GWL was not encountered

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

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

Piezometer Construction



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

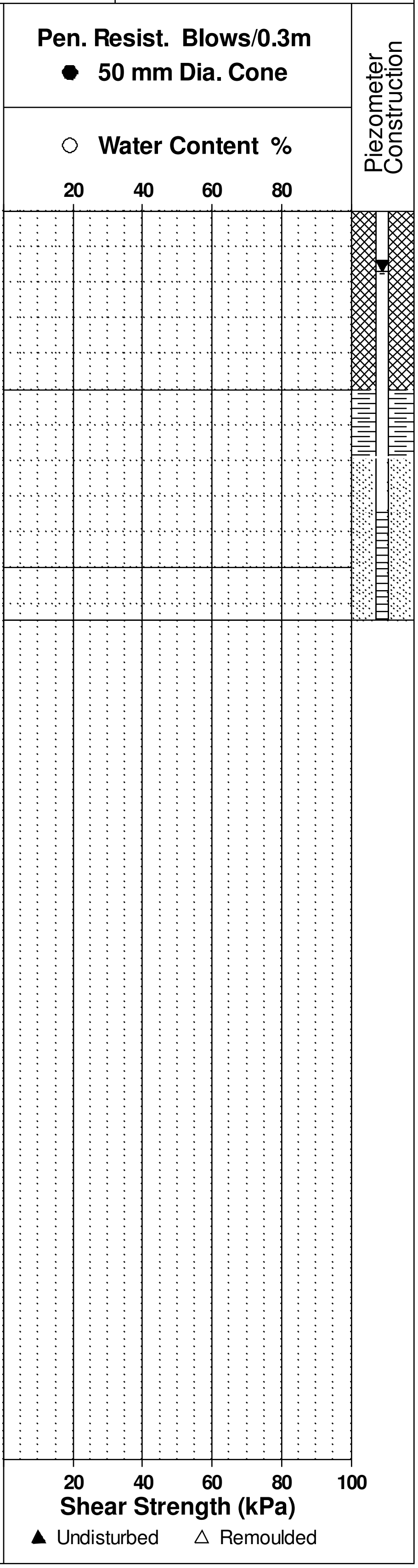
**BORINGS BY** CME 55 Power Auger

**DATE** 5 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH 9

| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | 0 | 104.48 |
| **TOPSOIL** 0.18 | | | | | | |
| Compact, brown **SILTY SAND,** trace gravel 0.60 | AU | 1 | | | | |
| **GLACIAL TILL:** Compact to dense, brown silty sand with gravel, cobbles, boulders | SS | 2 | 67 | 12 | 1 | 103.48 |
| | SS | 3 | 88 | 47 | 2 | 102.48 |
| 2.29 | | | | | | |
| End of Borehole<br>Practical refusal to augering at 2.29m depth<br>(GWL @ 0.34m-Nov. 18, 2015)<br>Based on field observations, the long-term GWL was not encountered | | | | | | |



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

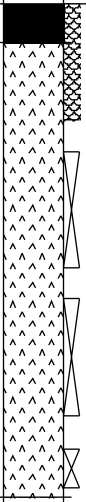
**BORINGS BY** CME 55 Power Auger

**DATE** 5 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH10

| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | 0 | 104.63 |
| **TOPSOIL** 0.20 | AU | 1 | | | | |
| **GLACIAL TILL:** Compact to dense, brown silty sand with gravel, cobbles and boulders | SS | 2 | 71 | 11 | 1 | 103.63 |
| | SS | 3 | 83 | 10 | 2 | 102.63 |
| 2.54 | SS | 4 | 75 | 50+ | | |

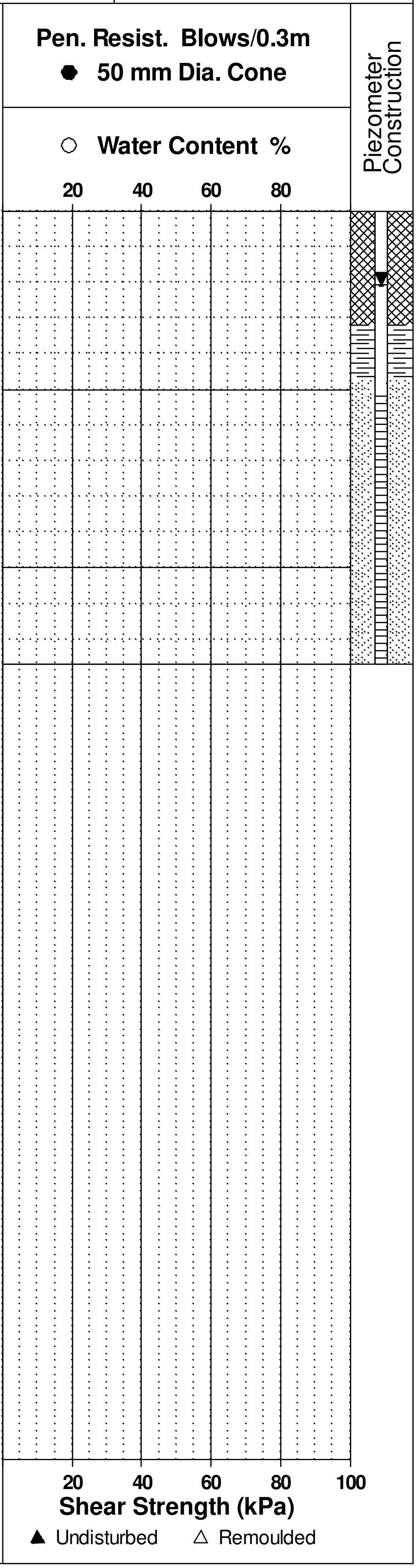

STRATA PLOT

End of Borehole

Practical refusal to augering at 2.54m depth

(GWL @ 0.41m-Nov. 18, 2015)

Based on field observations, the long-term GWL was not encountered



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**FILE NO.** PG3544

**REMARKS**

**HOLE NO.** BH11

**BORINGS BY** CME 55 Power Auger

**DATE** 5 November 2015

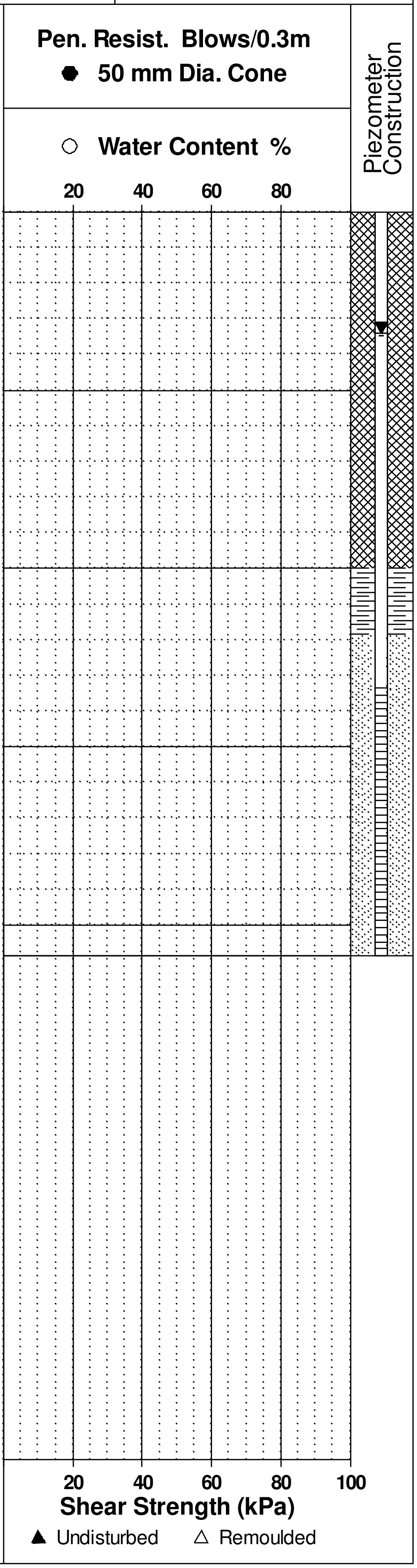
| SOIL DESCRIPTION | | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 104.61 |
| **TOPSOIL** | 0.13 | | | | | | |
| Loose, brown **SILTY SAND,** trace gravel | 0.33 | AU | 1 | | | | |
| **GLACIAL TILL:** Loose to dense, brown silty sand with gravel, cobbles, boulders | | SS | 2 | 4 | 7 | 1 | 103.61 |
| | | SS | 3 | 58 | 20 | 2 | 102.61 |
| | | SS | 4 | 83 | 9 | | |
| | | SS | 5 | 79 | 48 | 3 | 101.61 |
| - grey by 3.6m depth | | SS | 6 | 78 | 50+ | 4 | 100.61 |
| | 4.17 | | | | | | |

End of Borehole

Practical refusal to augering at 4.17m depth

(GWL @ 0.68m-Nov. 18, 2015)

Based on field observations, the long-term GWL was not encountered



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**FILE NO.** PG3544

**REMARKS**

**HOLE NO.** BH12

**BORINGS BY** CME 55 Power Auger

**DATE** 5 November 2015

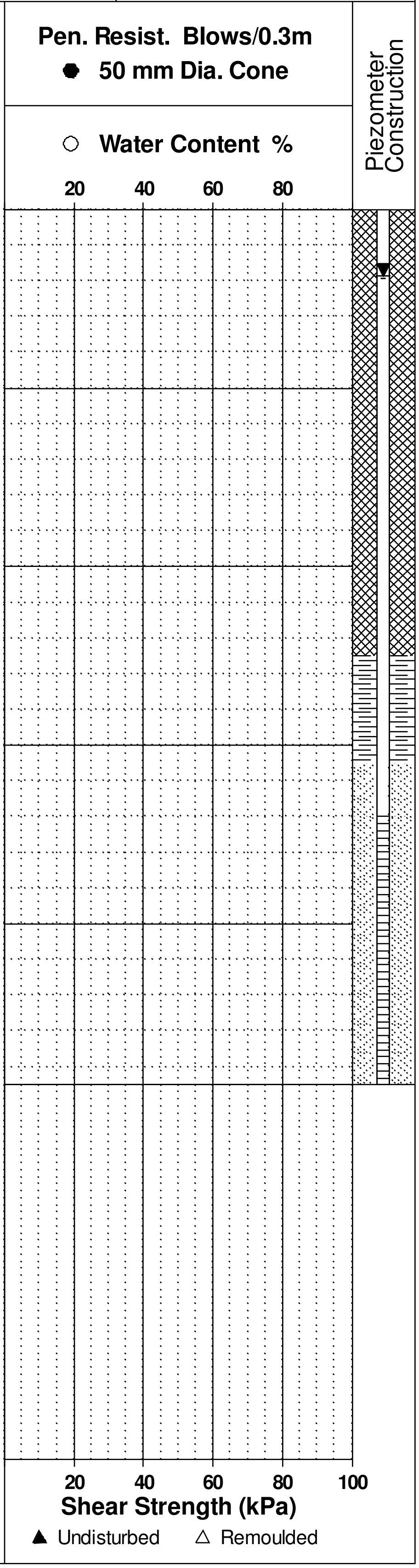
| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 104.28 |
| **TOPSOIL** 0.20 | | AU | 1 | | | | |
| Loose, brown **SILTY CLAY TO CLAYEY SILT,** trace gravel 1.27 | | SS | 2 | 25 | 4 | 1 | 103.28 |
| **GLACIAL TILL:** Compact to dense, brown silty sand with gravel, cobbles, boulders | | SS | 3 | 75 | 10 | 2 | 102.28 |
| | | SS | 4 | 67 | 21 | 3 | 101.28 |
| - grey by 3.6m depth | | SS | 5 | 67 | 16 | | |
| | | SS | 6 | 88 | 33 | 4 | 100.28 |
| 4.90 | | | | | | | |

End of Borehole

Practical refusal to augering at 4.90m depth

(GWL @ 0.37m-Nov. 18, 2015)

Based on field observations, the long-term GWL was not encountered



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**FILE NO.** PG3544

**REMARKS**

**HOLE NO.** BH13

**BORINGS BY** CME 55 Power Auger

**DATE** 5 November 2015

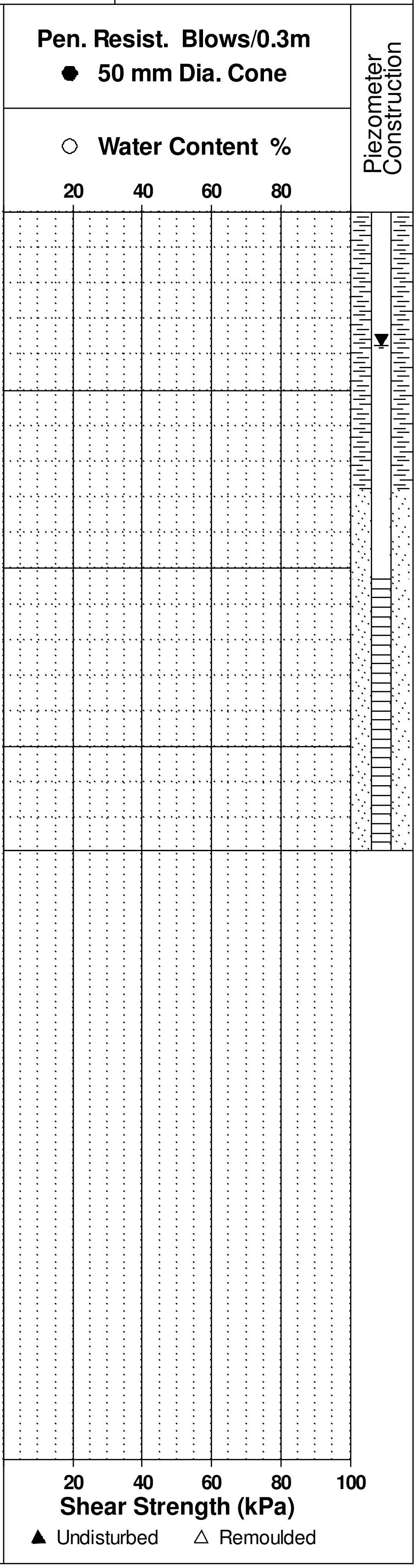
| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 108.32 |
| **FILL:** Dark brown silty sand with crushed stone and cobbles | | AU | 1 | | | | |
| | | SS | 2 | 21 | 10 | 1 | 107.32 |
| 1.42 | | | | | | | |
| **FILL:** Crushed stone with silty sand, some cobbles | | SS | 3 | 33 | 14 | 2 | 106.32 |
| 2.23 | | | | | | | |
| **GLACIAL TILL:** Very dense, brown silty sand with gravel, cobbles and boulders | | SS | 4 | 83 | 54 | 3 | 105.32 |
| | | SS | 5 | 92 | 50+ | | |
| 3.58 | | | | | | | |

End of Borehole

Practical refusal to augering at 3.58m depth

(GWL @ 0.75m-Nov. 16, 2015)

Based on field observations, the long-term GWL was not encountered



**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** 5 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH14

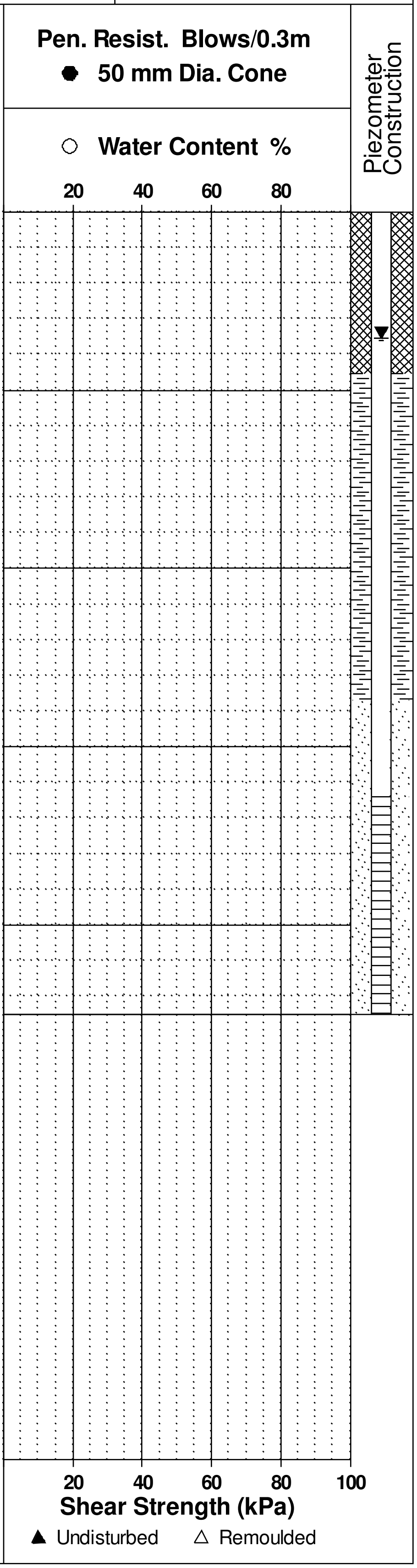
| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | 0 | 108.26 |
| **FILL:** Crushed stone with silty sand | AU | 1 | | | | |
| 0.71 | | | | | | |
| **GLACIAL TILL:** Compact to dense, brown silty sand with gravel, cobbles, boulders | SS | 2 | 100 | 24 | 1 | 107.26 |
| | SS | 3 | 100 | 47 | 2 | 106.26 |
| | SS | 4 | 100 | 19 | | |
| | | | | | 3 | 105.26 |
| | SS | 5 | 55 | 32 | | |
| | | | | | 4 | 104.26 |
| 4.50 | | | | | | |

End of Borehole

Practical refusal to augering at 4.50m depth

(GWL @ 0.71m-Nov. 16, 2015)

Based on field observations, the long-term GWL was not encountered



## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial and Residential Development**
**5 Orchard Drive, Ottawa, Ontario**

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

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** 5 November 2015

**FILE NO.** PG3544

**HOLE NO.** BH15

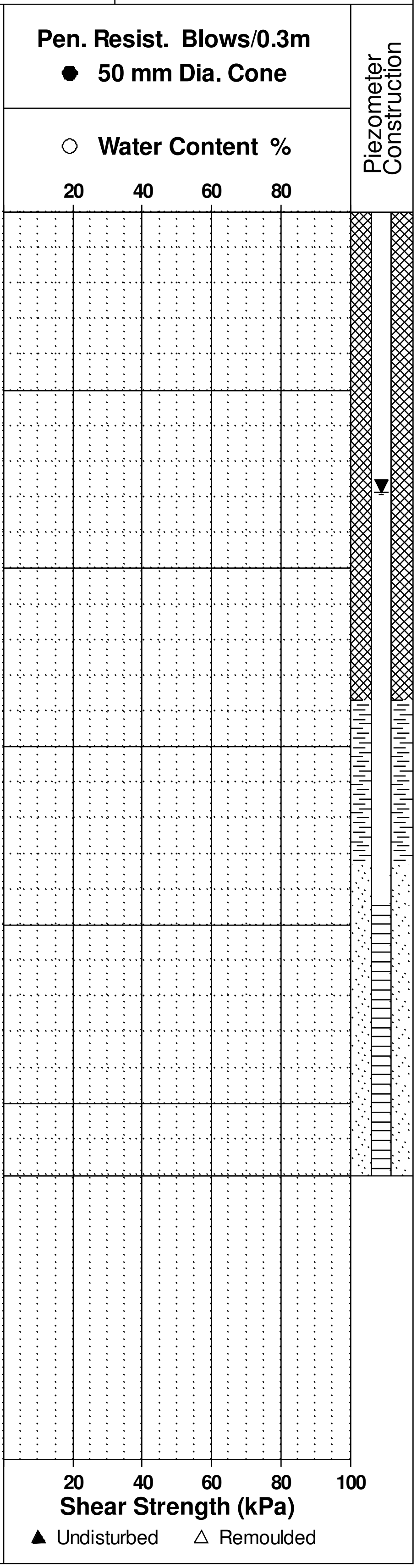
| SOIL DESCRIPTION | TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|
| GROUND SURFACE | | | | |
| **FILL:** Dark brown sandy silt with gravel, some cobbles | AU | 1 | | |
| 0.66 | | | | |
| **GLACIAL TILL:** Compact to dense, brown silty sand with gravel, cobbles, boulders | SS | 2 | 83 | 12 |
| | SS | 3 | 88 | 26 |
| | SS | 4 | 100 | 38 |
| | SS | 5 | 71 | 50+ |
| 3.22 | | | | |
| **BEDROCK:** Grey limestone | RC | 1 | 88 | 77 |
| | RC | 2 | 100 | 97 |
| 5.41 | | | | |

End of Borehole

(GWL @ 1.57m-Nov. 16, 2015)

Based on field observations, the long-term GWL was not encountered

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 106.99 |
| 1 | 105.99 |
| 2 | 104.99 |
| 3 | 103.99 |
| 4 | 102.99 |
| 5 | 101.99 |



# SYMBOLS AND TERMS

## SOIL DESCRIPTION

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

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

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

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

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

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

# SYMBOLS AND TERMS (continued)

## SOIL DESCRIPTION (continued)

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

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

## ROCK DESCRIPTION

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

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

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

## SAMPLE TYPES

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

## SYMBOLS AND TERMS (continued)

## GRAIN SIZE DISTRIBUTION

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

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

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

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

## CONSOLIDATION TEST

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

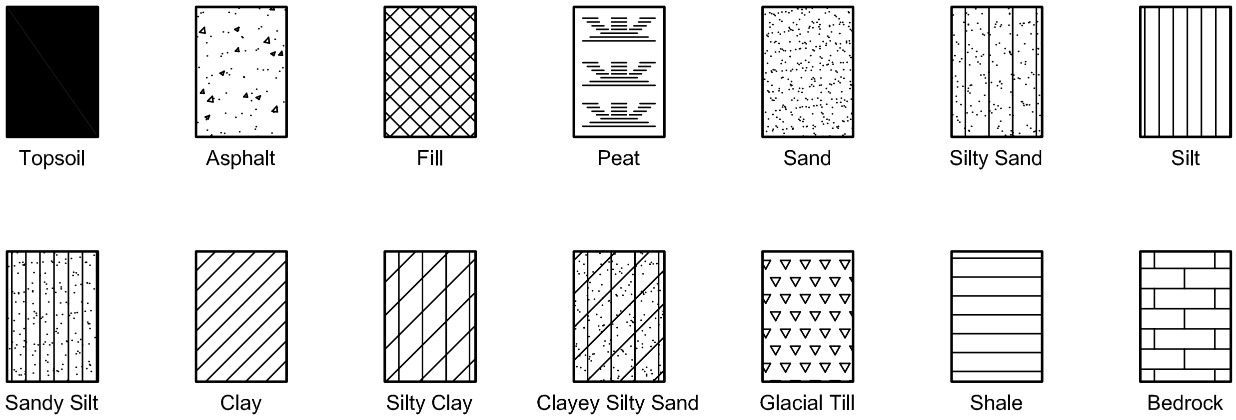
OC Ratio Overconsolidaton ratio = $p'_c / p'_o$
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

## PERMEABILITY TEST

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

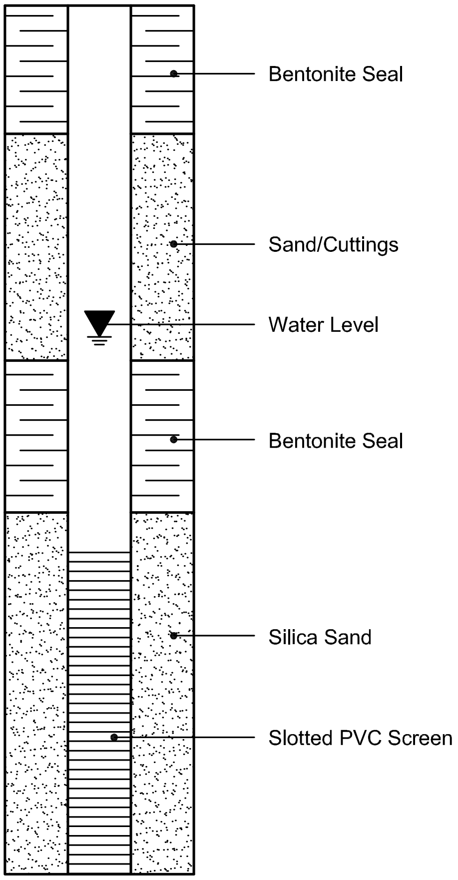
## SYMBOLS AND TERMS (continued)

### STRATA PLOT

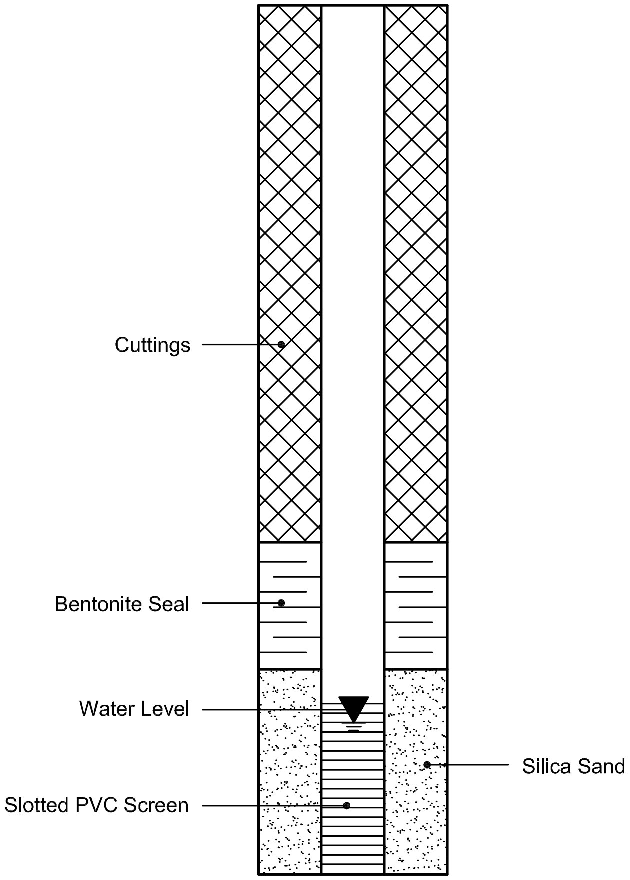


### MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



# APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2A TO 3B - SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG4428-1 - TEST HOLE LOCATION PLAN

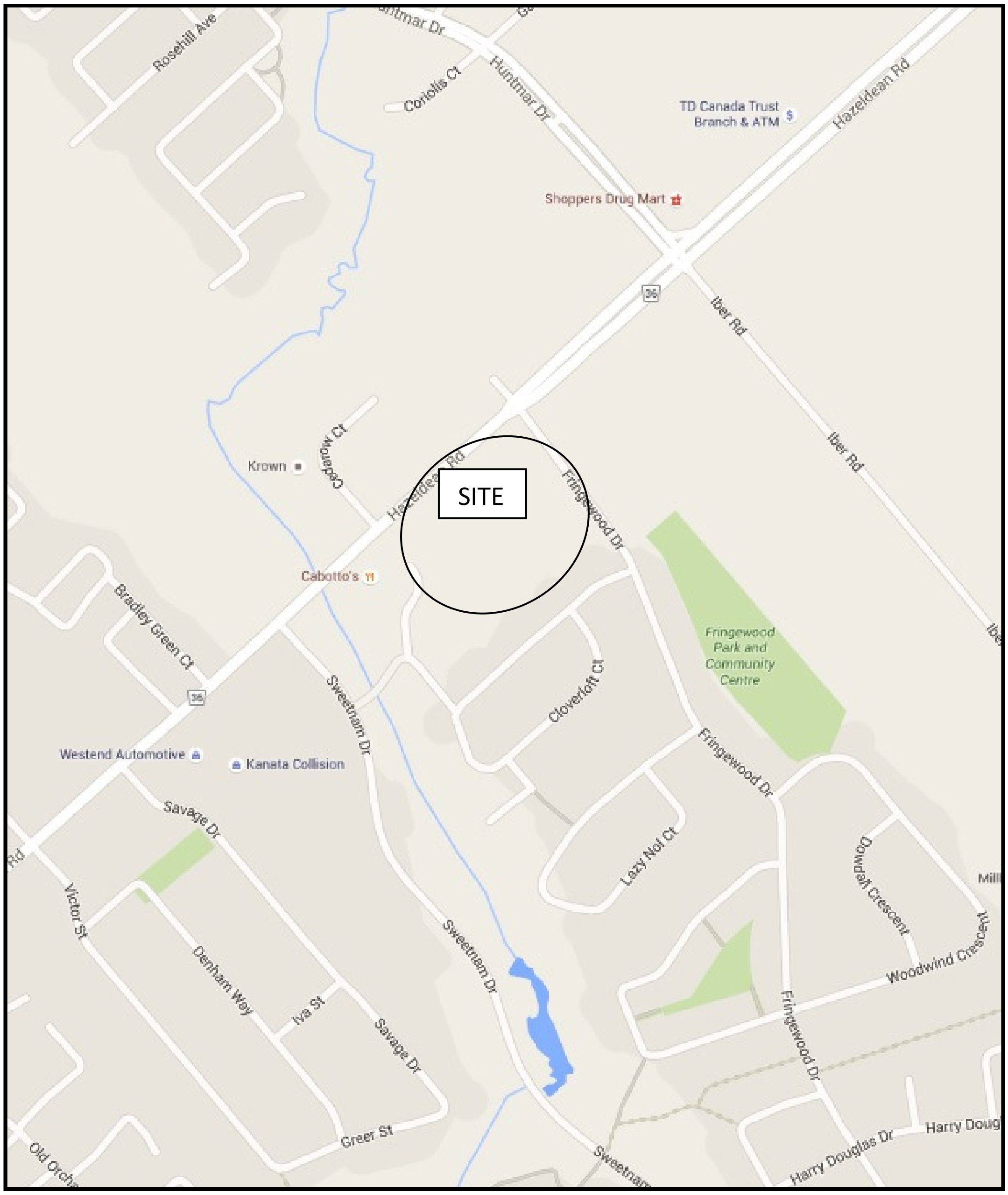
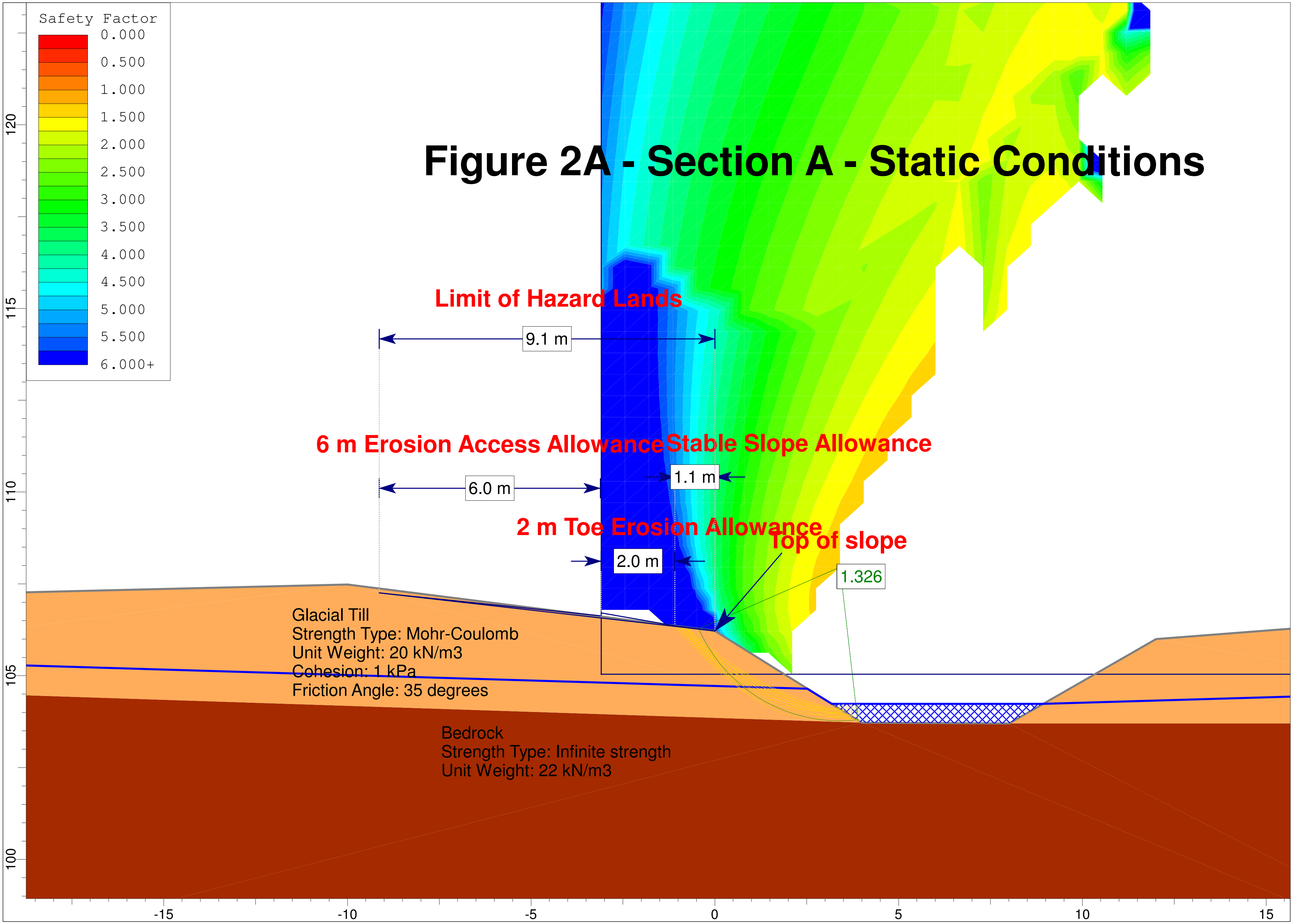


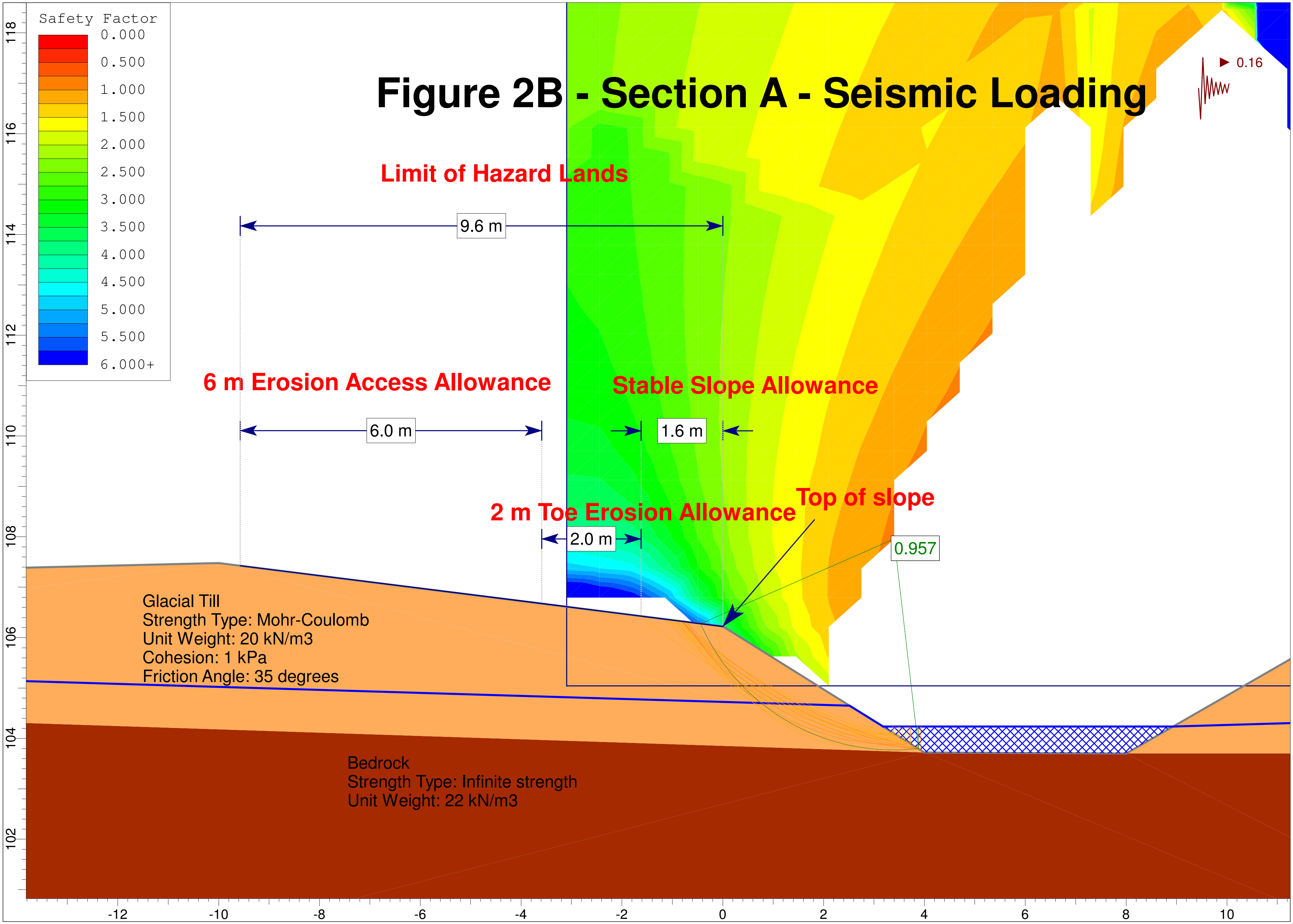
FIGURE 1

KEY PLAN

# Figure 2A - Section A - Static Conditions



# Figure 2B - Section A - Seismic Loading



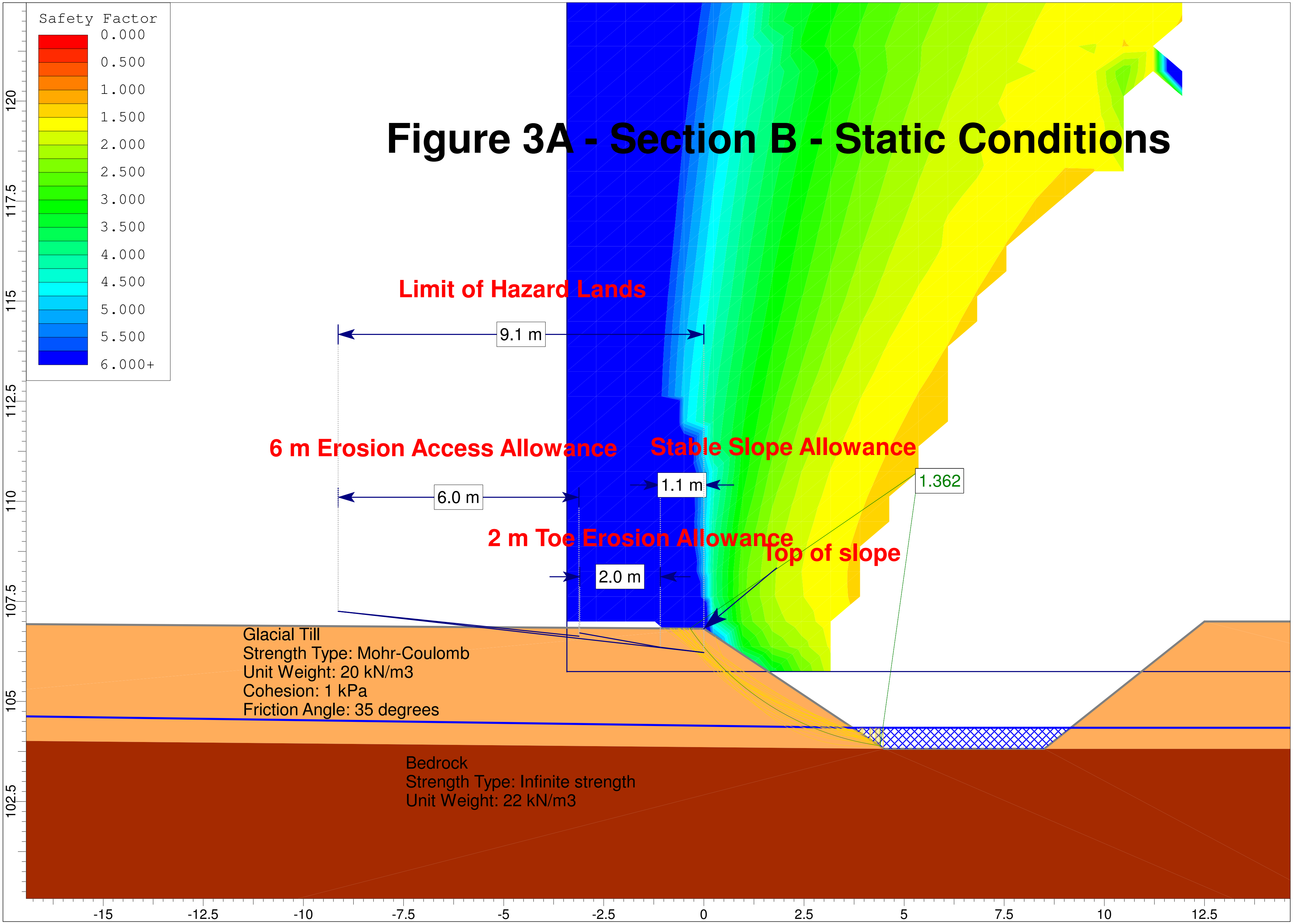
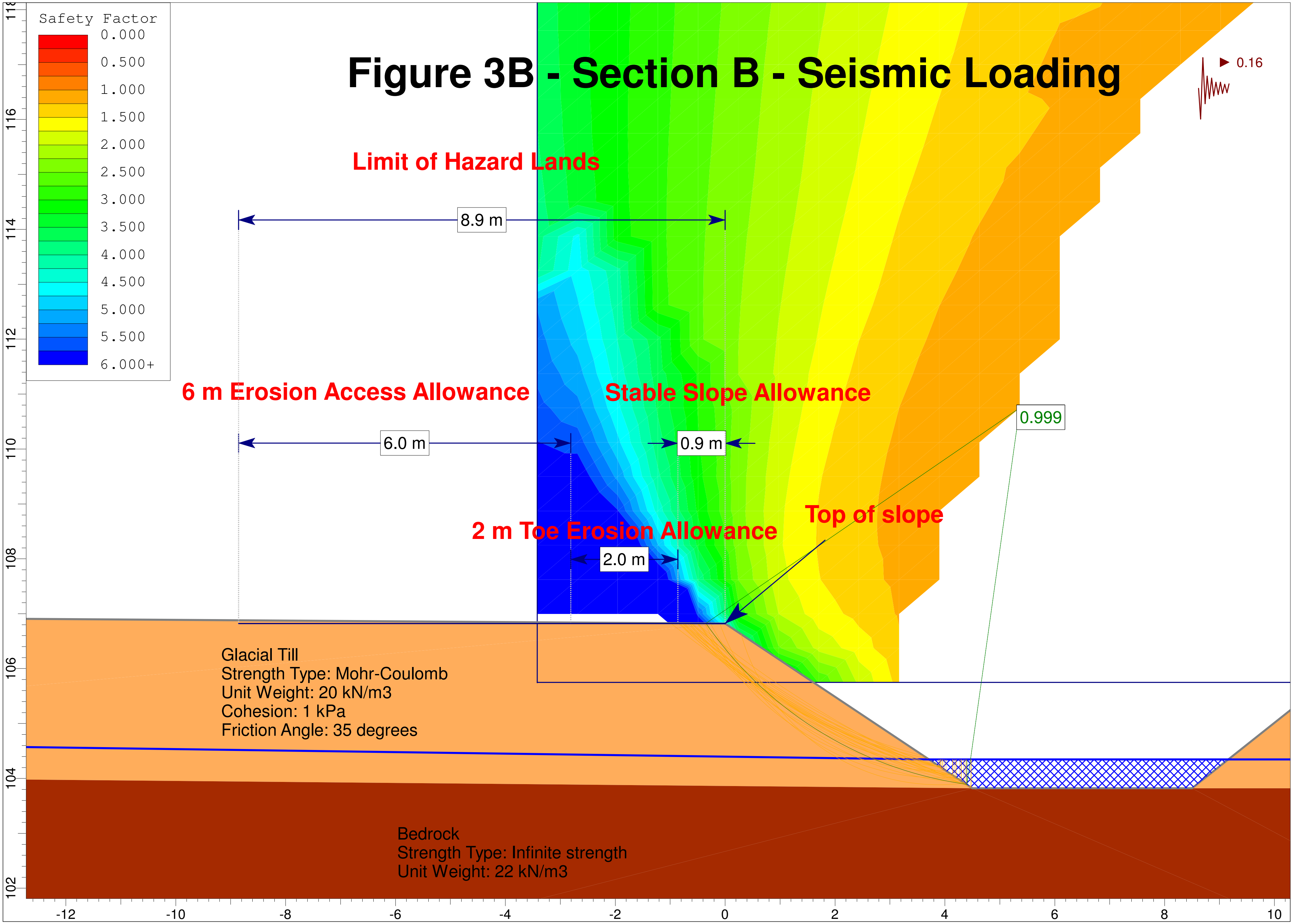


Figure 3A - Section B - Static Conditions



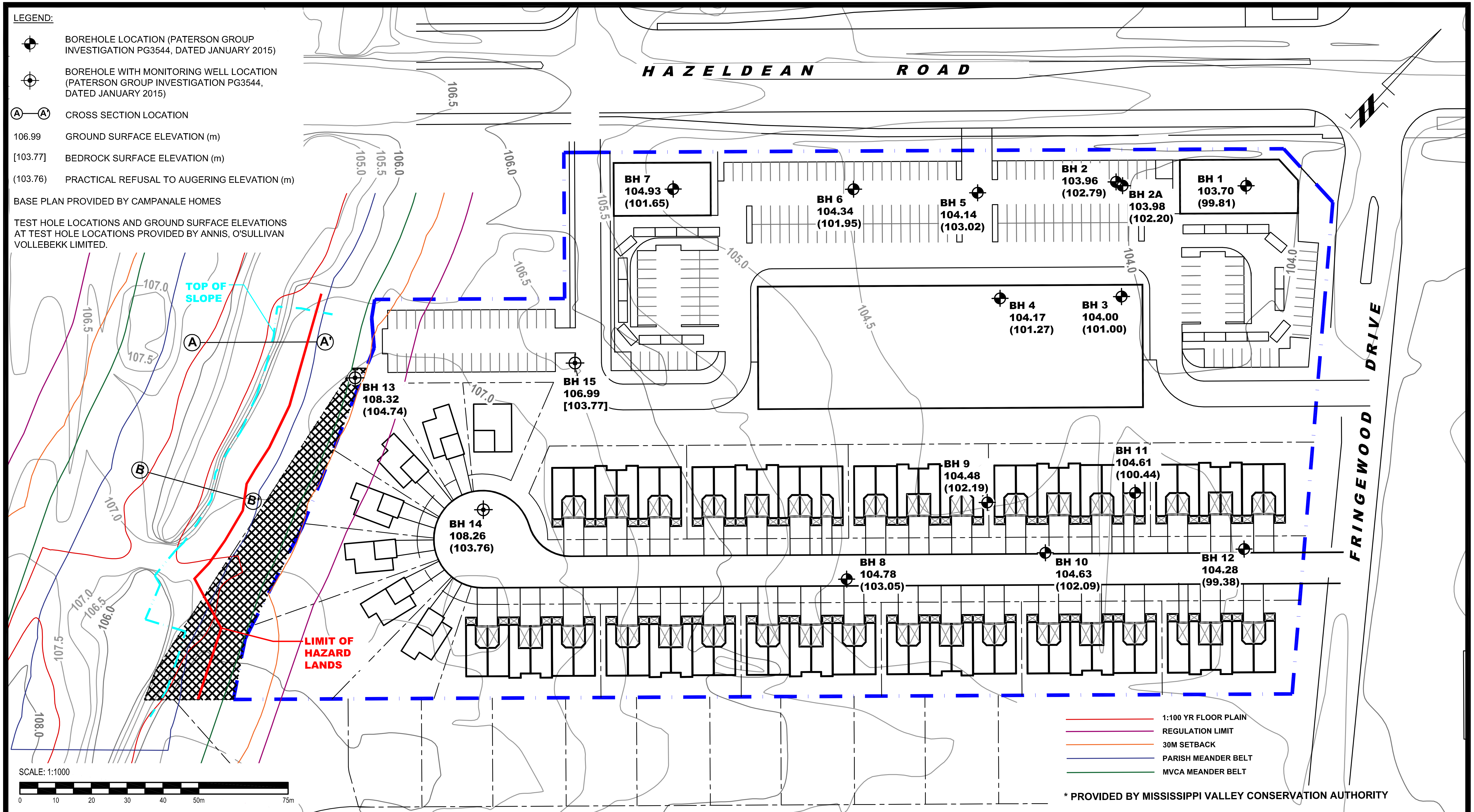
# Figure 3B - Section B - Seismic Loading

LEGEND:

| | |
|---|---|
| ⌖ | BOREHOLE LOCATION (PATERSON GROUP INVESTIGATION PG3544, DATED JANUARY 2015) |
| ⌖ | BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP INVESTIGATION PG3544, DATED JANUARY 2015) |
| Ⓐ—Ⓐ' | CROSS SECTION LOCATION |
| 106.99 | GROUND SURFACE ELEVATION (m) |
| [103.77] | BEDROCK SURFACE ELEVATION (m) |
| (103.76) | PRACTICAL REFUSAL TO AUGERING ELEVATION (m) |

BASE PLAN PROVIDED BY CAMPANALE HOMES

TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT TEST HOLE LOCATIONS PROVIDED BY ANNIS, O'SULLIVAN VOLLEBEKK LIMITED.



| | |
|---|---|
| 🟥 | 1:100 YR FLOOR PLAIN |
| 🟪 | REGULATION LIMIT |
| 🟧 | 30M SETBACK |
| 🟦 | PARISH MEANDER BELT |
| 🟩 | MVCA MEANDER BELT |

* PROVIDED BY MISSISSIPPI VALLEY CONSERVATION AUTHORITY

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**consulting engineers**

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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 0 | | | |

CAMPANALE HOMES

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
5 ORCHARD DRIVE

OTTAWA, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 02/2018
Drawn by: RCG
Report No.: PG4428-1
Checked by: SD
Dwg. No.: PG4428-1
Approved by: DJG
Revision No.: 0

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