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Building Science

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Geotechnical Investigation

Proposed Residential Development
Kanata West - Block 29 - Ottawa

Prepared For

Richcraft Group of Companies

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Report: PG5398-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Richcraft Group of Companies to conduct a geotechnical investigation for Block 29 of the proposed Kanata West development to be located along Maple Grove 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 test holes.

- ❑ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

It is understood that the proposed development will consist of 4 townhouse blocks, each with one basement level. The proposed townhouse blocks will be surrounded by asphalt paved access lanes and parking areas with landscaped margins. It is also understood that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current geotechnical investigation was carried out on June 24, 2020 and consisted of advancing 3 boreholes (BH 1 to BH 3) to a maximum depth of 6.7 m below existing ground surface. One borehole from a previous investigation (BH 7) was also located within the boundaries of the subject site. The test hole locations were determined in the field by Paterson personnel taking into consideration site features and underground services. The test hole locations are presented on Drawing PG5398-1 - Test Hole Location Plan included in Appendix 2.

The test holes 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 drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the current and previous investigations were recovered using a 50 mm diameter split-spoon sampler or 73 mm diameter thin walled Shelby tubes in combination with a piston sampler. Auger cutting samples were recovered from surficial soils. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. The Shelby tubes were sealed at both ends. All samples were transported to our laboratory. The depths at which the auger, split-spoon and Shelby tube samples were recovered from the test holes are shown as AU, SS and TW, respectively, on the applicable Soil Profile and Test Data sheets 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 at regular depth intervals in cohesive soils.

Overburden thickness was evaluated during the course of the current and previous investigations by dynamic cone penetration testing (DCPT) at BH 2 and BH 7. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment. Due to the low resistance exerted by the silty clay in some boreholes, the cone was pushed using the hydraulic head of the drill rig until resistance to penetration was encountered. The hammer was then used to further advance the cone to practical refusal.

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

Sample Storage

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

3.2 Field Survey

The test holes from the current investigation were located and surveyed in the field by Paterson personnel. The locations and ground surface elevations for the current investigation were determined using a hand held GPS unit and are referenced to a geodetic datum. Test hole BH 7, from the previous investigation, was located and surveyed by Annis, Vollebekk and O'Sullivan, and is understood to be referenced to a geodetic datum.

The locations of the test holes and the ground surface elevation at each test hole location are presented on Drawing PG5398-1 - Test Hole Location Plan included in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the test holes were examined in our laboratory to review the results of the field logging. From the 3 current test holes, 13 split spoon samples were submitted for moisture content testing. Among these samples, 3 samples were submitted for Atterberg Limits testing, and 1 sample was submitted for grain size distribution testing.

One (1) soil sample from the previous borehole BH 7 was also submitted for unidimensional consolidation testing. This is discussed further in Section 5.3.

The results of the Atterberg Limits testing, grain size distribution testing, and unidimensional consolidation testing are presented in Appendix 1 and are further discussed in Sections 4 and 5.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site, Block 29, is currently vacant and grass covered across the majority of the site. The site, which has an approximate triangular shape, is bordered by Maple Grove Road to the north, Poole Creek to the northwest, and vacant undeveloped lands to the south and west. The existing ground surface across the site is generally level at approximate geodetic elevation 97 m.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the borehole locations consists of an approximate 0.2 to 0.5 m thickness of fill underlying the existing ground surface. The fill was generally observed to consist of a brown silty sand with some clay and crushed stone. An approximate 140 mm thickness of topsoil was also encountered underlying the fill at BH 1.

On the northwest end of the site, within BH 1 and BH 7, a layer of loose to compact, brown silty sand to sandy silt was encountered underlying the fill and/or topsoil, extending to an approximate depth of 1.5 m below the existing ground surface.

A silty clay deposit was encountered underlying the fill, topsoil, and/or silty sand to sandy silt. The silty clay deposit had a very stiff to stiff, brown silty clay crust in the upper 3 to 4 m, becoming a stiff to firm, grey silty clay with depth. Boreholes BH 1 through BH 3 were terminated in the silty clay deposit at approximate depths of 5.9 to 6.7 m below the existing ground surface.

A glacial till deposit was encountered in BH 7 underlying the silty clay at an approximate depth of 7 m. The glacial till was generally observed to consist of a compact, grey silty sand with gravel. Borehole BH 7 was terminated in the glacial till deposit at an approximate depth of 8.2 m below the existing ground surface.

Practical refusal to the DCPT was encountered at depths ranging from 9.1 m in BH 7, at the northwest end of the site, to 16 m at BH 2, located at the southeast end of the site.

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.

Laboratory Testing

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay samples at 3 selected locations throughout the subject site.

The results of the Atterberg limits tests are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1. The tested silty clay samples classify as inorganic clays of low plasticity (CL) in accordance with the Unified Soil Classification System.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 1	2.0	33	20	13	CL
BH 2	2.6	42	19	23	CL
BH 3	2.6	39	20	19	CL

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

The results of the shrinkage limit test indicate a shrinkage limit of 20% and a shrinkage ratio of 1.77.

Grain size distribution (sieve and hydrometer analysis) was also completed on one selected soil sample. The result of the grain size analysis is summarized in Table 2 and presented on the Grain Size Distribution Results sheet in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt & Clay (%)
BH 2	SS 4	0.0	13.9	86.1

Bedrock

Based on available geological mapping, the bedrock in this area consists of interbedded limestone and shale of the Verulam Formation with an overburden drift thickness of approximately 10 to 15 m depth.

4.3 Groundwater

Based on groundwater level measurements, field observations during excavation, knowledge of the groundwater within the local area of the subject site, and the recovered soil samples' moisture levels, consistency and colouring, the long-term groundwater table can be expected between a 3 to 4 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed buildings be founded on conventional shallow foundations bearing on the undisturbed, stiff to firm silty clay, compact silty sand to sandy silt, or on engineered fill which is placed and compacted directly over the undisturbed stiff to firm silty clay or compact silty sand to sandy silt.

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

Topsoil, asphalt, and deleterious fill, such as material containing a high content of organic materials, should be stripped from under the proposed building footprints and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed buildings should consist 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 building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Using continuously applied loads, footings for the proposed buildings can be designed with the following bearing resistance values presented in Table 3.

Table 3 - Bearing Resistance Values

Undisturbed Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Compact Silty Sand/Sandy Silt	100	150
Stiff Silty Clay	120	180
Firm Silty Clay	80	120
Engineered Fill	100	150

Note: Strip footings, up to 2 m wide, and pad footings, up to 3 m wide, placed over an undisturbed, silty clay bearing surface can be designed using the abovenoted bearing resistance values.

If the silty sand subgrade is observed to be in a loose state of compactness, the material should be proof rolled using suitable vibratory equipment making several passes under dry conditions and above freezing temperatures and approved by Paterson at the time of construction.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

The bearing resistance value at SLS given for footings will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a silty clay, silty sand to sandy silt, or engineered fill bearing surface 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.

Settlement/Grade Raise

During the previous investigations, 1 consolidation test was completed within the boundaries of the subject site. The results of the consolidation test from the previous investigation are presented in Table 4 and in Appendix 1.

The value for p'_c is the preconsolidation pressure and p'_o is the effective overburden pressure of the test sample. The difference between these values is the available preconsolidation. The increase in stress on the soil due to the cumulative effects of the fill surcharge, the footing pressures, the slab loadings and the lowering of the groundwater should not exceed the available preconsolidation if unacceptable settlements are to be avoided.

The values for C_{cr} and C_c are the recompression and compression indices, respectively. These soil parameters are a measure of the compressibility due to stress increases below and above the preconsolidation pressures. The higher values for the C_c, as compared to the C_{cr}, illustrate the increased settlement potential above, as compared to below, the preconsolidation pressure.

Table 4 - Summary of Consolidation Test Results

Borehole	Sample	Elevation (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c
BH7	TW 6	91.56	107	79	0.025	0.742

The values of p'_c, p'_o, C_{cr} and C_c are determined using standard engineering testing procedures and are estimates only. Natural variations within the soil deposit will affect the results. The p'_o parameter is directly influenced by the groundwater level. Groundwater levels were measured during the site investigation. Groundwater levels vary seasonally which has an impact on the available preconsolidation. Lowering the groundwater level increases the p'_o and therefore reduces the available preconsolidation. Unacceptable settlements could be induced by a significant lowering of the groundwater level. The p'_o values for the consolidation tests during the investigation are based on the long term groundwater level being at 0.5 m below the existing groundwater table. The groundwater level is based on the colour and undrained shear strength profile of the silty clay.

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

The potential post construction total and differential settlements are dependent on the position of the long term groundwater level when buildings are situated over deposits of compressible silty clay. Efforts can be made to reduce the impacts of the proposed development on the long term groundwater level by placing clay dykes in the service trenches, reducing the sizes of paved areas, leaving green spaces to allow for groundwater recharge or limiting planting of trees to areas away from the buildings. However, it is not economically possible to control the groundwater level.

Buildings on silty clay deposits increases the likelihood of movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking compared to unreinforced foundations.

Based on the test hole information and consolidation testing results, a permissible grade raise restriction of **1.5 m** is recommended for grading within the subject site.

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the existing fill or native soil subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for basement slab construction. Where the subgrade consists of existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

It is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its MPMDD.

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

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45\text{-}a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where K_o = 0.5 for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

Where required at the subject site, the recommended pavement structures for car only parking areas and access lanes are shown in Tables 5 and 6.

Table 5 - 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 6 - Recommended Pavement Structure - Access Lanes	
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 - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

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

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

Pavement Structure Drainage

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

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

Sub-slab Drainage

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

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as 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.

6.2 Protection Against Frost Action

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

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

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. However, it is expected 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 excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for sewer and water pipes should consist of a minimum of 150 mm of OPSS Granular A material. Where the bedding is located within the firm to stiff grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in a maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay and existing groundwater level, it is anticipated that groundwater infiltration into the excavations should be low to medium and controllable using open sumps. A perched groundwater condition may be encountered within the silty sand to sandy silt deposit which may produce significant temporary groundwater infiltration levels. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

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.

Long-term Groundwater Control

Our recommendations for the long-term groundwater control for proposed construction are presented in Subsection 6.1. Any groundwater encountered along the proposed structure's perimeter or sub-slab drainage system will be directed to the proposed structure's sump pit. It is expected that groundwater flow will be low (i.e.- less than 10,000 L/day) with peak periods noted after rain events.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

The results on analytical testing show that the sulphate content is less than 0.1%. The results are indicative that Type 10 Portland 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 in indicative of a moderate to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Setbacks

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Grain size distribution testing was also completed on a selected soil sample from BH 2. The above-noted soil samples were recovered from elevations below the anticipated design underside of footing elevation and 3.5 m depth below anticipated finished grade. The results of our testing are presented in Subsection 4.2 and in Appendix 1.

Based on the results of our review, a low to medium sensitivity clay soil is present within the proposed development.

Low/Medium Sensitivity Clay Soils

Based on our Atterberg Limits test results, the modified plasticity limit does not exceed 40% at the subject site. The following tree planting setbacks are recommended for the low to medium sensitivity area. Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to **4.5 m** for small (mature tree height up to 7.5 m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the conditions noted below are met:

❏ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

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

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

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

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

Swimming Pools

The in-situ soils are considered to be acceptable for in-ground swimming pools. Above ground swimming pools must be placed at least 5 m away from the residence foundation and neighbouring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Aboveground Hot Tubs

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, hot tub construction is considered routine, and could be constructed in accordance with the manufacturer’s specifications.

Installation of Decks or Additions

Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

6.9 Limit of Hazard Lands

Poole Creek

A section of Poole Creek is located within the west portion of the site. The slope condition was reviewed by Paterson field personnel as part of the geotechnical investigation. One (1) slope cross-section (Section B) was studied as the worst case scenario, where Poole Creek has meandered in close proximity (less than 1 m) from the toe of the upper slope or valley corridor. In addition, a second slope cross-section (Section C) was also analyzed at Block 29. The cross section locations are presented on Drawing PG5398-2 - Limit of Hazard Lands in Appendix 2. The subject section of Poole Creek is approximately 2 to 3 m wide, approximately 0.3 to 0.6 m depth, and meanders across the valley floor.

Poole Creek is observed within a 15 to 25 m wide flood plain. A 3 to 4 m high stable slope confines the flood plain. The upper slope is observed to be well vegetated and stable with little to no signs of active erosion. Signs of erosion were noted along the subject section of Poole Creek where the watercourse has meandered in close proximity to the toe of the corridor wall. The majority of the subject slope was shaped between a 2.2H:1V to 3.5H:1V slope.

A slope stability analysis was carried out to determine the required stable slope allowance setback from the top of slope based on a factor of safety of 1.5. A toe erosion and 6 m erosion access allowances were also considered in the determination of Limit of Hazard Lands and are discussed on the following pages. The proposed Limit of Hazard Lands, including the stable slope allowance, where required, toe erosion allowance, 6 m erosion access allowance, and top of slope are shown on Drawing PG5398-2 - Limit of Hazard Lands 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 0.16 g 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 analyzed taking into account a groundwater level at ground surface, which represents a worse-case scenario that can be reasonably expected to occur in cohesive soils. The stability analysis assumes full saturation of the soil with groundwater flow parallel to the slope face. 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 B and C are presented in Figures 2 and 4 in Appendix 2. Section B requires a stable slope allowance due to the slope stability factor of safety being less than 1.5. It should be noted that the cross-section was analyzed as the worst case scenario for the subject slope. The remainder of the slope reviewed along the subject section of Poole Creek was noted to be shaped to at least a 3H:1V profile.

Based on the soil conditions observed and slope profile along the subject section of Poole Creek, the remainder of the slope has a slope stability factor of safety of greater than 1.5 and does not require a stable slope allowance.

The results of the analyses including seismic loading are shown in Figures 3 and 5 for the slope sections. The results indicate that the factor of safety for the sections are greater than 1.1 for the sections.

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 valley corridor wall slope was based on the cohesive nature of the soils, the observed current erosional activities and the width and location of the current watercourse. Signs of erosion were noted along the subject section of Poole Creek where the watercourse has meandered in close proximity to the toe of the corridor wall.

It is considered that in areas where the water course has meandered in close proximity (less than 15 m) to the toe of the upper slope, a toe erosion allowance of 5 m and an erosion access allowance of 6 m are required from the top of slope. Where the watercourse is greater than 15 m from the toe of the slope, the toe erosion allowance should be taken from the watercourse edge. The Limit of Hazard Lands, which includes these allowances, is indicated on Drawing PG5398-2 - Limit of Hazard Lands in Appendix 2.

Minimum Setback Requirements of the Official Plan

Minimum setbacks have been established by Council for the Official Plan for rivers, lakes, streams and other surface water features. It should be noted that where a council-approved watershed, sub-watershed or environmental management plan does not exist, the minimum setback will be the greater of the following:

- ❏ Development limits as established by the regulatory flood line
- ❏ Development limits as established by the geotechnical Limit of the Hazard Lands
- ❏ 30 m from normal high water mark of rivers, lakes and streams as determined in consultation with the conservation authority, or

- 15 m from existing top of bank, where there is a defined bank.

However, it should also be noted that where the geotechnical Limit of Hazard Lands line and regulatory flood line are within 15 m of top of slope, the development limits can be established as the geotechnical limit of hazard lands line provided the Conservation Authority approves.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- ❑ A review of the final grading plan should be completed 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 material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

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

A geotechnical investigation of this nature is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Richcraft Group of Companies or their agents is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group

Yolanda Tang

LICENSED PROFESSIONAL ENGINEER
July 14, 2020
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Yolanda Tang, M.Sc.Eng

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Richcraft Group of Companies
- ❑ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

UNIDIMENSIONAL CONSOLIDATION TEST SHEETS

ATTERBERG LIMITS RESULTS

GRAIN SIZE DISTRIBUTION RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Block 29 - Maple Grove Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5398**

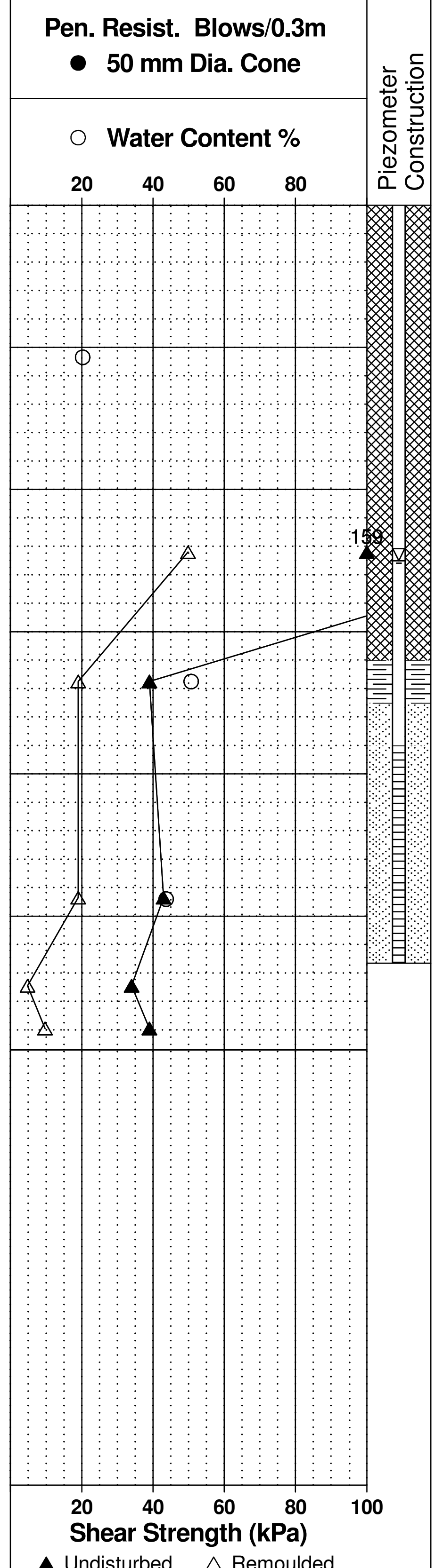
REMARKS

HOLE NO. **BH 1**

BORINGS BY Track-Mount Power Auger

DATE June 24, 2020

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	96.70
FILL: Brown silty sand, some crushed stone (to 0.46)	AU	1				
TOPSOIL (0.46 to 0.60)						
Compact, brown **SANDY SILT,** trace clay to **CLAYEY SILT** (0.60 to 1.52)	SS	2	58	16	1	95.70
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	3	83	2	2	94.70
- firm and grey by 3.0m depth					3	93.70
	SS	4	100	P	4	92.70
(to 5.94)	SS	5	100	P	5	91.70
End of Borehole						
(GWL @ 2.5m depth based on field observations)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Block 29 - Maple Grove Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5398

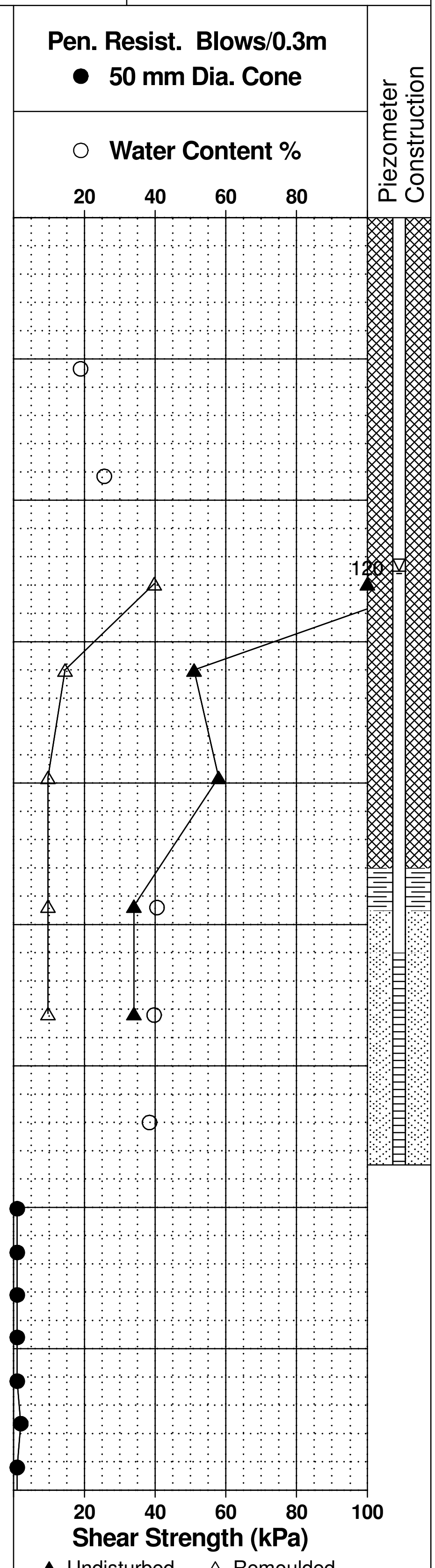
REMARKS

HOLE NO. BH 2

BORINGS BY Track-Mount Power Auger

DATE June 24, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.08
FILL: Brown silty sand, some clay, crushed stone and boulders 0.36		AU	1				
Brown **CLAYEY SILT,** trace sand						1	96.08
		SS	2	58	19		
		SS	3	67	7		
2.20						2	95.08
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	4	100	P		
- stiff to firm and grey by 3.0m depth						3	94.08
						4	93.08
		SS	5	100	P	5	92.08
		SS	6	100	P		
						6	91.08
		SS	7	100	P		
6.70							
Dynamic Cone Penetration Test commenced at 6.70m depth.						7	90.08
						8	89.08
						9	88.08



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Block 29 - Maple Grove Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

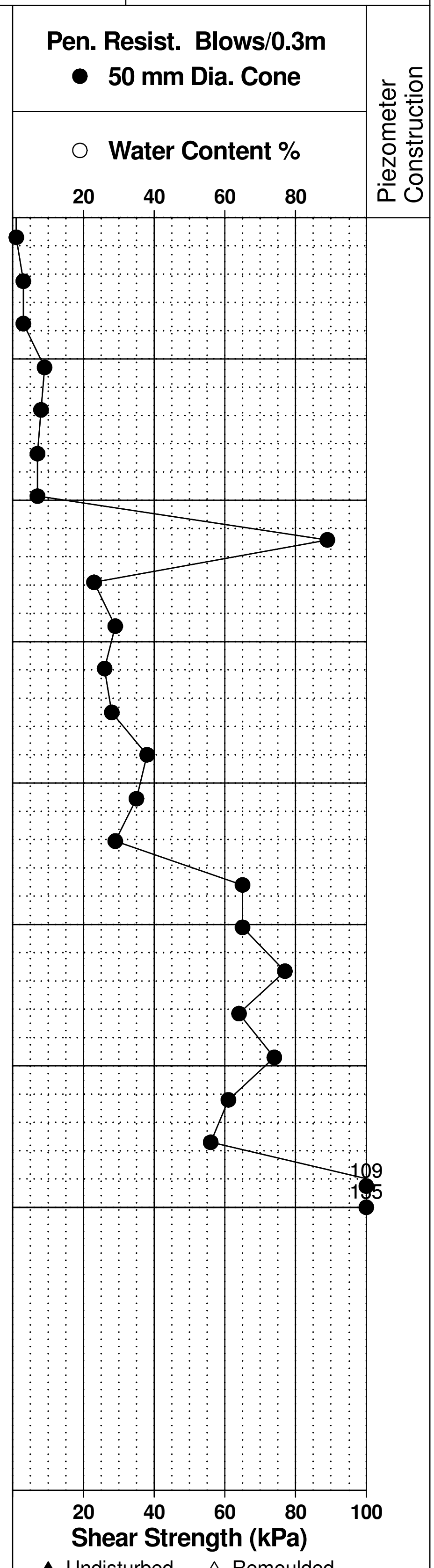
BORINGS BY Track-Mount Power Auger

DATE June 24, 2020

FILE NO. PG5398

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						9	88.08
						10	87.08
						11	86.08
						12	85.08
						13	84.08
						14	83.08
						15	82.08
16.00						16	81.08
End of Borehole Practical DCPT refusal at 16.00m depth (GWL @ 2.5m depth based on field observations)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Block 29 - Maple Grove Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 24, 2020

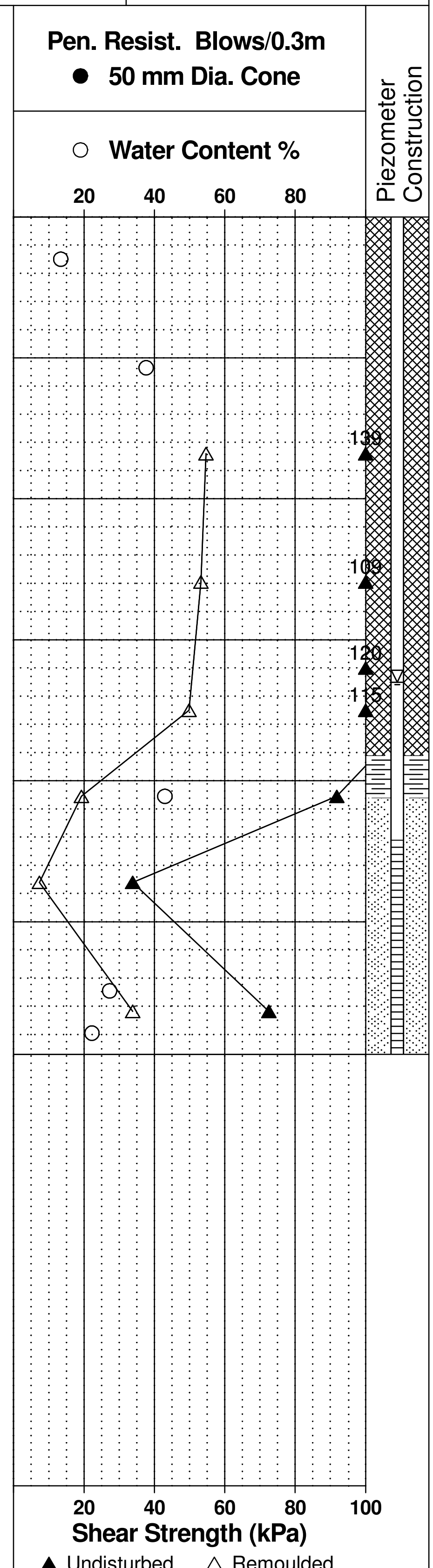
FILE NO. PG5398

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.24
FILL: Brown silty sand, trace crushed stone (0.20)		SS	1	17	6		
		SS	2	75	6	1	96.24
Very stiff to stiff, brown **SILTY CLAY,** trace sand						2	95.24
		SS	3	100	P		
						3	94.24
- firm to stiff and grey by 3.8m depth						4	93.24
		SS	4	100			
						5	92.24
		SS	5	50	3		
(5.94)							

End of Borehole

(GWL @ 3.3m depth based on field observations)



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Kanata West Subdivision
Ottawa (Kanata), Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 75 Power Auger

DATE Aug 1, 03

FILE NO. G9012

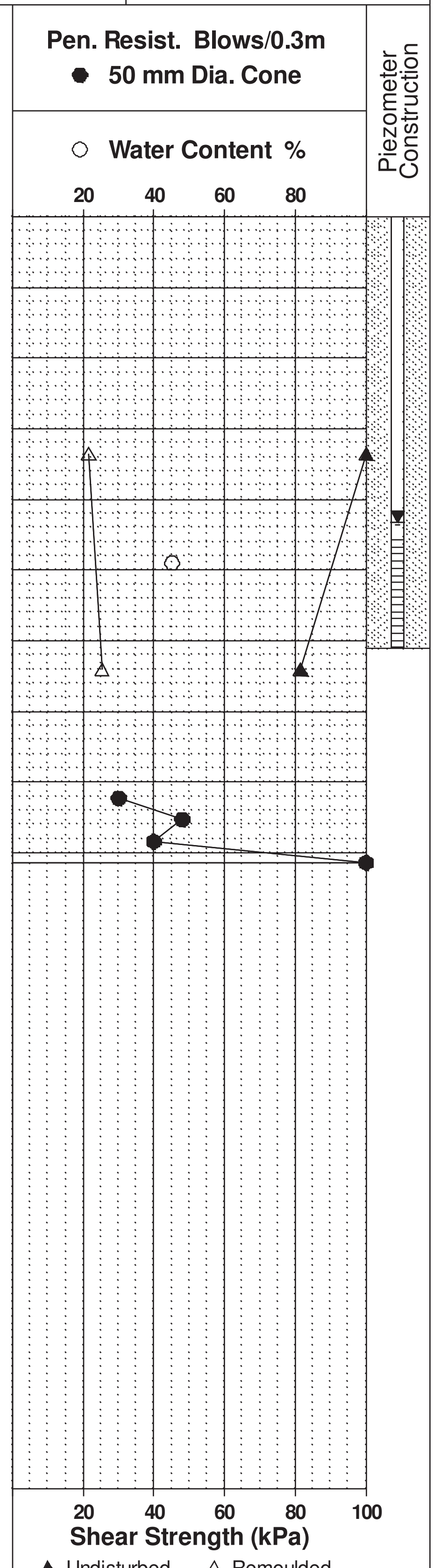
HOLE NO. BH 7

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	96.46
TOPSOIL	0.23						
Loose, brown **SILTY** fine **SAND**	1.45	SS	15	29	8	1	95.46
Very stiff to stiff, brown **SILTY CLAY**		SS	16	96	5	2	94.46
						3	93.46
- grey by 4.0m depth						4	92.46
		TW	6			5	91.46
	7.01					6	90.46
GLACIAL TILL: Compact, grey silty fine sand with gravel	8.23	SS	17	75	18	7	89.46
Dynamic Cone Penetration Test commenced @ 8.23m depth						8	88.46
Inferred **GLACIAL TILL**	9.14					9	87.46

End of Borehole

DCPT refusal @ 9.14m depth

(GWL @ 4.33m-Aug. 18/03)



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

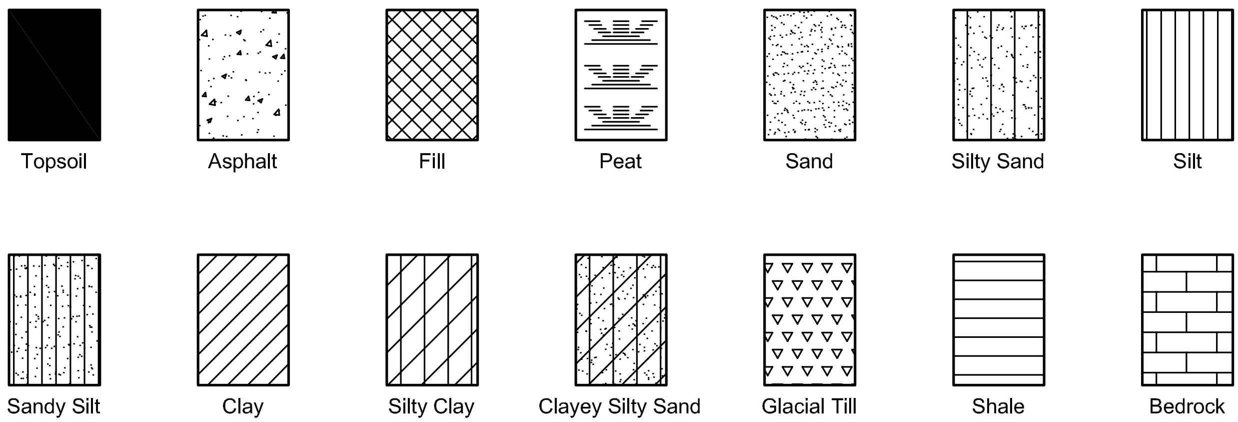
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

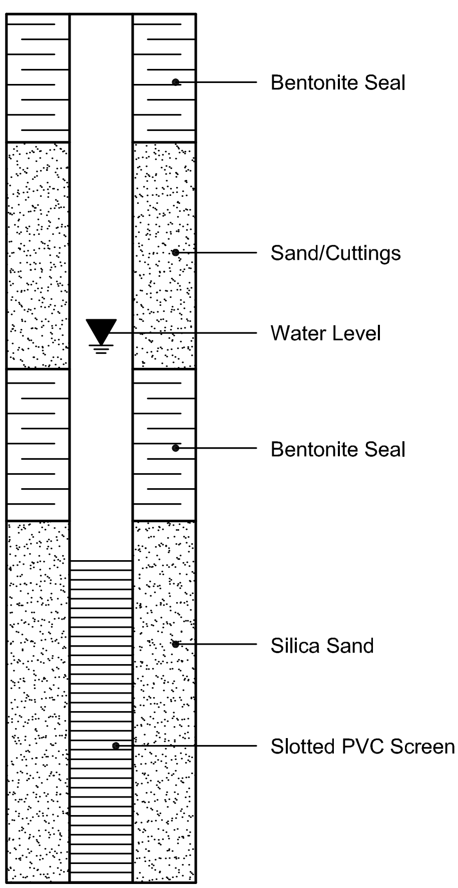
SYMBOLS AND TERMS (continued)

STRATA PLOT

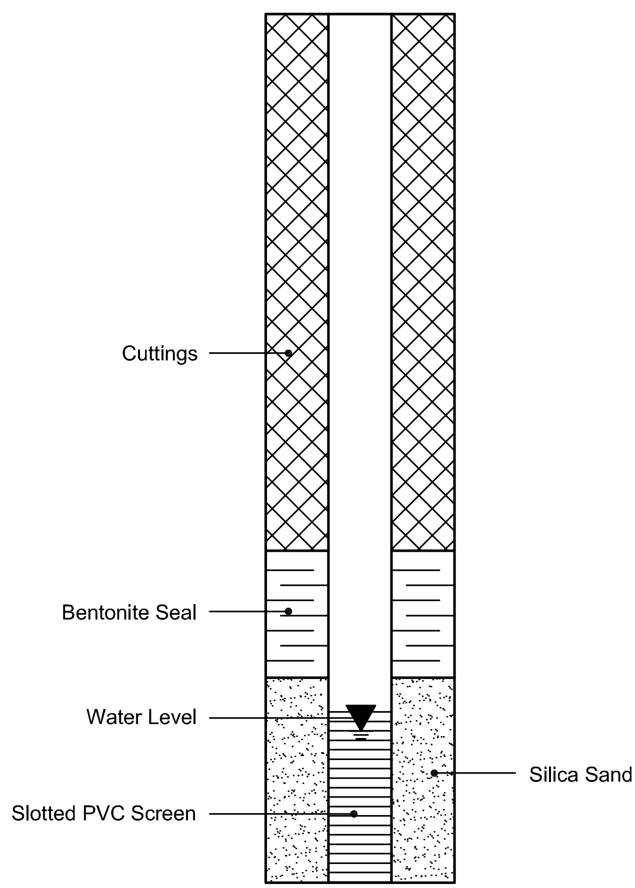


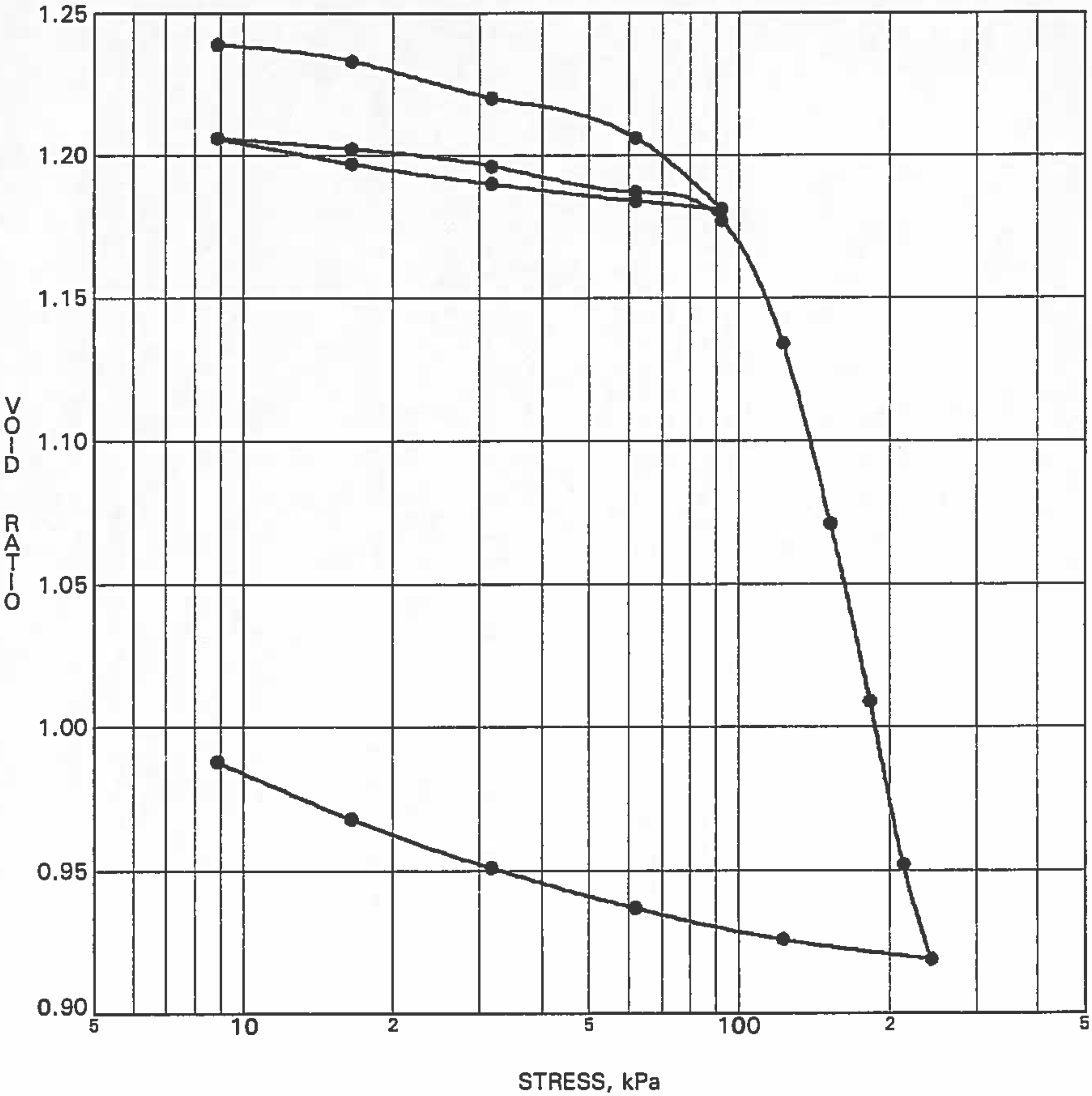
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



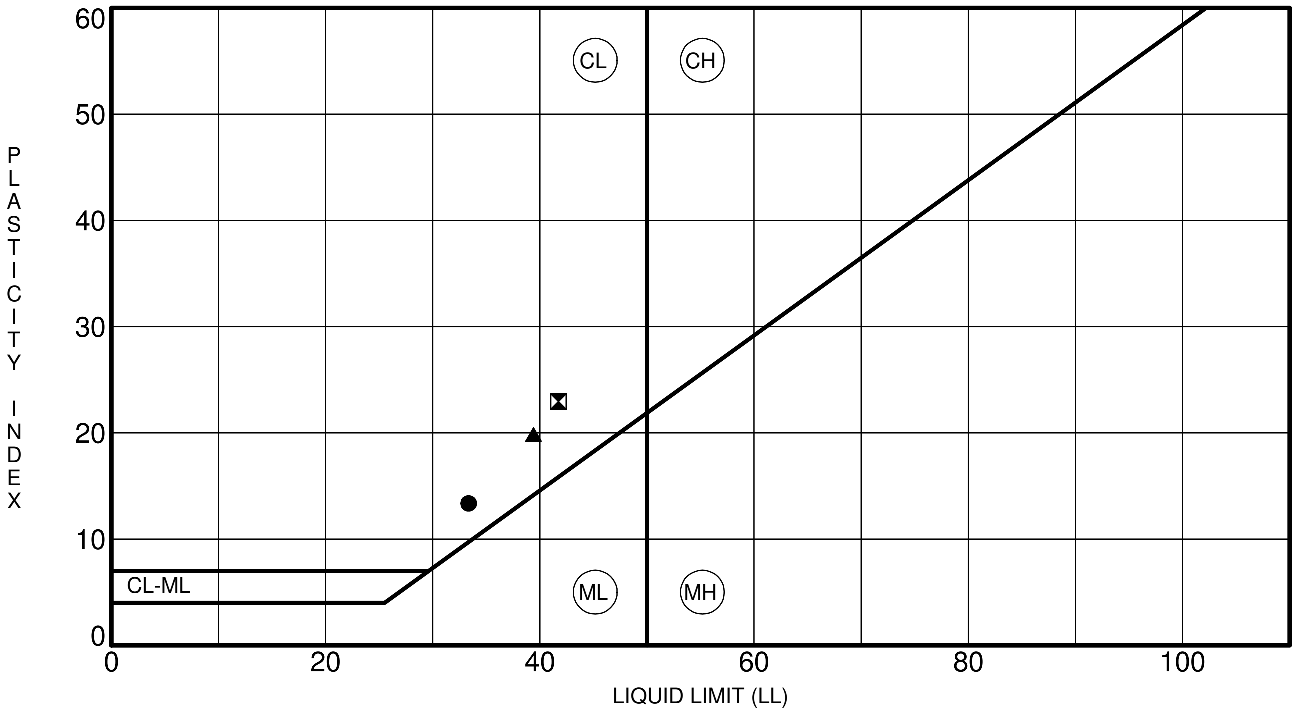


CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 7	p'_o	79 kPa	Ccr	0.025
Sample No.	TW 6	p'_c	107 kPa	Cc	0.742
Sample Depth	4.90 m	OC Ratio	1.4	Wo	45.1 %
Sample Elev.	91.56 m	Void Ratio	1.240	Unit Wt.	17.5 kN/m^3

CLIENT	Richcraft Homes	FILE NO.	G9012
PROJECT	Geotechnical Investigation - Proposed Kanata West Subdivision	DATE	19/08/03

CONSOLIDATION TEST

JOHN D. PATERSON & ASSOCIATES LTD.
Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1**	**SS 3**	**33**	**20**	**13**		**CL - Inorganic clays of low plasticity**
⊠	**BH 2**	**SS 4**	**42**	**19**	**23**		**CL - Inorganic clays of low plasticity**
▲	**BH 3**	**SS 3**	**39**	**20**	**20**		**CL - Inorganic clays of low plasticity**

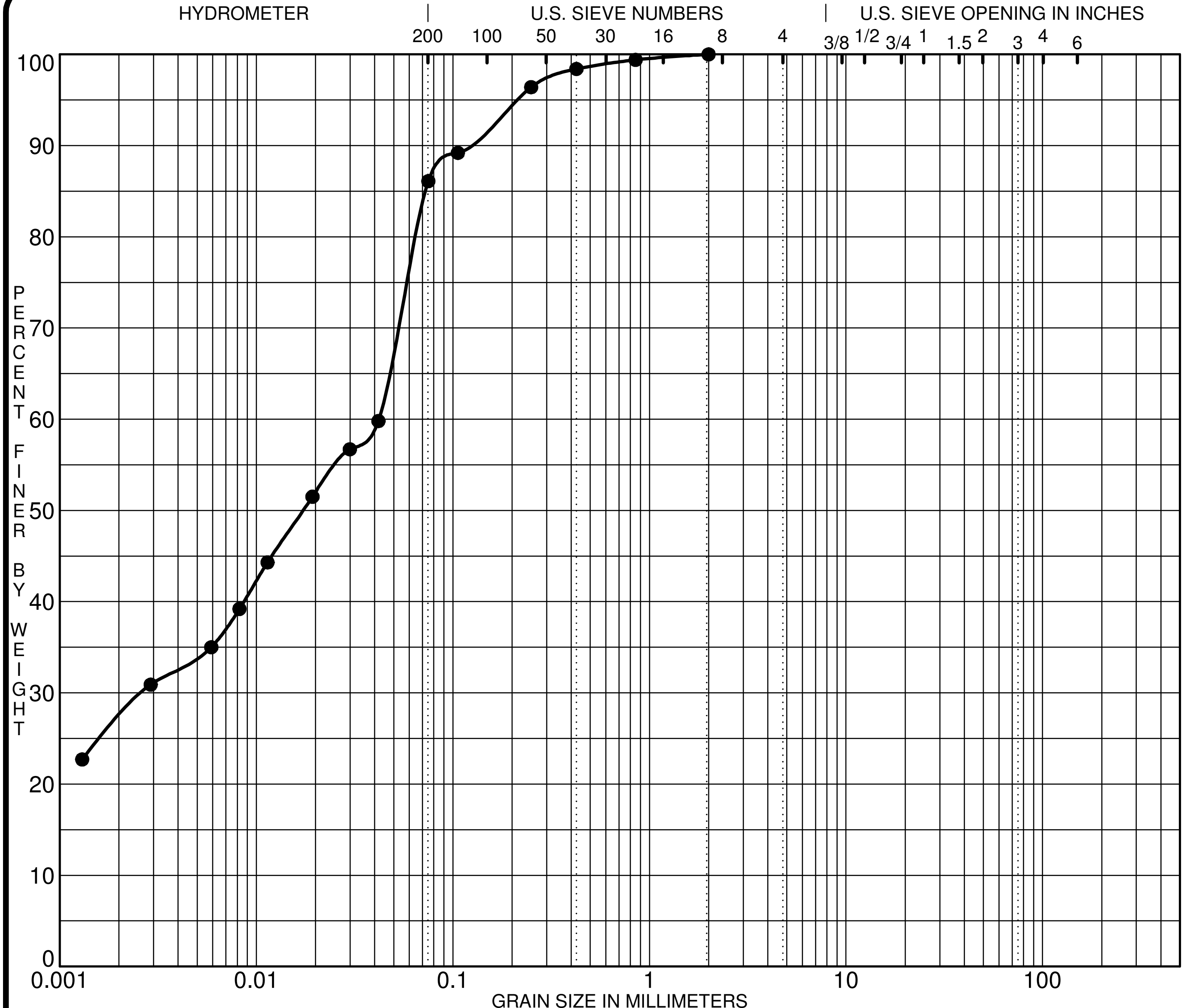
CLIENT **Richcraft Homes** FILE NO. **PG5398**

PROJECT **Geotechnical Investigation - Kanata West Block 29 - Maple Grove Road** DATE **24 Jun 20**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 2**	**SS 4**	**CL - Inorganic clays of low plasticity**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 2**	**SS 4**	**2.00**	**0.04**	**0.003**		**0.0**	**13.9**	**86.1**	

CLIENT **Richcraft Homes**

FILE NO. **PG5398**

PROJECT **Geotechnical Investigation - Kanata West Block 29 - Maple Grove Road**

DATE **24 Jun 20**

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29945

Report Date: 02-Jul-2020

Order Date: 25-Jun-2020

Project Description: PG5398

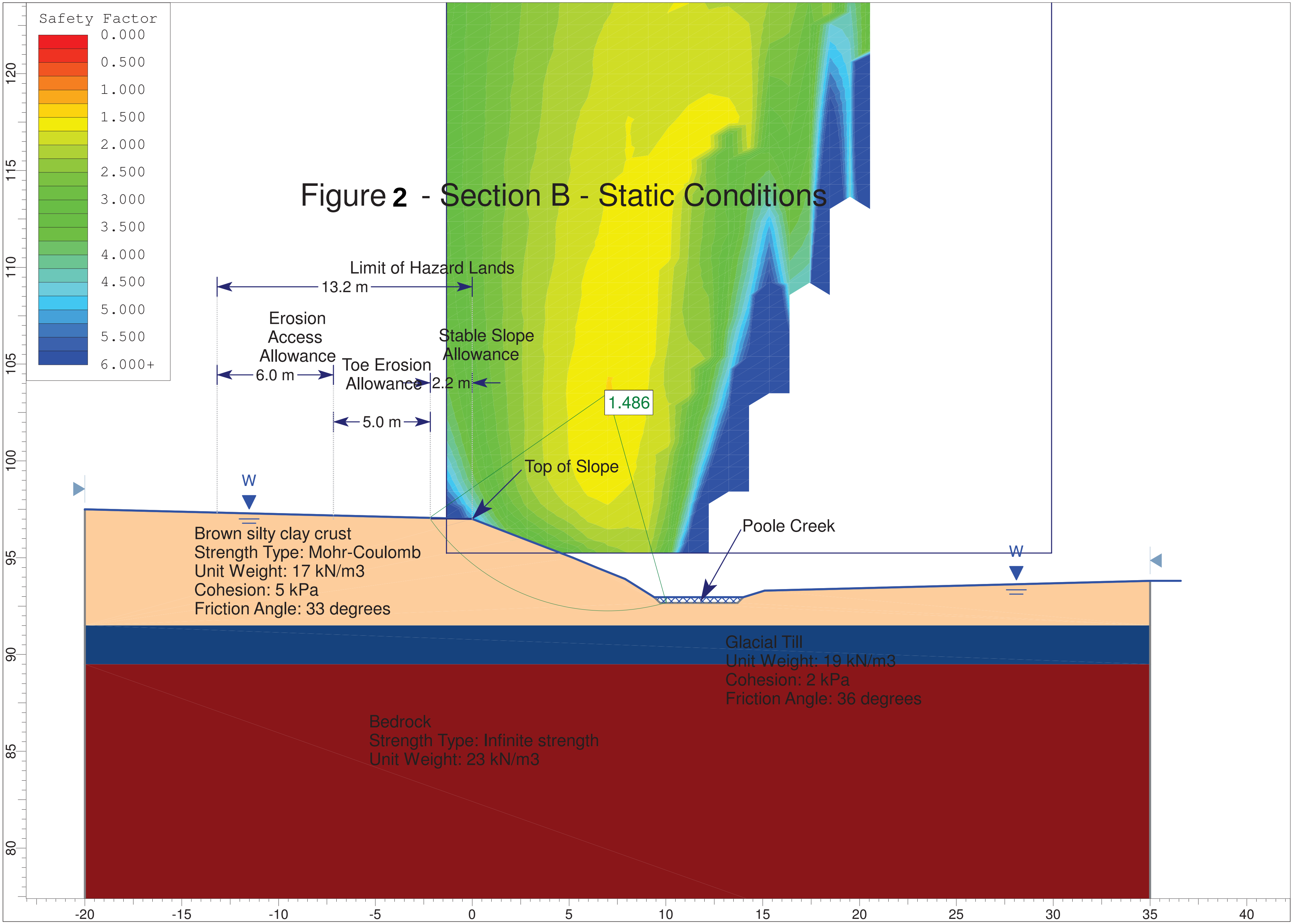
	Client ID:	BH3-SS2	-	-	-
	Sample Date:	24-Jun-20 12:00	-	-	-
	Sample ID:	2026396-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	76.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.27	-	-	-
Resistivity	0.10 Ohm.m	135	-	-	-
Anions					
Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	6	-	-	-

APPENDIX 2

FIGURE 1

KEY PLAN

Figure 2 - Section B - Static Conditions



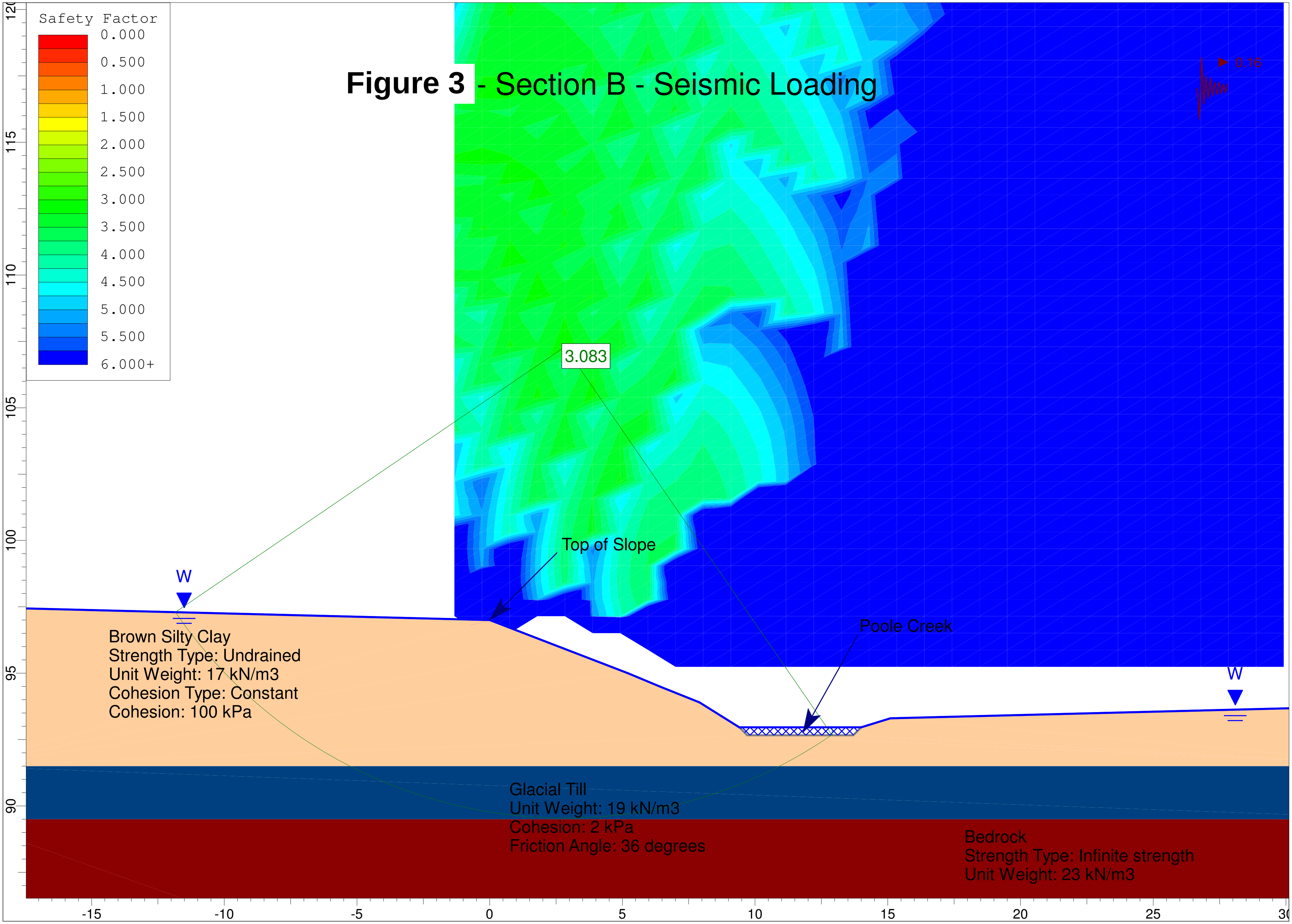


Figure 3 - Section B - Seismic Loading
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
3.083
0.16
W
W
Top of Slope
Poole Creek
Brown Silty Clay
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 100 kPa
Glacial Till
Unit Weight: 19 kN/m3
Cohesion: 2 kPa
Friction Angle: 36 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 23 kN/m3
120
115
110
105
100
95
90
-15
-10
-5
0
5
10
15
20
25
30

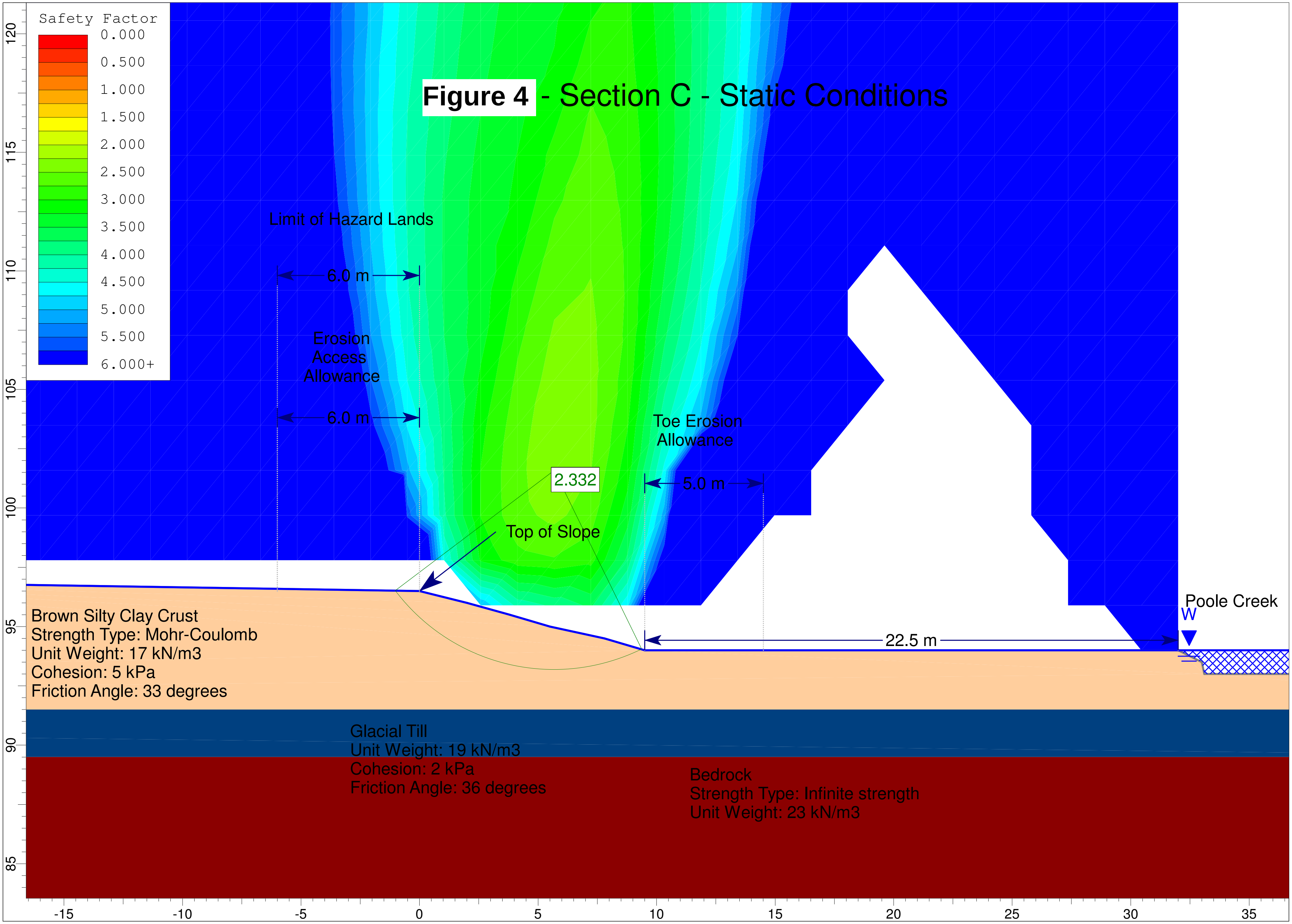


Figure 4 - Section C - Static Conditions
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Limit of Hazard Lands
6.0 m
Erosion Access Allowance
6.0 m
Toe Erosion Allowance
5.0 m
2.332
Top of Slope
Poole Creek
W
22.5 m
Brown Silty Clay Crust
Strength Type: Mohr-Coulomb
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Glacial Till
Unit Weight: 19 kN/m3
Cohesion: 2 kPa
Friction Angle: 36 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 23 kN/m3
120
115
110
105
100
95
90
85
-15
-10
-5
0
5
10
15
20
25
30
35

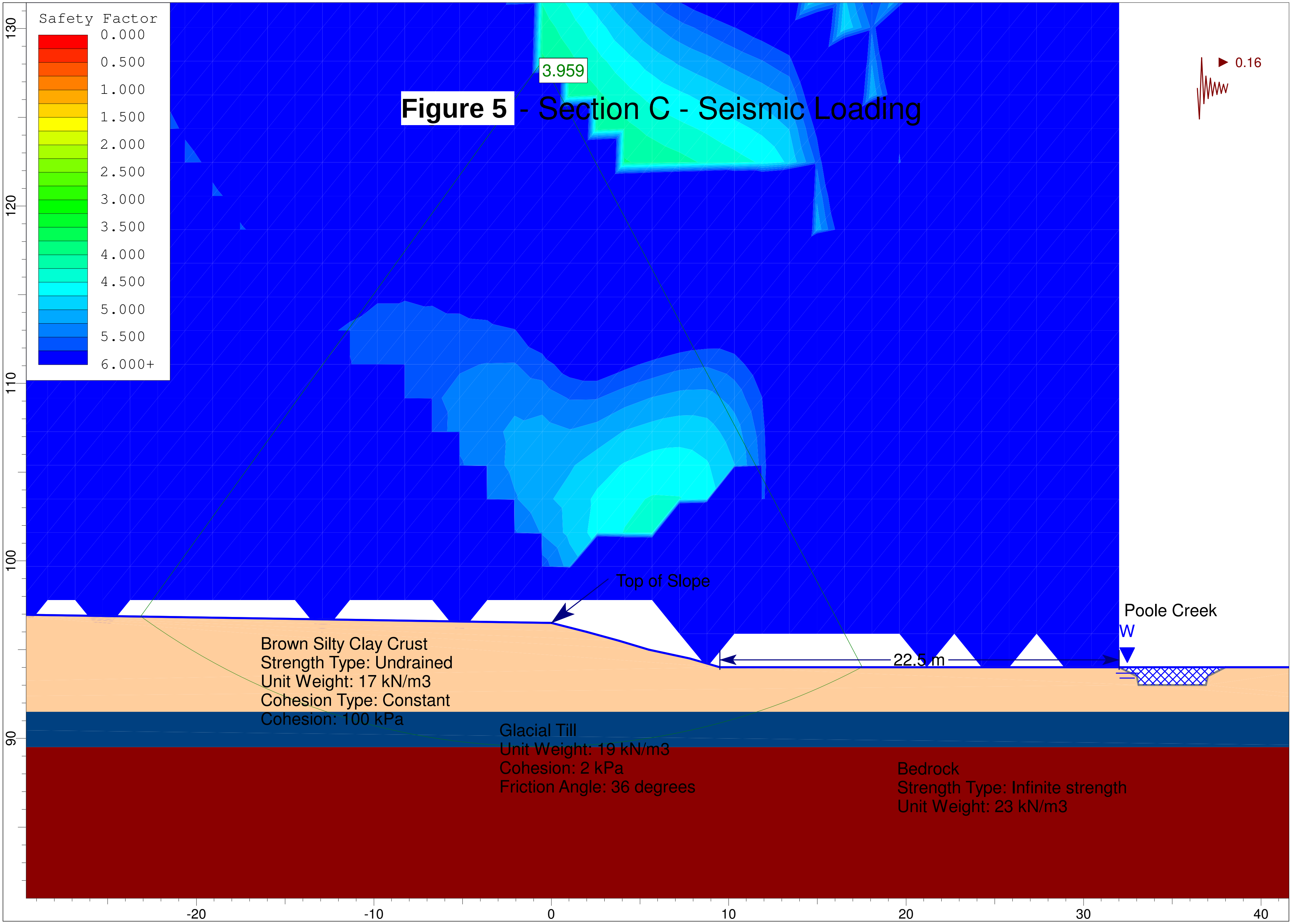
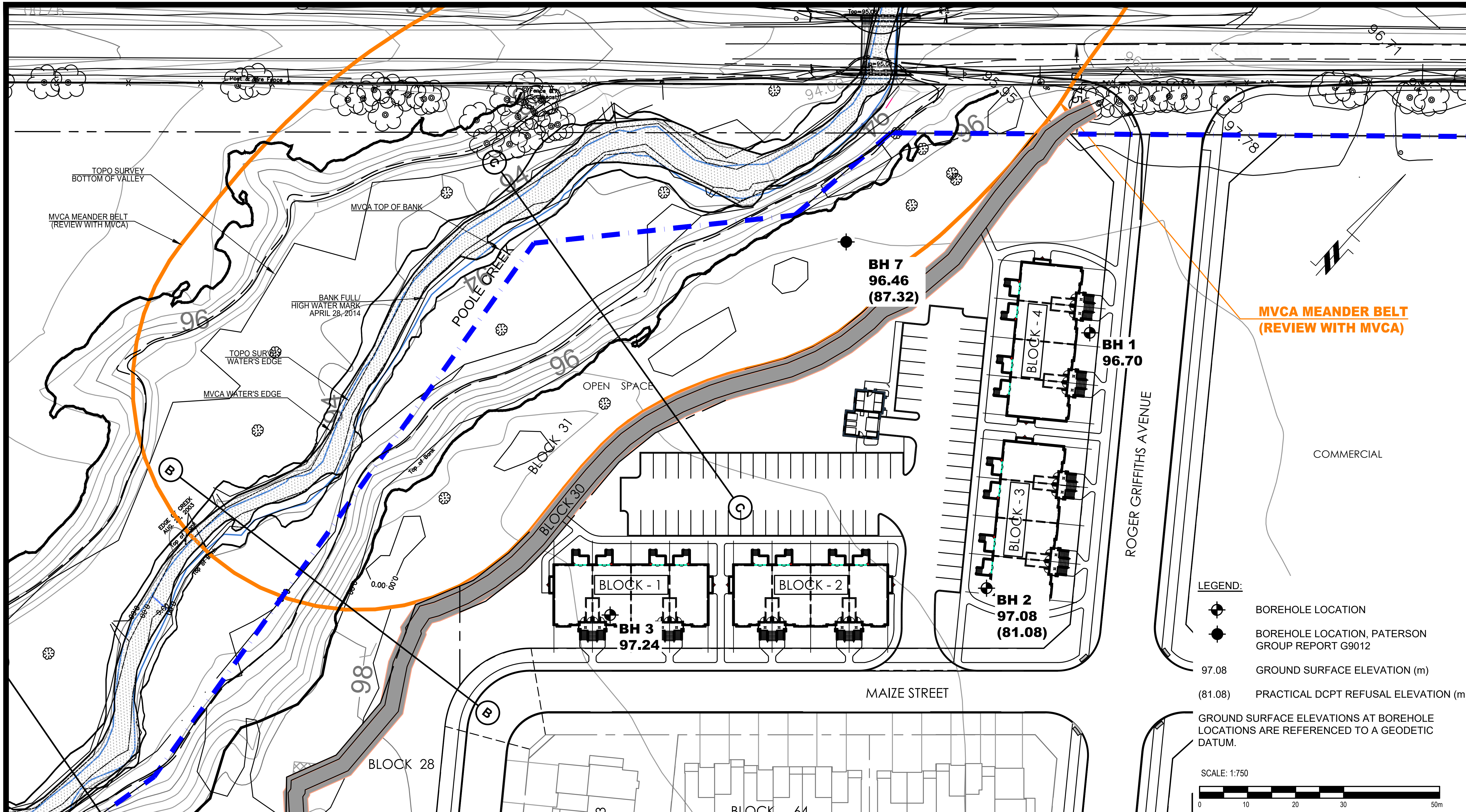


Figure 5 - Section C - Seismic Loading
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
3.959
0.16
Top of Slope
Poole Creek
W
22.5 m
Brown Silty Clay Crust
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 100 kPa
Glacial Till
Unit Weight: 19 kN/m3
Cohesion: 2 kPa
Friction Angle: 36 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 23 kN/m3
130
120
110
100
90
-20
-10
0
10
20
30
40



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RICHCRAFT HOMES

GEOTECHNICAL INVESTIGATION
KANATA WEST - BLOCK 29 - MAPLE GROVE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 07/2020
Drawn by: MPG	Report No.: PG5398-1
Checked by: YT	Dwg. No.: PG5398-1
Approved by: SD	Revision No.:

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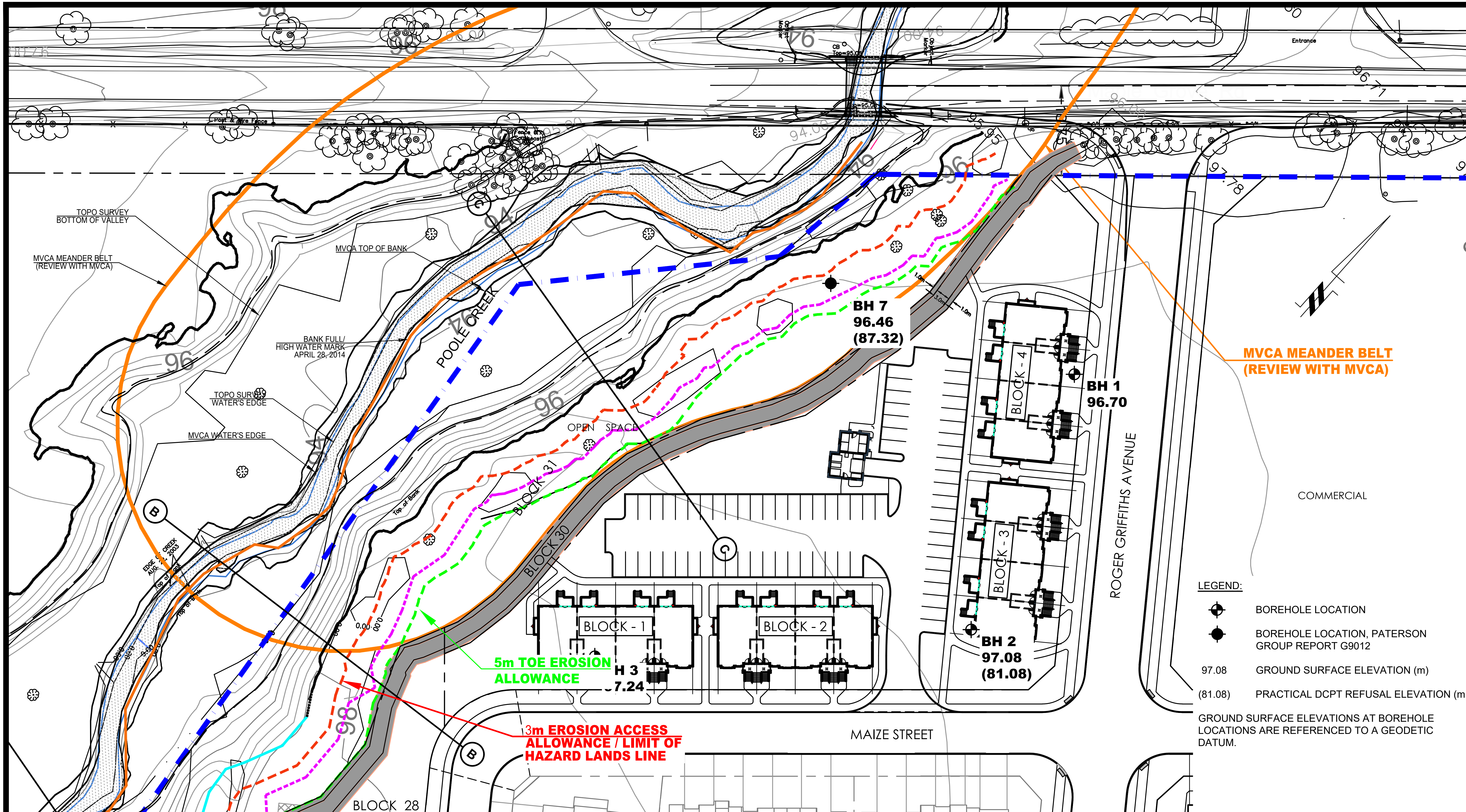

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RICHCRAFT HOMES

GEOTECHNICAL INVESTIGATION
KANATA WEST - BLOCK 29 - MAPLE GROVE ROAD

OTTAWA, ONTARIO

Title:

LIMIT OF HAZARD LANDS

Scale: 1:750	Date: 07/2020
Drawn by: MPG	Report No.: PG5398-1
Checked by: YT	Dwg. No.: PG5398-2
Approved by: SD	Revision No.:

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