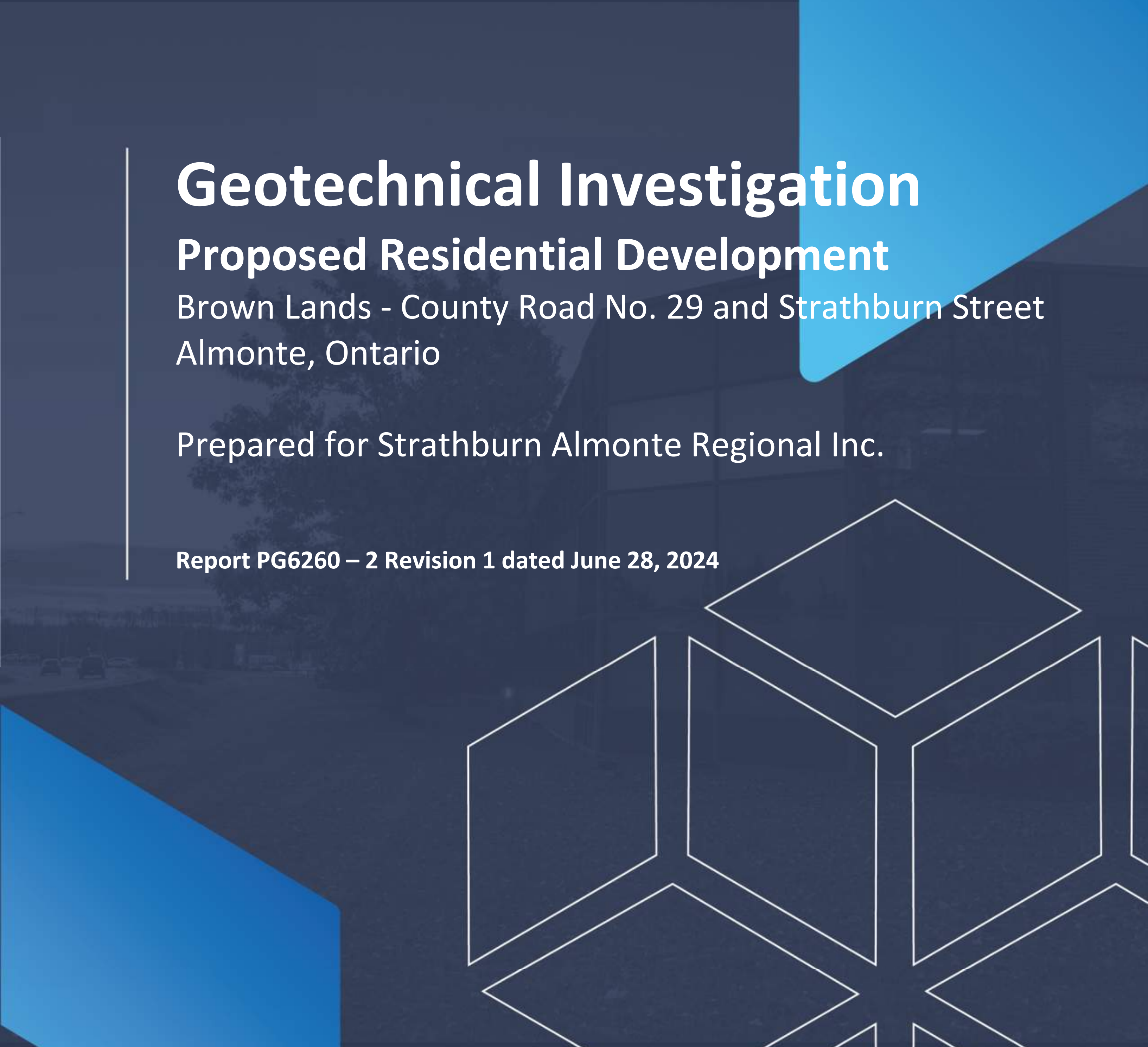

Geotechnical Investigation

Proposed Residential Development

Brown Lands - County Road No. 29 and Strathburn Street
Almonte, Ontario

Prepared for Strathburn Almonte Regional Inc.

Report PG6260 – 2 Revision 1 dated June 28, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Strathburn Almonte Regional Inc. to conduct a geotechnical investigation for the proposed residential development to be located at the northeast corner of County Road No. 29 and Strathburn Street, known as the Brown Lands, in Almonte, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

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 pertaining to 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.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on our review of available drawings, the proposed residential development will consist of low-rise residential dwellings and local roadways. It is anticipated that the residential dwellings will consist of low-rise buildings, each with a basement level, as well as attached garages and landscaped areas.

It is understood that the site will be municipally serviced by future water, sanitary and storm services. It is further understood that a pump station is proposed to service the subject site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on November 28, 29, and 30, 2022 and consisted of a total of thirteen (13) boreholes sampled to a maximum depth of 10.2 m below ground surface throughout the subject site. Further, five (5) probeholes were advanced to a maximum depth of 2.1 m below ground surface throughout the subject site.

Paterson had undertaken a previous preliminary investigation on May 19, 2022. At that time, four (4) boreholes and two (2) hand-auger holes were advanced to a maximum depth of 5.9 m. The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG6260-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU), collected using a 50 mm diameter split-spoon (SS) sampler, or grab samples (G) collected from the auger-head at hand-auger test hole locations. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags.

All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and grab samples were recovered from the boreholes are shown as AU, SS and G respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

The thickness of the overburden was also evaluated by the use of probeholes at several test hole locations. This technique consisted of advancing augers until refusal to augering was reached by the drill rig. Select soil samples were recovered from auger flights as the augers were advanced to refusal.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils.

Rock samples were recovered at borehole BH 7-22 using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the borehole is shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Groundwater monitoring wells were installed in boreholes BH 1-22 and BH 7-22, and flexible standpipe piezometers were installed in all other boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS 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 PG6260-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging. Three (3) soil samples were submitted for Atterberg Limit testing, one (1) sample was submitted for grain-size distribution analysis, and one (1) sample was submitted for shrinkage limit testing. All samples were submitted for moisture content testing. The test results are included in Appendix 1.

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

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 Section 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site consists of agricultural fields and several one to two-storey residential dwellings. The eastern portion of the subject is also currently occupied by several silos in proximity to the residential dwellings. The southwestern corner of the subject site is incised by a watercourse feature connecting to Wolf Creek. Bedrock outcroppings were also generally observed at the existing ground surface within the eastern half of the subject site.

The subject site is bordered to the north by agricultural fields, to the west by County Road No. 29, to the south by Strathburn Street and further by forested areas or single-family residential dwellings, and to the east by the Mississippi River. The ground surface across the western half of the site slopes down from west to east between approximate geodetic elevations 124 to 112 m. The slope varies between 3H:1V and 10H:1V throughout the central portion of the subject site. The eastern portion of the subject site is relatively flat.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile was observed to consist of a deposit of fill underlain by silty clay, and/or glacial till.

Fill, consisting of silty sand with gravel and trace amounts of clay, was encountered at boreholes HA 5-22, PH 17-22 and PH 19-22. The thickness of the fill layer was observed to range between approximately 0.2 m and 2.0 m.

A deposit of hard to very stiff silty clay was encountered at BH 1-22 to BH 4-22, BH 7-22, BH 10-22 and BH 12-22 to BH 16-22.

A compact deposit of glacial till was encountered below the topsoil layer at boreholes BH 8-22, BH 9-22, and PH 11-22, and below the silty clay layer at borehole BH 7-22 and BH 12-22. The glacial till was generally observed to consisted of silty sand with a variable amount of gravel, cobbles and boulders.

Practical refusal to augering, hand-augering and DCPT was encountered at all test holes at depths ranging between 0.2 and 18.8 m.

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

Bedrock

Based on geological mapping, the overburden drift thickness ranges between 0 and 3 m. The western and eastern halves of the subject site are underlain by dolomite of the Oxford Formation and interbedded sandstone and dolomite of the March Formation, respectively.

At borehole BH 7-22, where bedrock was cored with the drilling equipment, the bedrock was observed to consist of good to excellent, light brown to grey sandstone.

4.3 Groundwater

Groundwater level readings were measured on May 25, 2022 and December 7, 2022 and are presented in Table 1 below, and on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-22	115.01	4.48	110.53	December 7, 2022
BH 2-22	123.34	Dry	-	May 25, 2022
BH 3-22	118.18	1.83	116.35	May 25, 2022
BH 4-22	111.17	Dry	-	December 7, 2022
BH 7-22*	114.07	4.48	109.59	December 7, 2022
BH 10-22	124.97	Dry	-	December 7, 2022
BH 12-22	122.23	3.75	118.48	December 7, 2022
BH 13-22	122.43	2.68	119.75	December 7, 2022
BH 14-22	119.93	Dry	-	December 7, 2022
BH 15-22	117.37	7.08	110.29	December 7, 2022
BH 16-22	115.89	5.65	110.24	December 7, 2022

Note:
- The ground surface elevations are referenced to a geodetic datum.
- * Borehole with groundwater monitoring well

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is anticipated to be below the bedrock surface throughout the western and eastern portions of the subject site, respectively.

However, groundwater levels are subject to seasonal fluctuations and could vary during the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed buildings be founded on conventional spread footings bearing on the undisturbed, hard to very stiff silty clay, compact glacial till, approved engineered fill, and/or on clean surface-sounded bedrock.

It is anticipated that bedrock removal will be required for basement construction and/or site servicing activities in portions of the site. Therefore, all contractors should be prepared for bedrock removal within the subject site.

Due to the presence of a silty clay deposit throughout portions of the site, permissible grade raise restrictions have been provided.

Foundation uplift resistance may be required for the pump station, recommendations for this are provided in Section 5.6.

A slope stability assessment has been completed to evaluate the stability of the existing slopes at the site, and to provide Limit of Hazard Lands setbacks, where required. This is discussed further in Section 6.9.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by Paterson personnel at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with 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 the proximity of the blasting operations should be carried out 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 or 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 surrounding structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations 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, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill placed 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 imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building areas should be compacted to a minimum of 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 lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 98% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geo-composite drainage membrane connected to a perimeter drainage system.

If site-excavated/site-blasted rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm and sampled, reviewed and approved by Paterson prior to use throughout the subject site. The material is generally recommended to be placed in maximum 300 mm thick loose lifts and compacted using a suitably sized vibratory roller. Any site-excavated rock material greater than 300 mm in diameter should be segregated and hoe-rammed into acceptable fragments. Where the fill is open-graded, a blinding layer of finger granular fill or a geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements.

5.3 Foundation Design

Conventional Spread Footings

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, very stiff silty clay or compact to dense glacial till, or on engineered fill which is placed and compacted directly over these strata, can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applies to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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

Footings supported directly on clean, surface-sounded sandstone bedrock can be designed using a bearing resistance value at ultimate limit states (ULS) of **750 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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

Footings supported directly on clean, surface sounded bedrock and design for the bearing resistance values provided above will be subject to negligible post-construction total and differential settlements.

Proposed Sanitary Pump Station

It is anticipated the proposed sanitary pump station will consist of a wet well structure with an adjacent slab on grade. The wet well structure is expected to be founded on a bedrock surface and can be designed using the bearing resistance value provided above. Recommendations to resist potentially buoyancy of the wet well structure are provided in Section 5.6.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Adequate lateral support is provided to a soil bearing medium 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.

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

Soil/Bedrock Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long-term total and differential settlements.

At the soil/bedrock transitions, it is recommended that a minimum depth of 300 mm of bedrock be removed from below the founding elevation for a minimum length of 2.0 m on the bedrock side. This area should be subsequently reinstated with an engineered fill, such as OPSS Granular A or OPSS Granular B Type II crushed stone and compacted to a minimum of 98% of the materials SPMDD.

Permissible Grade Raise

Due to the presence of the silty clay deposit throughout the western portion of the subject site, a permissible grade raise restriction is recommended for grading at the subject site where silty clay is present. Reference should be made to Drawing PG6260-2 – Permissible Grade Raise Plan in Appendix 2 of this report for the areas where the permissible grade raise restriction are recommended.

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 C** for the foundations considered. If a higher seismic site class is required (Class A or B), and the proposed foundations are within 3 m of the bedrock surface, a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings, as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2020. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2020 for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab-on-Grade Construction

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

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone.

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

5.6 Foundation Uplift Resistance for Pump Station

Buoyancy forces on the proposed pump station’s wet well structure can be resisted by either designing the expanded concrete base and overlying soil to act as a deadman anchor, or alternatively using rock anchors. Recommendations for the design of deadman anchors and rock anchors are provided in the following subsections.

Deadman Anchor Design

The dead weight of the concrete base and the weight of soil over the expanded base could be designed to act as a deadman anchor to resist the buoyant uplift loads on the wet well structure.

Geotechnical parameters for typical backfill materials compacted to 98% of SPMDD in 300 mm lift thicknesses are provided in Table 2, along with the associated earth pressure coefficients for horizontal resistance calculations for deadman anchors. Also, friction factors between concrete and the various subgrade materials are also provided in Table 2 below.

For soil above the groundwater level, calculate using the “drained” unit weight and below groundwater level use the “effective” unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage is not provided.

Table 2 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal Friction Angle (°) φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A Fill (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.58
OPSS Granular B Type I Fill (Well-Graded Sand-Gravel)	21.5	13.5	36	0.55	0.26	0.41	3.85
OPSS Granular B Type II Fill (Crushed Stone)	22.5	14	40	0.6	0.2	0.33	5.04

Notes:

- Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
- The earth pressure coefficients provided are for horizontal backfill profile.

Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed. The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long-term performance of the foundation of the proposed building, if required, any rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of sandstone ranges between about 40 and 50 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a Rock Mass Rating (RMR) of 65 was assigned to the bedrock, and Hoek and Brown parameters (m and s) were taken as 0.575 and 0.00293, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 3 below:

Table 3 – Parameters used in Rock Anchor Design	
Grout to Rock Bond Strength – Factored at ULS	1.0 MPa
Compressive Strength – Grout	40 MPa
Rock Mass Rating (RMR) – Good quality Sandstone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength – Sandstone bedrock	40 MPa
Unit weight – Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 4 below.

The factored tensile resistance values given in Table 4 are based on a single anchor with no group influence effects.

Table 4 – Recommended Rock Anchor Lengths – Grouted Rock Anchor

Diameter of Drill Hole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (kN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	2.0	0.8	2.8	450
	2.6	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed pump station are determined.

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel, and should be flushed clean prior to grouting.

A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request.

5.7 Pavement Structure

For design purposes, the following pavement structures, presented below, are recommended for the design of the car parking areas and local roadways.

Table 5 - Recommended Pavement Structure – Driveways

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 – Local Residential Roadways

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, bedrock 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 99% of the material's SPMDD using suitable vibratory equipment.

If bedrock is encountered at the subgrade level, the total thickness of the pavement granular materials (base and subbase) could be reduced to 300 mm for the above-noted pavement structures. The upper 300 mm of the bedrock surface should be reviewed and approved by Paterson prior to placing the base and subbase materials. Care should be exercised to ensure that the bedrock subgrade does not have depressions that will trap the water.

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.

Where silty clay is encountered at subgrade level, consideration should be given to installing subdrains during the pavement construction. The invert of the subdrain pipe is recommended to be located a minimum depth of 300 mm below the pavement structure subgrade and located centrally along the roadway alignment. The subdrain pipe is recommended to consist of a minimum 150 mm diameter corrugated and perforated plastic pipe surrounded by a minimum of 150 mm of 10 mm clear crushed stone on all of its sides. The clear stone layer is recommended to be wrapped by a geotextile layer. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

Foundation Backfill

For proposed buildings with below-grade space, backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) connected to a drainage system is provided.

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. Generally, a minimum of 1.5 m thick soil cover (or an equivalent combination of soil cover and foundation insulation) 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 structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

Where the bedrock is considered frost susceptible, foundation insulation will need to be provided or the frost susceptible bedrock will need to be removed and replaced with lean concrete (minimum 17 MPa 28-day strength).

6.3 Excavation Side Slopes

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. It is anticipated that sufficient space will be available for the great part of the excavations to be undertaken by open-cut methods (i.e., unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m, should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below ground water level. The subsoil at this site appeared to be mainly a Type 2 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. However, the bedding thickness should be increased to 300 mm for areas over a bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material’s standard Proctor maximum dry density.

Based on the soil profile encountered at the subject site, the subgrade for the services will be placed in both bedrock and overburden soils. It is recommended that the subgrade medium be inspected in the field to determine how steeply the bedrock surface, where encountered, drops off. A transition should be provided where the bedrock slopes more than 3H:1V.

At these locations, the bedrock should be excavated and replaced with additional bedding materials to provide a 3H:1V (or flatter) transition from the bedrock subgrade towards the soil subgrade. This treatment reduces the propensity for bending stress to occur in the services.

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 reduce potential differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in the longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will 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 package and issuance of the permit by the MECP.

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

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 using 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 building 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 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Considerations

Due to the presence of the silty clay deposit in the western portion of the site, the location of street trees will be governed by the potential for soil volume change where trees and houses are located above a silty clay deposit. The areas where tree planting setbacks are required have been outlined in Drawing PG6260-3 – Tree Planting Setback Plan presented in Appendix 2.

Area 1 – Clay Soil of Low/Medium Sensitivity to Soil Volume Change

Based on these current testing results, the plasticity index for all of the tested clay samples were found to be less than 40%, which would indicate the presence of a clay of low to medium potential for soil volume change. Buildings considered throughout the area with these clays may be provided a reduced tree-to-foundation setbacks.

Based on the current guidelines, large trees (mature height over 14 m) can be planted within Area 1 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 7.5m) 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 center of the tree trunk and verified by means of the Grading Plan.

☐ A small tree must be provided with a minimum of 25 m^3 of available soils 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 surrounding 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.

Area 2 – Soils Not Sensitive to Soil Volume Change

The east portion of the subject site was noted to consist of cohesionless soils overlying the bedrock. Cohesionless soils are not sensitive to soil volume change hence a tree-to-foundation setback is not specified for the subject area.

Above-Ground Swimming Pools, Hot Tubs, Decks and Additions

The in-situ soils are considered 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.

Additional grading around the hot tub should not exceed permissible grade raise restrictions. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer's specifications.

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

6.9 Slope Stability Analysis

Summary of Assessment

Paterson completed a field review of the slope along the east portion of the site, adjacent to the west and north tributaries, and alongside the Mississippi River as part of the current investigation.

The field review generally consisted of observing surface conditions along the length of the slope face and watercourse identifying the presence of vegetation, erosion and other features associated with slope stability. Paterson field personnel verified subsurface conditions at select slope sections using a hand auger.

Water levels and flow within the watercourses were reviewed generally, if present, including identifying signs of recent high-water marks or other signs of previous rises in the water levels. The top of slope alignment was determined in the field by Paterson personnel based on our field observations and recorded using a high-precision handheld GPS unit.

Topographic surface elevations were measured at select cross-sections to analyze slope stability using SLIDE, a computer program for two-dimensional slope stability analysis. Overall, a total of three (3) slope cross sections throughout the above-noted locations were analyzed as part of the slope stability analysis.

Based on the results of our field observations and slope stability analysis, a Limit of Hazard Lands was assigned from the top of slope for the above-noted cross-section A-A. The cross-section locations and associated Limit of Hazard Lands setbacks are presented on Drawing PG6260-1 – Test Hole Location Plan in Appendix 2.

Field Observations

The slope observed at the west portion of the site was observed to have an approximate incline ranging between 2.5H:1 to 3.5H:1V and surfaced with mature vegetation and small trees. A small tributary was located along the slope and was noted to be dry at the time of the field review. The majority of the abutting table lands are agricultural fields. The slope was noted to range between 4 m and 6 m in height. No signs of erosion, distress or sloughing were observed throughout the slope surface at the time of our field review.

The slope observed at the east portion of the site was observed to have an approximate incline ranging between 2H:1 to 3H:1V and surfaced with mature vegetation and small to medium trees. The Mississippi River was located along the slope at the east portion of the site. Bedrock outcrops were observed along the slope bordering the Mississippi River. The riverbed in this area was noted to consist of bedrock with minimal overburden.

Slope Stability Analysis

The slope stability analysis was modeled in SLIDE, a computer program which permits a two-dimensional slope stability analysis calculating several methods including the Bishop's method, which is a widely accepted slope analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to forces favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsurface soil and groundwater conditions, a factor of safety greater than 1.0 is generally required for the failure risk to be considered acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise 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.

Three (3) slope cross-sections (Sections A, B, and C) were studied as the worst case scenarios. The cross section locations are presented on Drawing PG6260-1 – Test Hole Location Plan in Appendix 2. It should be noted that details of the slope height and slope angle at the cross-section locations are presented in Figures 2A through4B in Appendix 2, based on the topographic data obtained during the field investigation, as well as the available topographic survey plan for the site.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 7 on the following page.

Table 7 – Effective Stress Soil Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay	17	33	7
Grey Silty Clay	16	33	10
Glacial Till	20	36	0
Bedrock	22	-	-

The total strength parameters for seismic analysis were chosen based on the subsurface conditions observed in the test holes, and our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 8 below.

Table 8 – Total Stress Soil Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay	17	-	150
Grey Silty Clay	16	-	75
Glacial Till	20	36	0
Bedrock	22	-	-

Stable Slope Allowance

West Tributary

The static analysis results for slope cross-sections A-A, and B-B are presented in Figures 2A and 3A, respectively, provided in Appendix 2. The factors of safety for the slopes was greater than 1.5 for slope cross-section B-B. A factor of safety less than 1.5 was noted for slope cross-section A-A, therefore, a slope stability setback would be required, if the existing slope was not re-graded as part of the proposed development. A stable slope setback of 9 m, for slope cross-section A-A, would be required if the existing slope is not modified.

The results of the analyses with seismic loading are shown in Figures 2B and 3B, presented in Appendix 2. The factor of safety for the slopes was greater than 1.1 for all slope cross-sections. Based on these results, the slopes are considered to be stable under seismic loading. No further stable slope setback is required.

North Tributary

The slope along the north tributary presents similar subsurface conditions to that along the west tributary, however, with a slightly flatter slope. Based on this and the results of the analysis, the slope is considered stable under static and seismic loading conditions. No further stable slope setback is required from a geotechnical perspective.

Mississippi River

Based on the results of our analysis, the factor of safety for static and seismic loading conditions at slope cross-section C-C exceed 1.5 and 1.1, respectively. Based on this, a stable setback allowance is not required for this area from a geotechnical perspective.

Toe Erosion and Erosion Access Allowance

West Tributary

The slope was generally observed to be vegetated with small trees and brush. Furthermore, no water or erosion was observed at the toe of the slope along the west tributary. Considering the existing conditions of the toe of the slope, a toe erosion allowance is not required for the subject slope.

North Tributary

The slope was generally observed to be covered by agricultural fields. Further, a small watercourse was observed to be located over 30 m away from the bottom of the subject slope. No erosion or distress associated with erosion was observed at the bottom of the slope at the time of our review. Based on current guidelines, since the edge of the watercourse is located more than 15 m from the bottom of the slope, a toe erosion setback is not considered applicable, from a geotechnical perspective.

Given that no stable slope setback or toe erosion setback is required along the slope adjacent to the north tributary, an erosion access allowance is not required.

Mississippi River

The slope was generally observed to be vegetated with small to medium trees and brush. The toe of the slope along the Mississippi River was noted to consist of a relatively large and active watercourse. The slope surface in contact with the watercourse was observed to consist of relatively intact bedrock which did not indicate signs of erosion. However, the bedrock surface is covered by a thin layer of overburden along the slope surface. Based on these observations, a toe erosion allowance of 1 m is recommended for the subject slope.

A 6 m erosion access allowance was applied from the top of stable slope to allow for future maintenance of the slope, if required.

Limit of Hazard Lands

West Tributary

The results of the slope stability assessment indicate that the Limit of Hazard Lands setback of 15 m measured from the top of the slope, should be provided for any proposed structures in the area of slope cross-section A-A, in order to provide a suitable factor of safety of 1.5 under static conditions and 1.1 under seismic conditions.

The Limit of Hazard Lands at this location setback may be avoided by lowering the finished grade to a geodetic elevation of approximately 118 m within the extents of the Limit of Hazard Lands setback. Alternatively, the grade at the toe of the slope can be raised to a geodetic elevation of approximately 115 m, to a distance of about 15 m away from the toe of the slope, in order to avoid the Limit of Hazard Lands in this area.

The subject site in the areas of slope cross-section B-B does not require a Limit of Hazard Lands setback in order to provide a suitable factor of safety of 1.5 under static conditions and 1.1 under seismic conditions.

North Tributary

The subject site in the areas adjacent to the north tributary does not require a Limit of Hazard Lands setback in order to provide a suitable factor of safety of 1.5 under static conditions and 1.1 under seismic conditions.

Mississippi River

The results of the slope stability assessment indicate that Limit of Hazard Lands setback of 7 m measured form the top of the slope, should be provided for any proposed structures at the subject site in the areas of slope cross-section C-C and adjacent to the Mississippi River, in order to provide a suitable factor of safety of 1.5 under static conditions and 1.1 under seismic conditions.

Additional Considerations

Grade raises above those provided on our plan PG6260-2 – Permissible Grade Raise Plan are not recommended throughout the sloped portions of the subject site. It should be noted that the proposed grading for areas in proximity to the subject slopes should be reviewed by Paterson, from a slope stability perspective, once finalized grading throughout these areas is known.

Based on the available Conceptual Grading Plan, it should be noted that the revised lots 87 to 95 along the West Tributary are stable, as the grades are within the permissible grade restriction. However, as noted above, this will need to be confirmed once the finalized Grading Plan is available.

Should proposed grading exceed our recommendations, Paterson should review the grading from a slope stability perspective to assess potential impacts to the slope.

It is recommended that the existing vegetation and mature trees not be removed from the slope faces as the presence of the vegetation reduces surficial erosion activities. If the existing vegetation needs to be removed along the slope faces, 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.

7.0 Recommendations

For the foundation design data provided herein to be applicable that a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

- ☐ Review preliminary and detailed grading and servicing 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.
- ☐ Observation of the placement of the foundation insulation, if applicable.
- ☐ 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 and follow-up field density tests to determine the level of compaction achieved.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations provided herein are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests 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 the 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 Strathburn Almonte Regional Inc., or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Nicolas Seguin

Nicolas Seguin, EIT

LICENSED PROFESSIONAL ENGINEER
June 28, 2024
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Scott Dennis
PROVINCE OF ONTARIO

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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG TESTING RESULTS

GRAIN SIZE ANALYSIS RESULTS

SHRINKAGE ANALYSIS RESULTS

ANALYTICAL TESTING RESULTS

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 19, 2022

FILE NO. PG6260

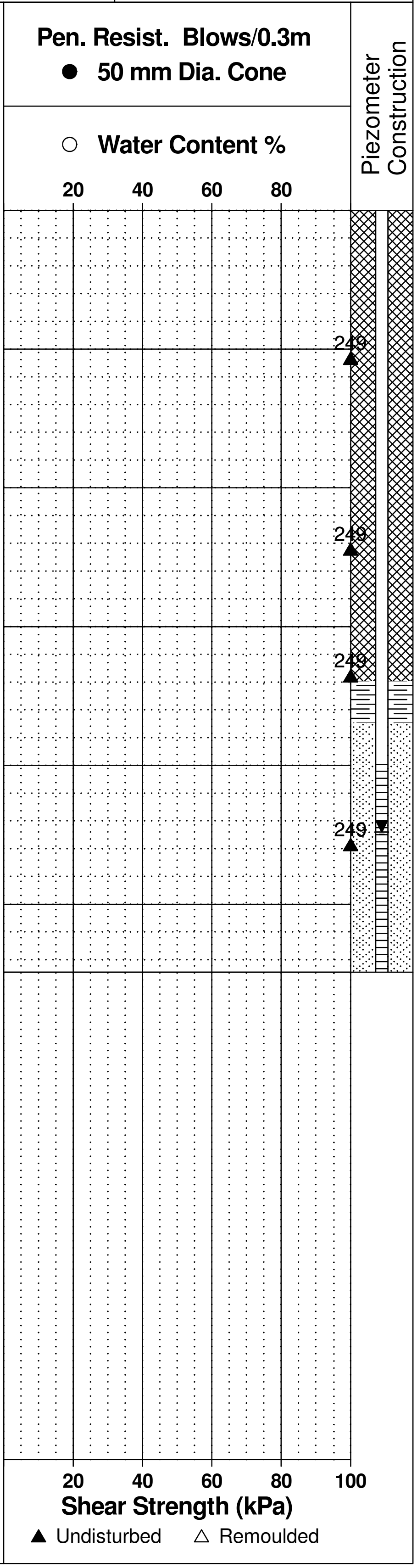
HOLE NO. BH 1-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	115.01
TOPSOIL 0.08							
		SS	1	8	13		
		SS	2	100	16	1	114.01
		SS	3	100	11		
						2	113.01
Hard to very stiff, brown **SILTY CLAY**							
						3	112.01
		SS	4	83	P		
						4	111.01
		SS	5	83	9		
						5	110.01
5.49							

End of Borehole

Practical refusal to augering at 5.49 m depth

(GWL at 4.48m - Dec. 7, 2022)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

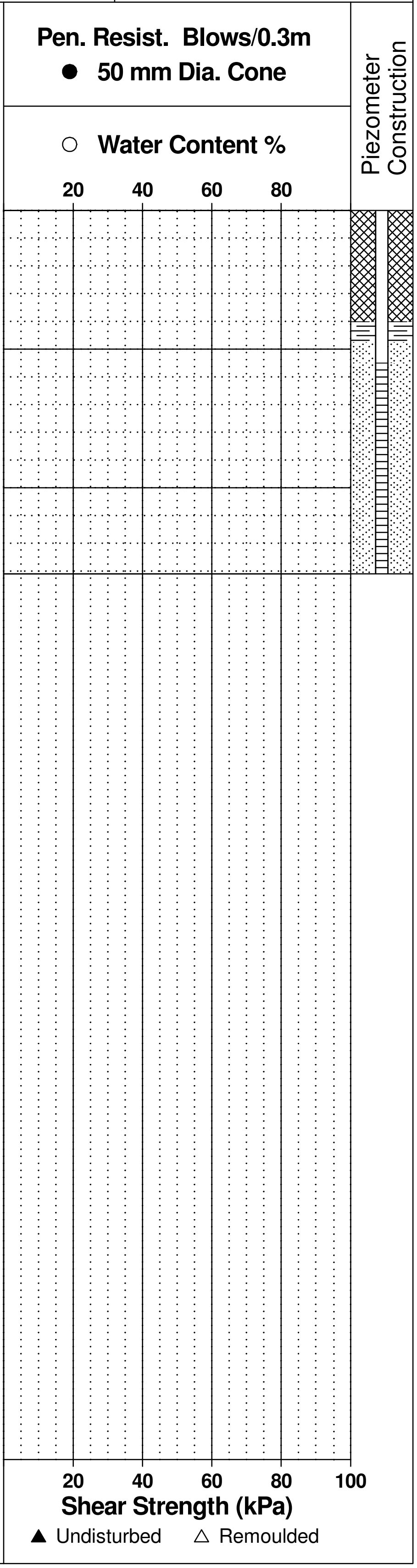
BORINGS BY CME 55 Power Auger

DATE May 19, 2022

FILE NO. PG6260

HOLE NO. BH 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	123.34
TOPSOIL 0.10							
Hard to very stiff, brown **SILTY CLAY**		SS	1	42	8		
		SS	2	100	19	1	122.34
		SS	3	92	13	2	121.34
2.62		SS	4	50	50+		
End of Borehole Practical refusal to augering at 2.62 m depth (Piezometer dry - May 25, 2022)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 19, 2022

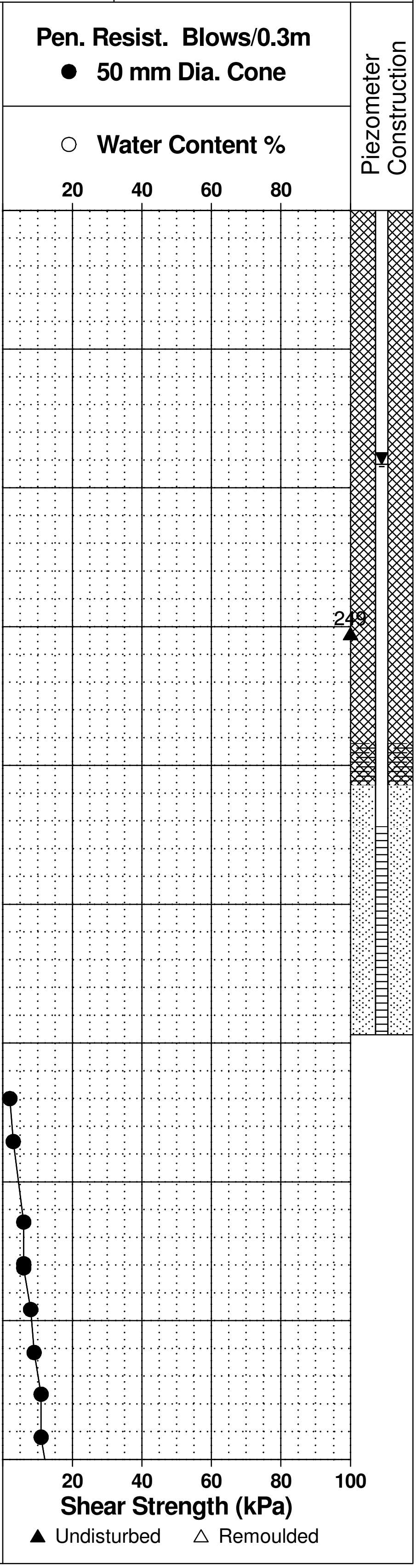
FILE NO. PG6260

HOLE NO. BH 3-22

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.13
Hard to very stiff, brown **SILTY CLAY**	5.94
Dynamic Cone Penetration Test commenced at 5.94m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	25	8
SS	2	75	15
SS	3	83	6
SS	4	100	5
SS	5	100	8
SS	6	83	11
SS	7	92	8

DEPTH (m)	ELEV. (m)
0	118.18
1	117.18
2	116.18
3	115.18
4	114.18
5	113.18
6	112.18
7	111.18
8	110.18
9	109.18



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DATUM Geodetic

REMARKS

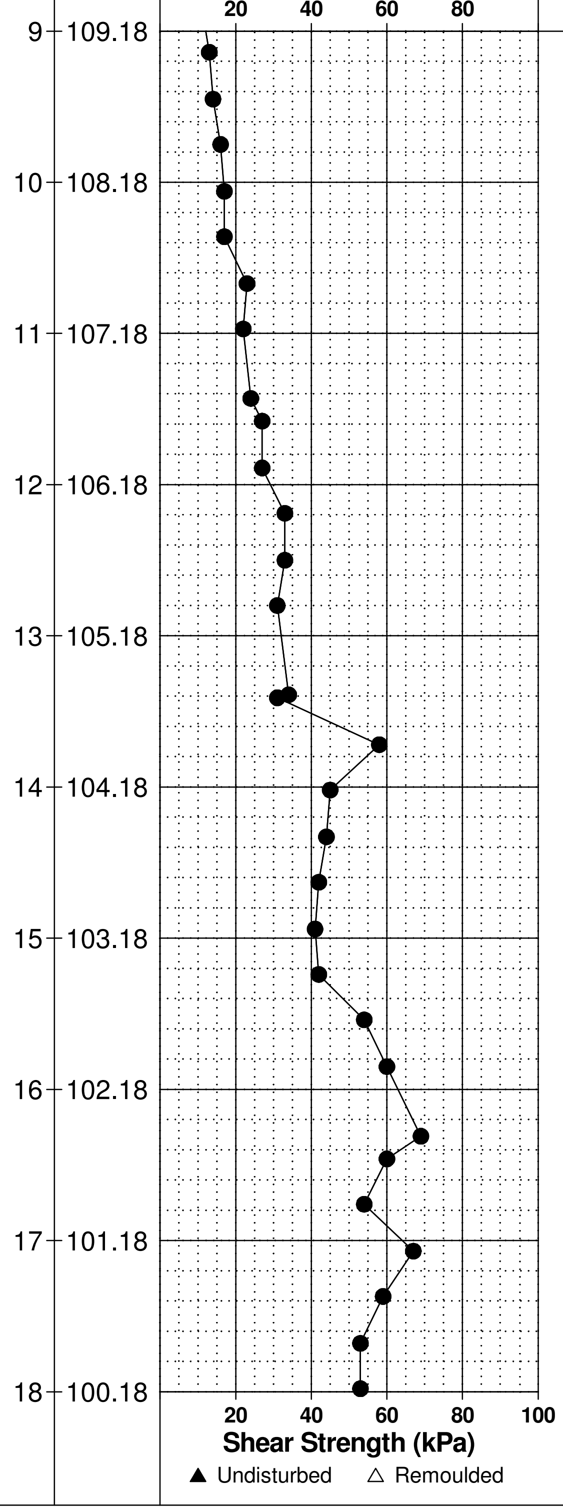
BORINGS BY CME 55 Power Auger

DATE May 19, 2022

FILE NO. PG6260

HOLE NO. BH 3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				



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DATUM Geodetic

REMARKS

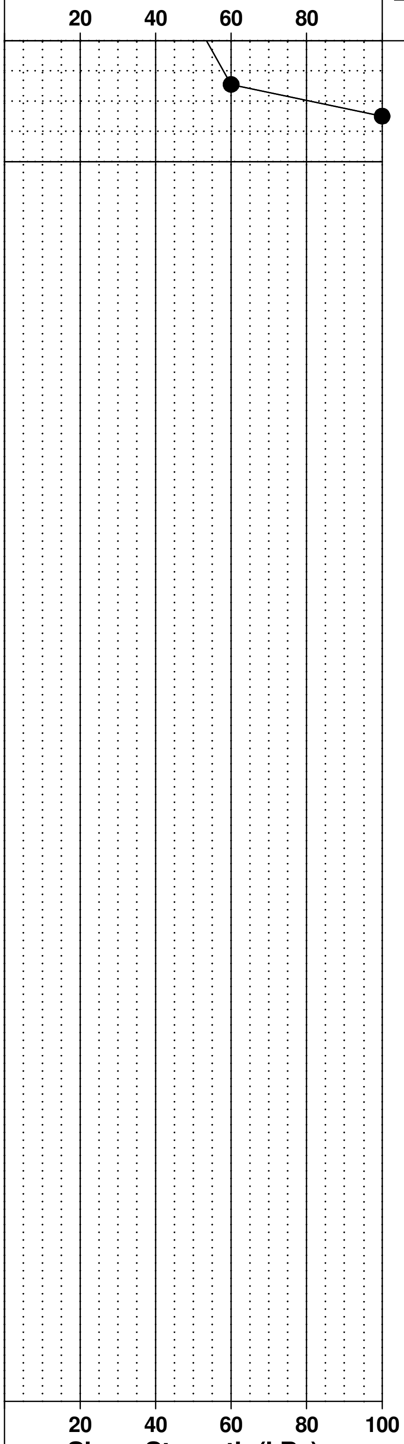
BORINGS BY CME 55 Power Auger

DATE May 19, 2022

FILE NO. PG6260

HOLE NO. BH 3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						18	100.18		
18.80 End of Borehole Practical refusal to DCPT at 18.80 m depth (GWL at 1.83m - May 25, 2022)									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

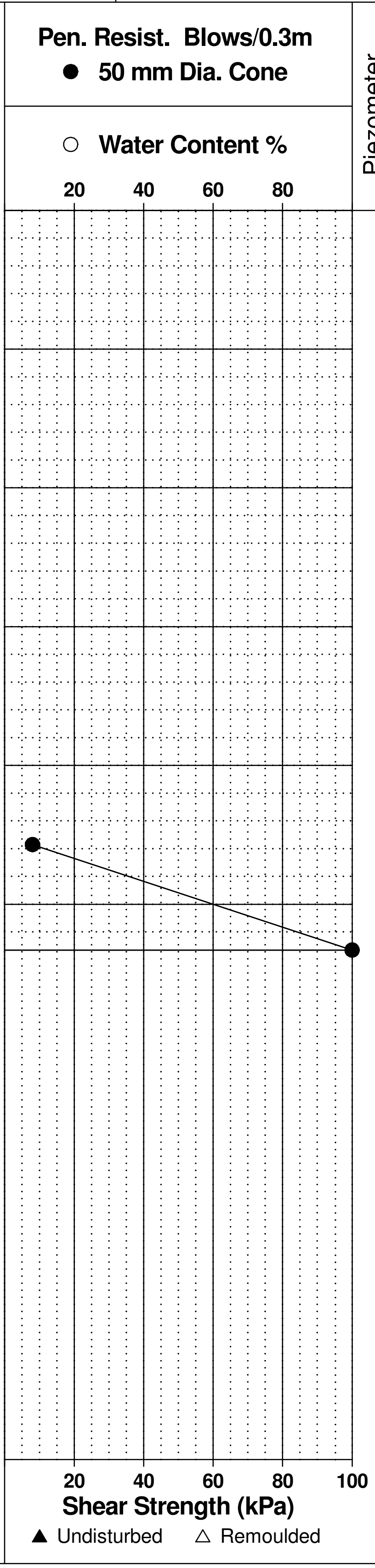
BORINGS BY CME 55 Power Auger

DATE May 19, 2022

FILE NO. **PG6260**

HOLE NO. **BH 4-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	111.17	
TOPSOIL 0.15	■							
Hard to very stiff, brown **SILTY CLAY**	▨	SS	1	8	4			
	▨	SS	2	33	11	1	110.17	
	▨	SS	3	67	12	2	109.17	
	▨	SS	4	33	12	3	108.17	
3.66	▨	SS	5	100	9			
Dynamic Cone Penetration Test commenced at 3.66m depth						4	107.17	
5.33						5	106.17	
End of Borehole Practical refusal to DCPT at 5.33 m depth								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hand Shovel

DATE May 19, 2022

FILE NO. PG6260

HOLE NO. HA 5-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE									
TOPSOIL 0.30	■					0	110.55		
FILL: Brown siltys and with gravel, trace clay 0.46	▨	G	1						
End of Borehole Practical refusal to hand augering at 0.46m depth.									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

DATUM Geodetic

REMARKS

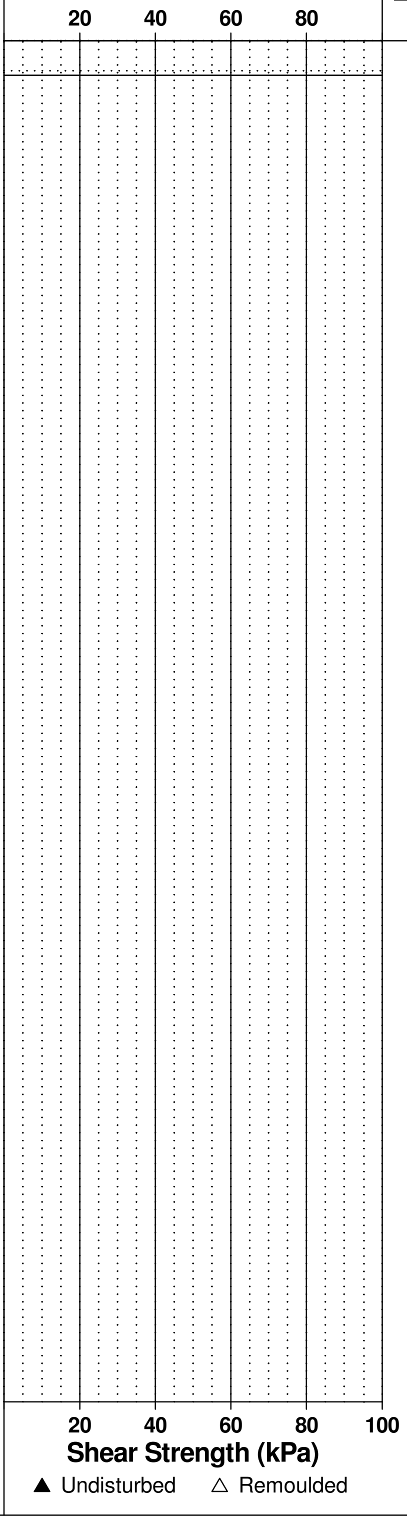
BORINGS BY Hand Shovel

DATE May 19, 2022

FILE NO. PG6260

HOLE NO. HA 6-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	111.84		
TOPSOIL 0.23	■	G	1						
End of Borehole Practical refusal to hand augering at 0.23m depth.									



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DATUM Geodetic

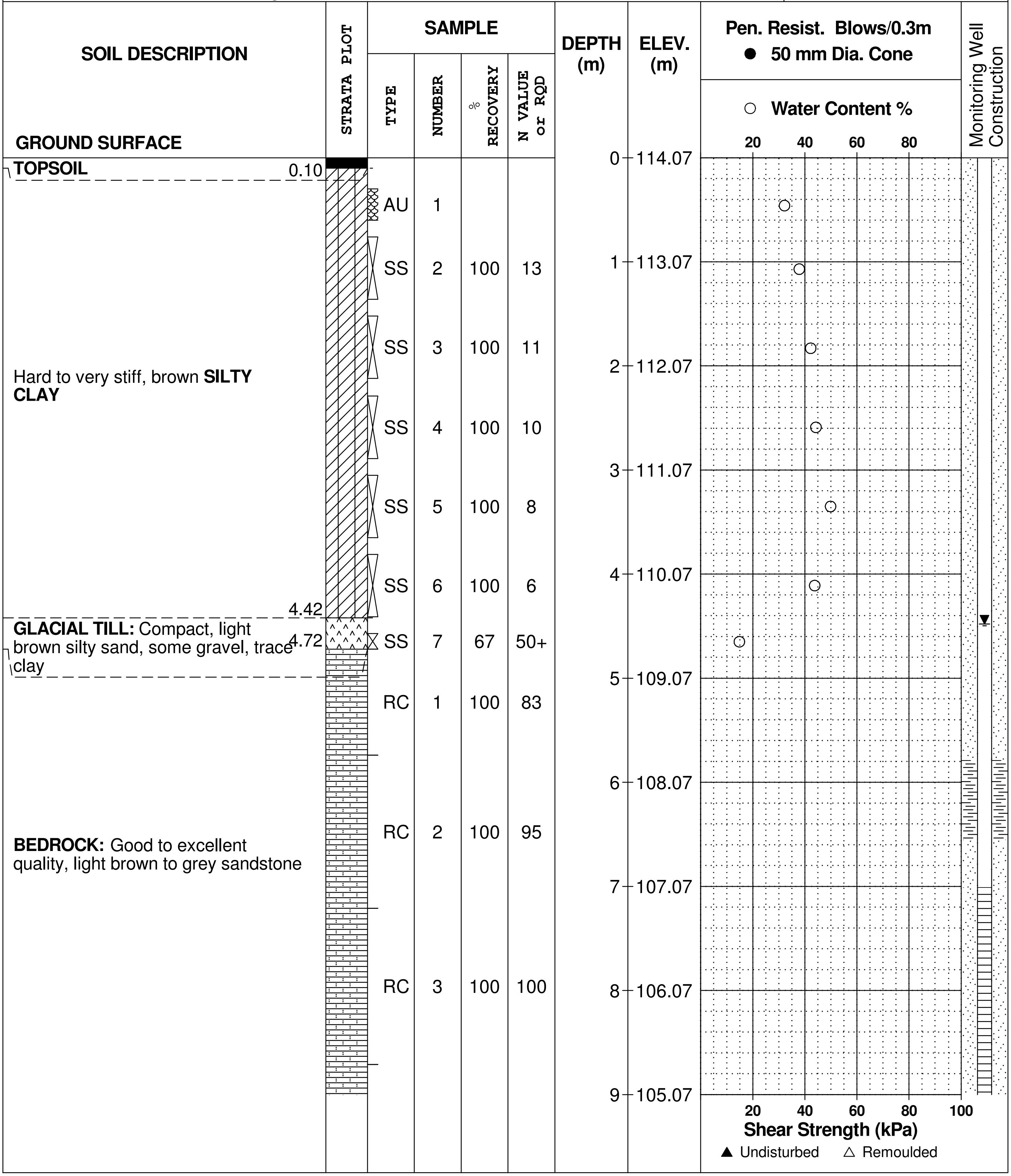
REMARKS

BORINGS BY CME 55 Power Auger

DATE November 28, 2022

FILE NO. PG6260

HOLE NO. BH 7-22



SOIL DESCRIPTION	Depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.10				
Hard to very stiff, brown **SILTY CLAY**		AU	1		
		SS	2	100	13
		SS	3	100	11
		SS	4	100	10
		SS	5	100	8
	4.42	SS	6	100	6
GLACIAL TILL: Compact, light brown silty sand, some gravel, trace clay	4.72	SS	7	67	50+
BEDROCK: Good to excellent quality, light brown to grey sandstone		RC	1	100	83
		RC	2	100	95
		RC	3	100	100

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REMARKS

BORINGS BY CME 55 Power Auger

DATE November 28, 2022

FILE NO. PG6260

HOLE NO. BH 7-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE									
						9	105.07		
BEDROCK: Good to excellent quality, light brown to grey sandstone		RC	4	100	100				
						10	104.07		
10.21									
End of Borehole (GWL at 4.48m - Dec. 7, 2022)									

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

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SOIL PROFILE AND TEST DATA

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REMARKS

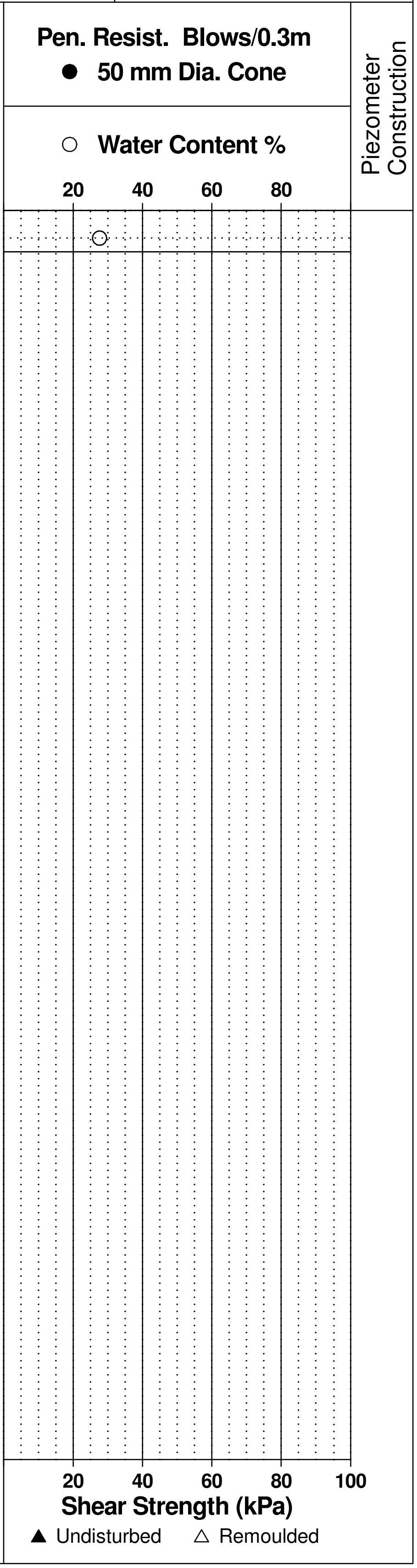
BORINGS BY CME 55 Power Auger

DATE November 28, 2022

FILE NO. PG6260

HOLE NO. BH 8-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	118.03
TOPSOIL 0.15							
GLACIAL TILL: Brown silty sand, some gravel, trace clay 0.30		AU	1				
End of Borehole Practical refusal to augering at 0.30m depth.							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - County Rd. 29 & Strathburn St.
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DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE November 28, 2022

FILE NO. PG6260

HOLE NO. BH 9-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE									
TOPSOIL 0.30						0	119.99		
GLACIAL TILL: Brown silty sand, some gravel, trace clay 0.46									
End of Borehole Practical refusal to augering at 0.46m depth.									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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REMARKS

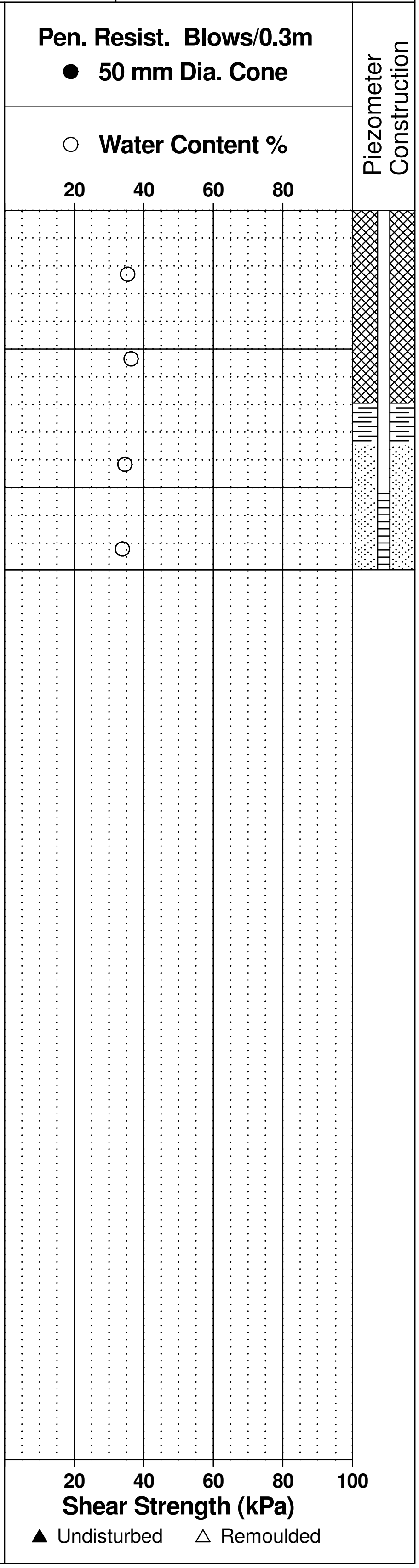
BORINGS BY CME 55 Power Auger

DATE November 28, 2022

FILE NO. PG6260

HOLE NO. BH10-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	124.97
TOPSOIL 0.13							
Hard to very stiff, brown **SILTY CLAY**		AU	1				
		SS	2	100	12	1	123.97
		SS	3	100	16	2	122.97
2.59		SS	4	100	50+		
End of Borehole Practical refusal to augering at 2.59m depth. (BH dry - December 7, 2022)							



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SOIL PROFILE AND TEST DATA

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REMARKS

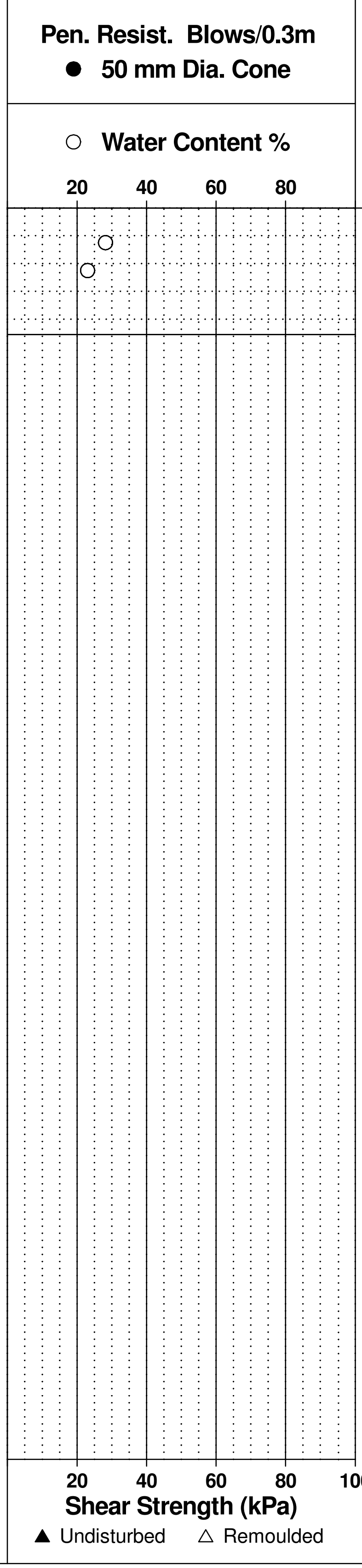
BORINGS BY CME 55 Power Auger

DATE November 28, 2022

FILE NO. PG6260

HOLE NO. PH11-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	118.14	
TOPSOIL 0.13								
GLACIAL TILL: Brown silty sand with clay, gravel, cobbles and boulders 0.91		AU AU	1 2					
End of Borehole Practical refusal to augering at 0.91m depth.								



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SOIL PROFILE AND TEST DATA

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Prop. Residential Dev. - County Rd. 29 & Strathburn St.
Almonte, Ontario

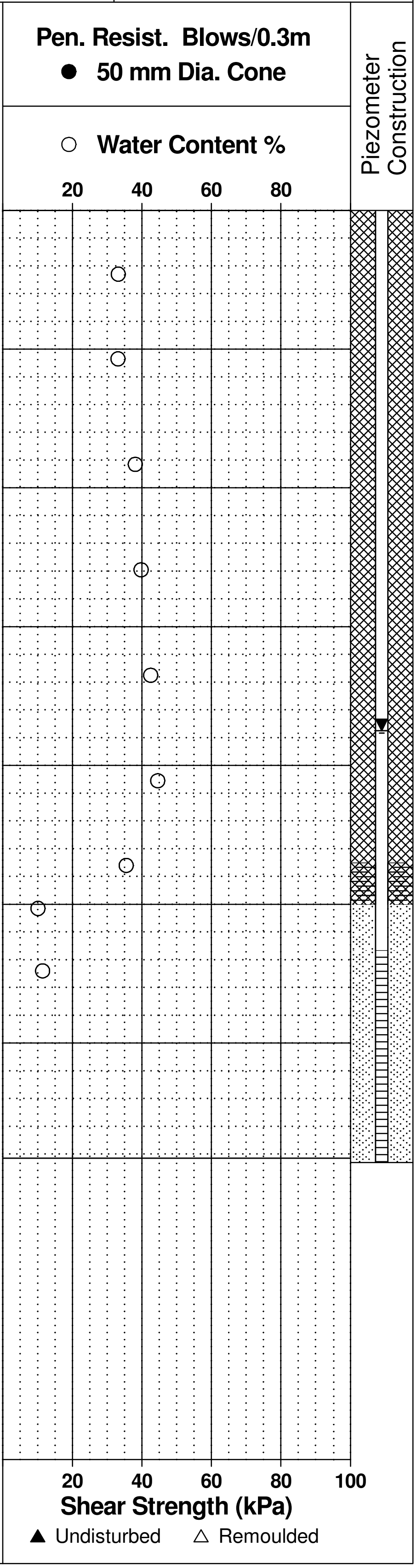
DATUM Geodetic
REMARKS
BORINGS BY CME 55 Power Auger
DATE November 28, 2022
FILE NO. **PG6260**
HOLE NO. **BH12-22**

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	122.23
TOPSOIL 0.13						
Hard to very stiff, brown **SILTY CLAY**	AU	1				
	SS	2	100	10	1	121.23
	SS	3	100	11	2	120.23
	SS	4	100	10		
	SS	5	100	10	3	119.23
	SS	6	100	10	4	118.23
5.00	SS	7	100	10	5	117.23
GLACIAL TILL: Light brown silty sand with gravel, cobbles and boulders, trace clay	SS	8	50	50+	6	116.23
6.83						

End of Borehole

Practical refusal to augering at 6.83m depth.

(GWL at 3.75m - Dec. 7, 2022)



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Prop. Residential Dev. - County Rd. 29 & Strathburn St.
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DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE November 29, 2022

FILE NO. PG6260

HOLE NO. BH13-22

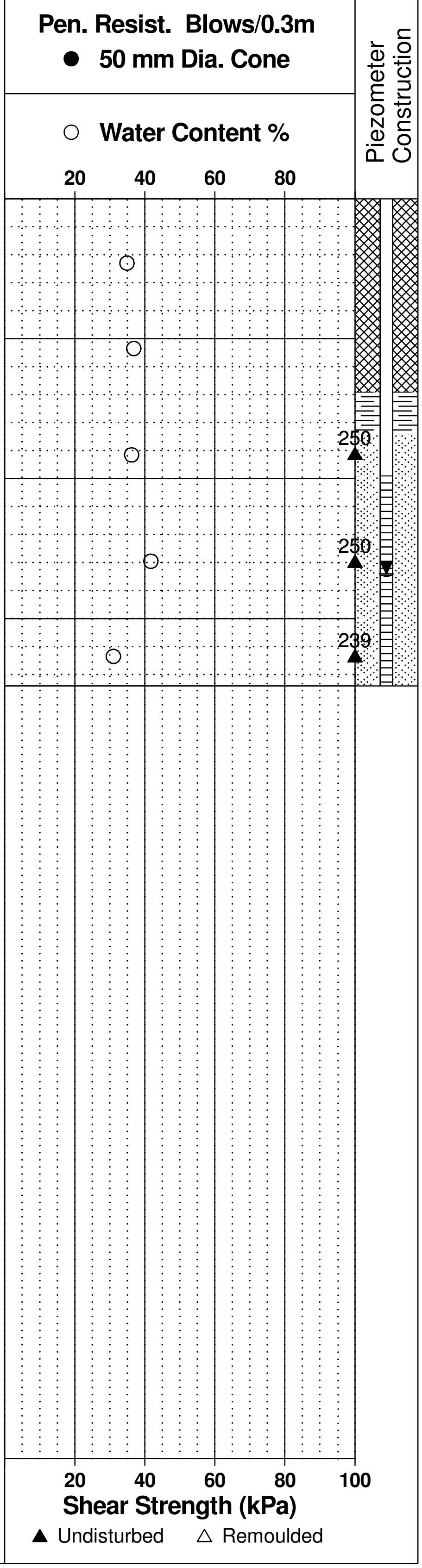
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.13					
Hard, brown **SILTY CLAY**		AU	1		
		SS	2	100	13
		SS	3	100	P
		SS	4	100	P
3.48		SS	5	94	P

DEPTH (m)	ELEV. (m)
0	122.43
1	121.43
2	120.43
3	119.43

End of Borehole

Practical refusal to augering at 3.48m depth.

(GWL at 2.68m - Dec. 7, 2022)



SOIL PROFILE AND TEST DATA

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Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

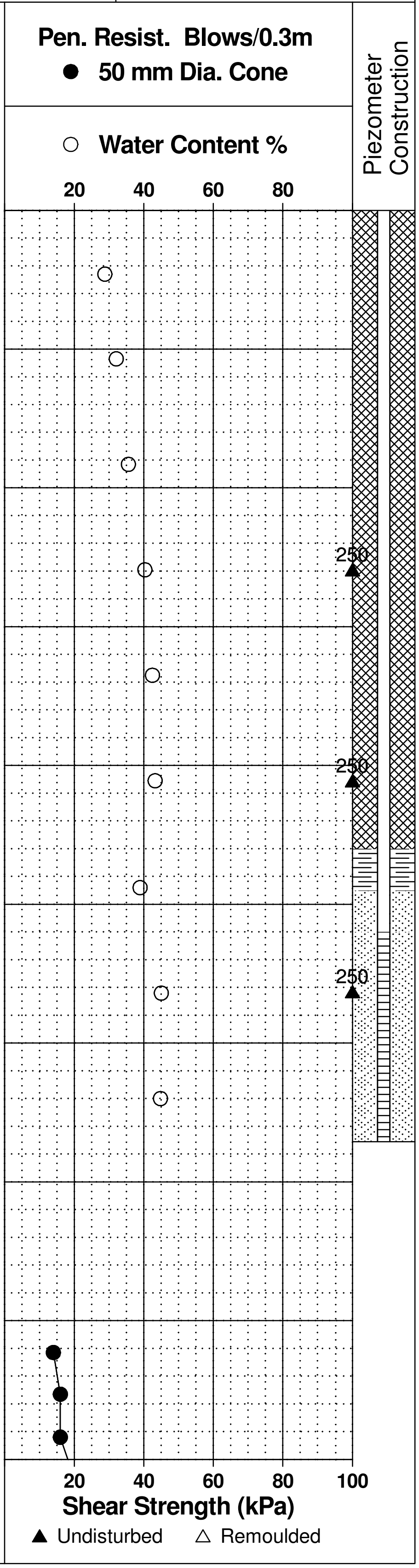
DATE November 29, 2022

FILE NO. PG6260

HOLE NO. BH14-22

SOIL DESCRIPTION	DEPTH BOUNDARY (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.10				
Hard, brown **SILTY CLAY**		AU	1		
		SS	2	100	6
		SS	3	100	7
		SS	4	100	P
		SS	5	100	7
		SS	6	100	P
		SS	7	100	8
		SS	8	100	P
		SS	9	100	8
	6.71				
Dynamic Cone Penetration Test commenced at 6.71m depth.					

DEPTH (m)	ELEV. (m)
0	119.93
1	118.93
2	117.93
3	116.93
4	115.93
5	114.93
6	113.93
7	112.93
8	111.93
9	110.93



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DATUM Geodetic

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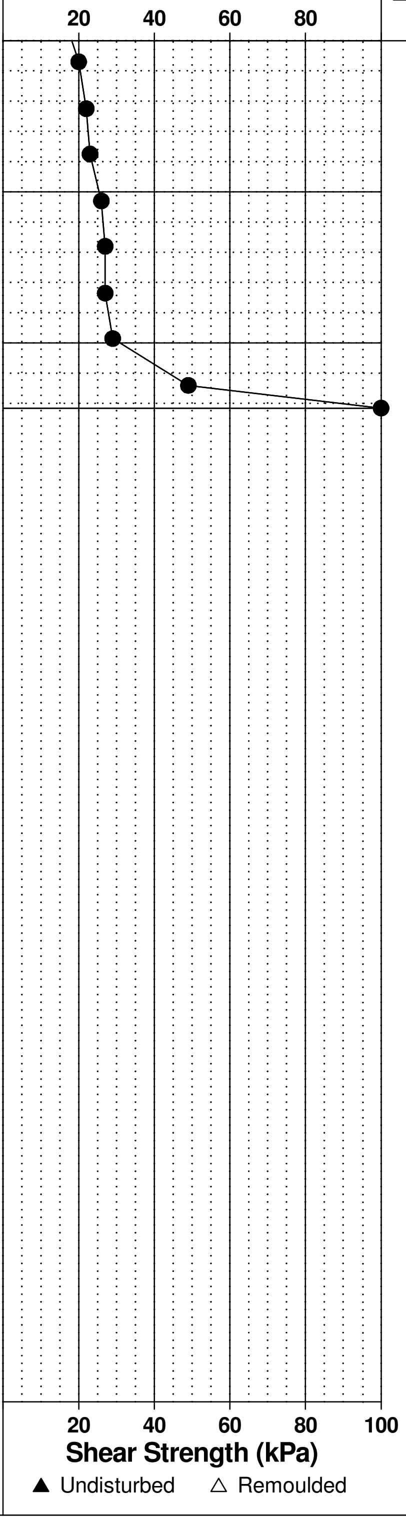
DATE November 29, 2022

FILE NO. PG6260

HOLE NO. BH14-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						9	110.93		
						10	109.93		
						11	108.93		
11.43									
End of Borehole Practical DCPT refusal at 11.43m depth. (BH dry - December 7, 2022)									



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Almonte, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

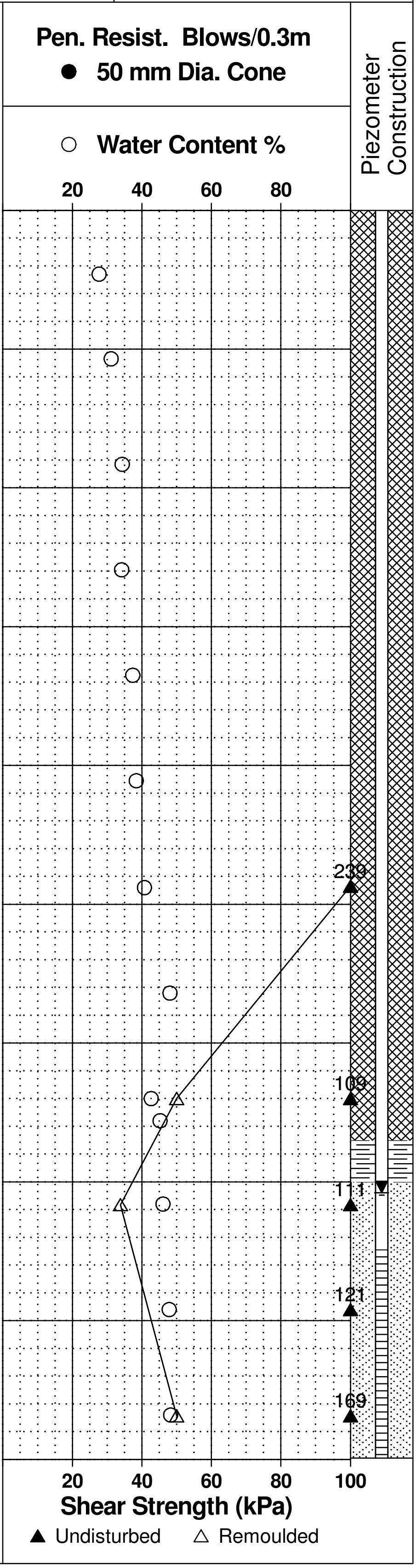
DATE November 29, 2022

FILE NO. PG6260

HOLE NO. BH15-22

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.10				
Hard to very stiff, brown **SILTY CLAY**	AU	1		
	SS	2	100	7
	SS	3	100	6
	SS	4	100	7
	SS	5	100	9
	SS	6	100	8
	SS	7	100	P
	SS	8	100	8
- grey by 6.25m depth	SS	9	100	P
	SS	10	100	P
	SS	11	100	P
8.99	SS	12	100	P

DEPTH (m)	ELEV. (m)
0	117.37
1	116.37
2	115.37
3	114.37
4	113.37
5	112.37
6	111.37
7	110.37
8	109.37
9	108.37



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SOIL PROFILE AND TEST DATA

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Almonte, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE November 29, 2022

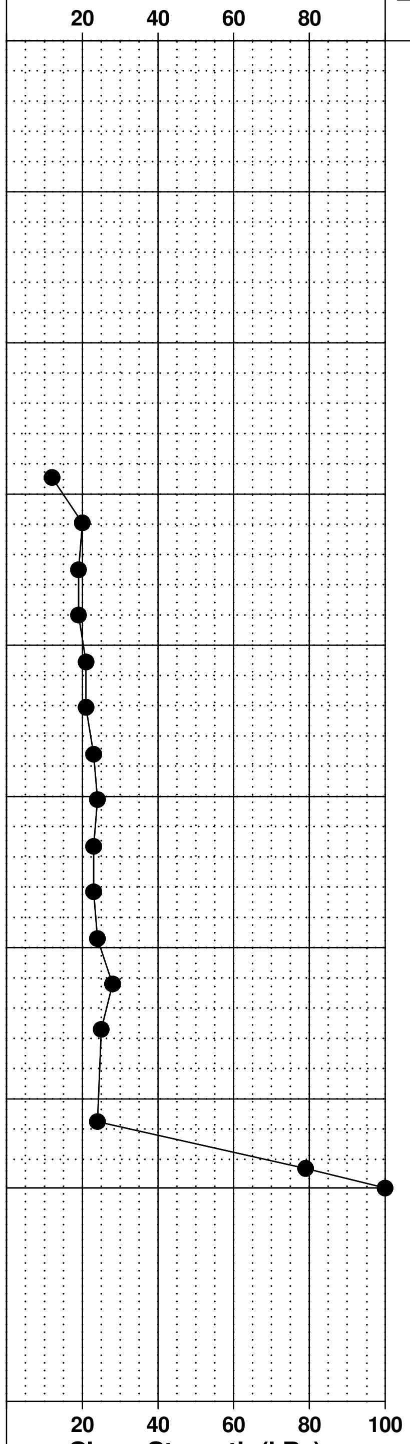
FILE NO. PG6260

HOLE NO. BH15-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Dynamic Cone Penetration Test commenced at 8.99m depth.						9	108.37
						10	107.37
						11	106.37
						12	105.37
						13	104.37
						14	103.37
						15	102.37
						16	101.37
16.59							
End of Borehole Practical DCPT refusal at 16.59m depth. (GWL at 7.08m - Dec. 7, 2022)							

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

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Almonte, Ontario

DATUM Geodetic

REMARKS

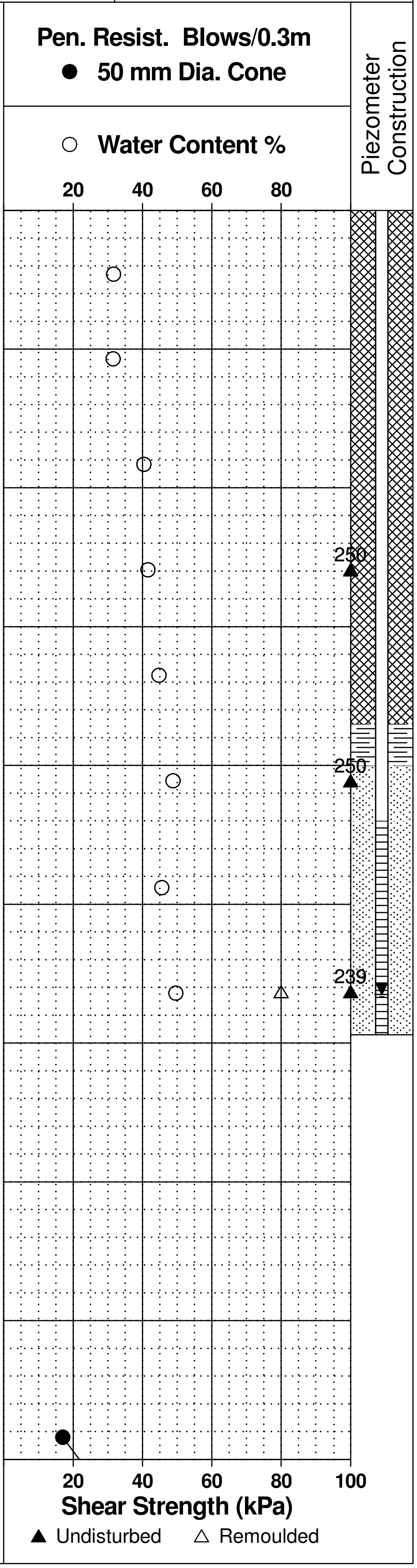
BORINGS BY CME 55 Power Auger

DATE November 30, 2022

FILE NO. PG6260

HOLE NO. BH16-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	115.89
TOPSOIL 0.15							
		AU	1				
		SS	2	100	10	1	114.89
		SS	3	100	12	2	113.89
		SS	4	100	P		
Hard, brown **SILTY CLAY**		SS	5	100	10	3	112.89
		SS	6	100	P	4	111.89
		SS	7	100	10	5	110.89
		SS	8	100	P		
5.94							
Dynamic Cone Penetration Test commenced at 5.94m depth.						6	109.89
						7	108.89
						8	107.89
						9	106.89



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SOIL PROFILE AND TEST DATA

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REMARKS

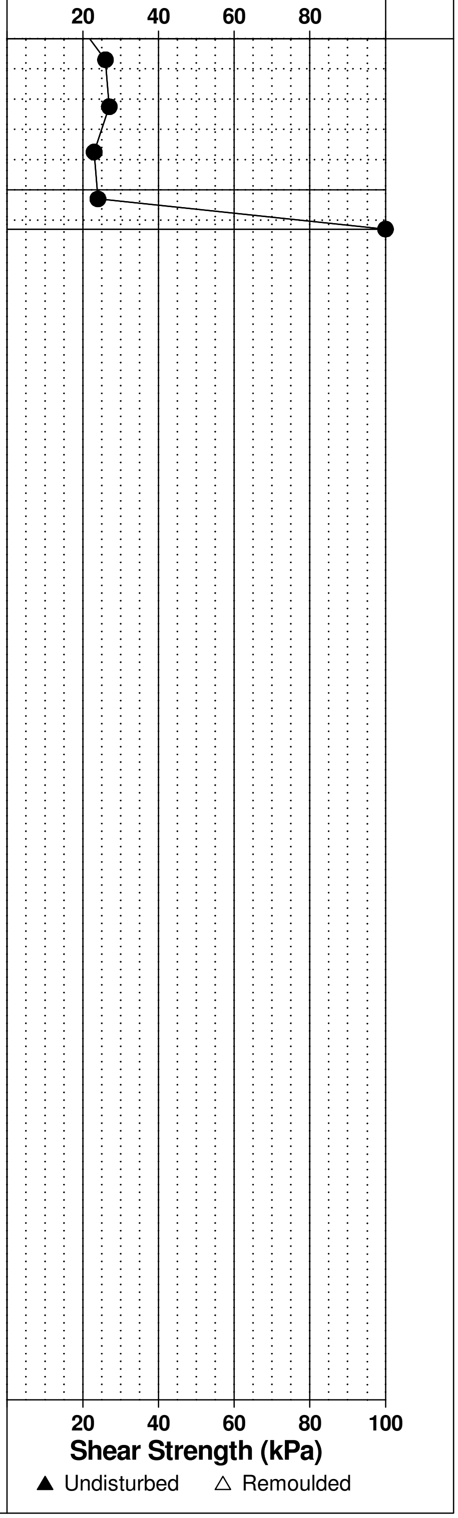
BORINGS BY CME 55 Power Auger

DATE November 30, 2022

FILE NO. PG6260

HOLE NO. BH16-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						9	106.89		
						10	105.89		
10.26 End of Borehole Practical DCPT refusal at 10.26m depth. (GWL at 5.65m - Dec. 7, 2022)									



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REMARKS

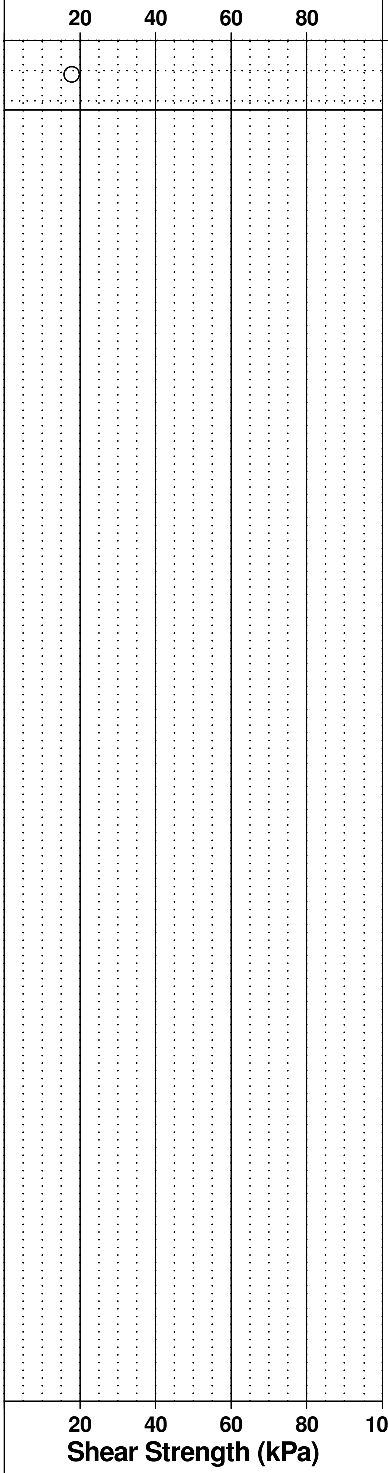
BORINGS BY CME 55 Power Auger

DATE November 30, 2022

FILE NO. PG6260

HOLE NO. PH17-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	111.99
TOPSOIL 0.05		AU	1				
Concrete 0.10							
FILL: Brown silty sand with gravel 0.46							
End of Borehole Practical refusal to augering at 0.46m depth.							



Piezometer Construction

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REMARKS

BORINGS BY CME 55 Power Auger

DATE November 30, 2022

FILE NO. PG6260

HOLE NO. PH18-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE									
TOPSOIL 0.20	■					0	112.94		
End of Borehole Practical refusal to augering at 0.20m depth.									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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REMARKS

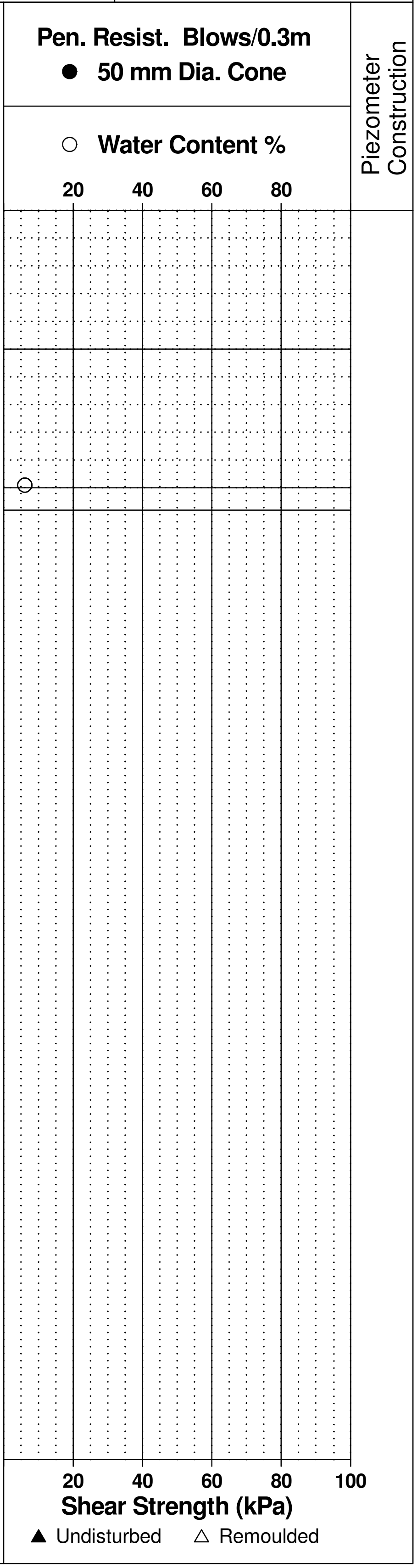
BORINGS BY CME 55 Power Auger

DATE November 30, 2022

FILE NO. PG6260

HOLE NO. PH19-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	110.89
TOPSOIL 0.15							
FILL: Light brown silty sand with gravel and cobbles		AU	1			1	109.89
2.16						2	108.89
End of Borehole Practical refusal to augering at 2.16m depth.							



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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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)

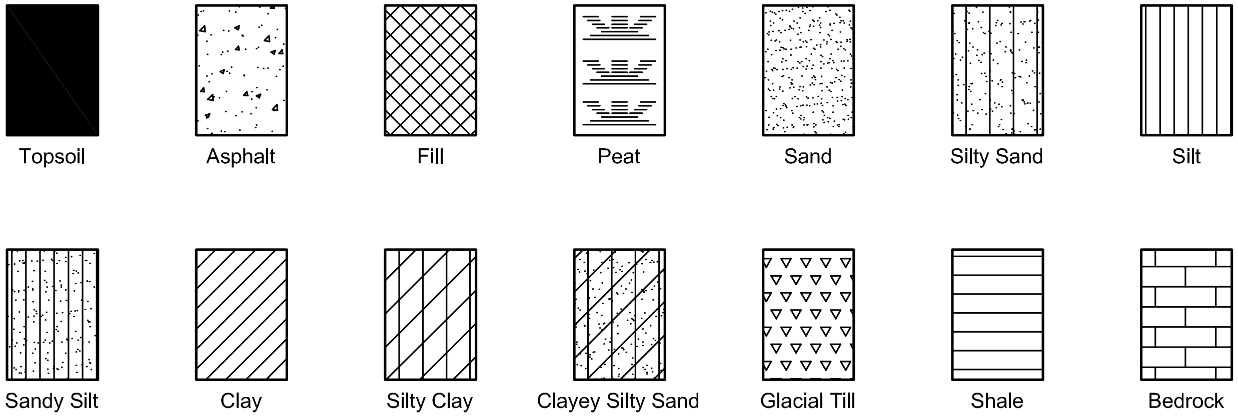
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

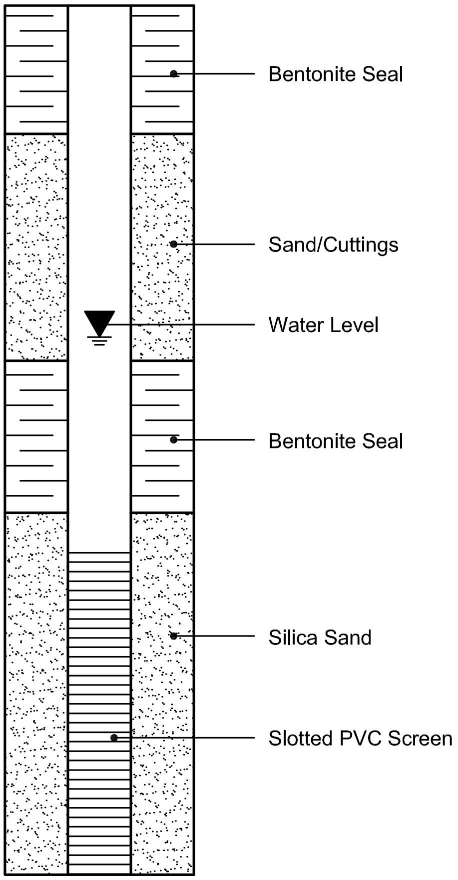
SYMBOLS AND TERMS (continued)

STRATA PLOT

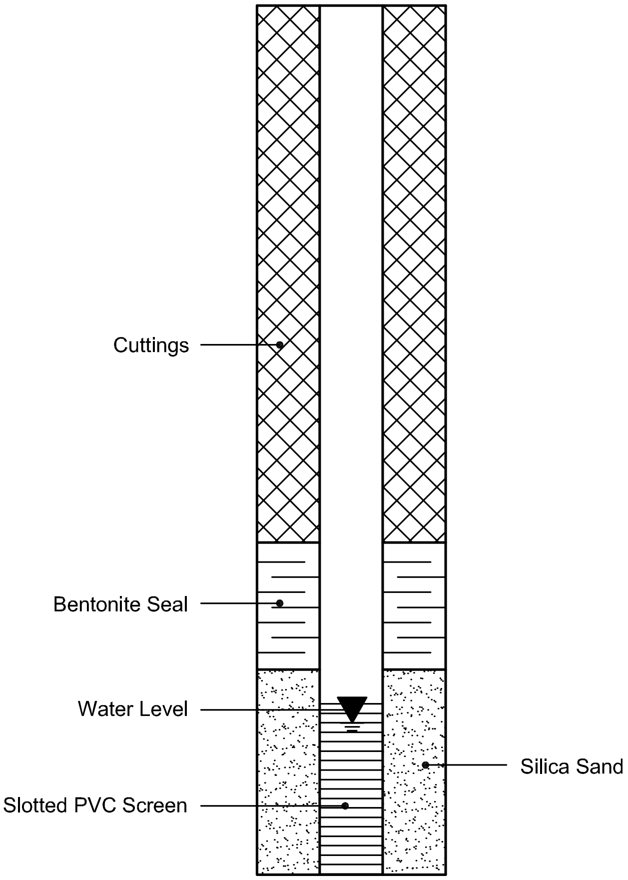


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PATERSON GROUP

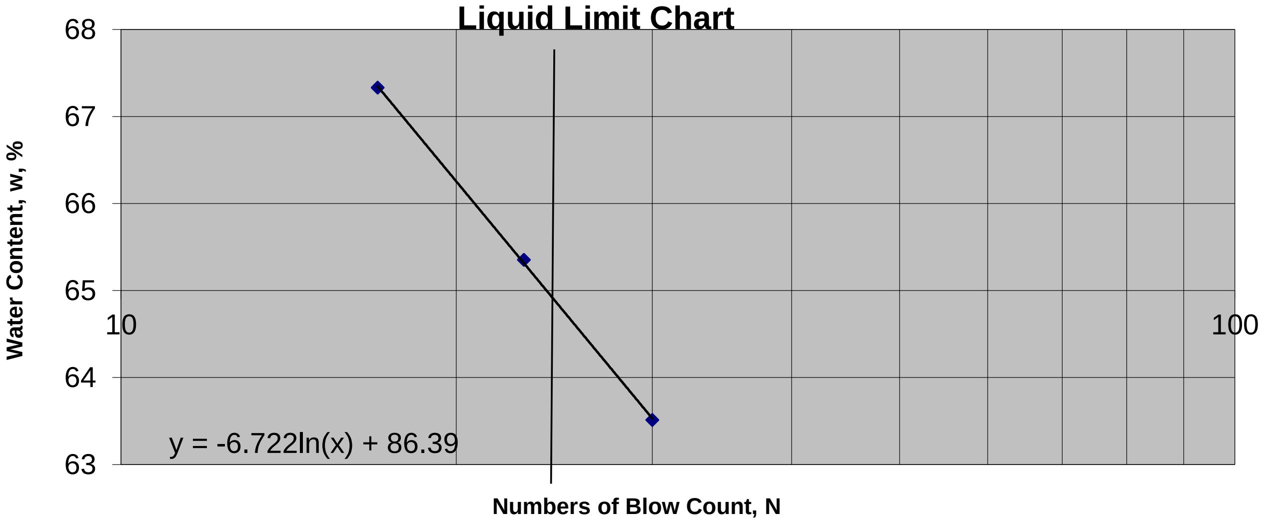
ATTERBERG LIMITS
LS-703/704

CLIENT:	Regional Group	FILE NO.:	PG6260
PROJECT:	County Rd 29 & Strathbury St	DATE SAMPLED:	28-Nov
LOCATION:	BH7-22 SS3	DATE REPORTED:	6-Dec

CAN NO.	**30**	**31**	**32**				
WT. OF CAN	4.37	4.36	4.40				
WT. OF SOIL & CAN	14.41	12.71	13.90				
WT. OF DRY SOIL & CAN	10.37	9.41	10.21				
WT. OF MOISTURE	4.04	3.3	3.69				
WT. OF DRY SOIL & CAN	6	5.05	5.81				
WATER CONTENT, w, %	**67.33**	**65.35**	**63.51**				
NO. OF BLOWS, N	17	23	30				

CAN NO.	**9**	**18**
WT. OF CAN	19.34	20.01
WT. OF SOIL & CAN	27.57	28.67
WT. OF DRY SOIL & CAN	25.43	26.41
WT. OF MOISTURE	2.14	2.26
WT. OF DRY SOIL & CAN	6.09	6.4
WATER CONTENT, w, %	**35.14**	**35.31**

RESULTS	
LIQUID LIMIT	**65**
PLASTIC LIMIT	**35**
PLASTICITY INDEX	**30**



TECHNICIAN: CS	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

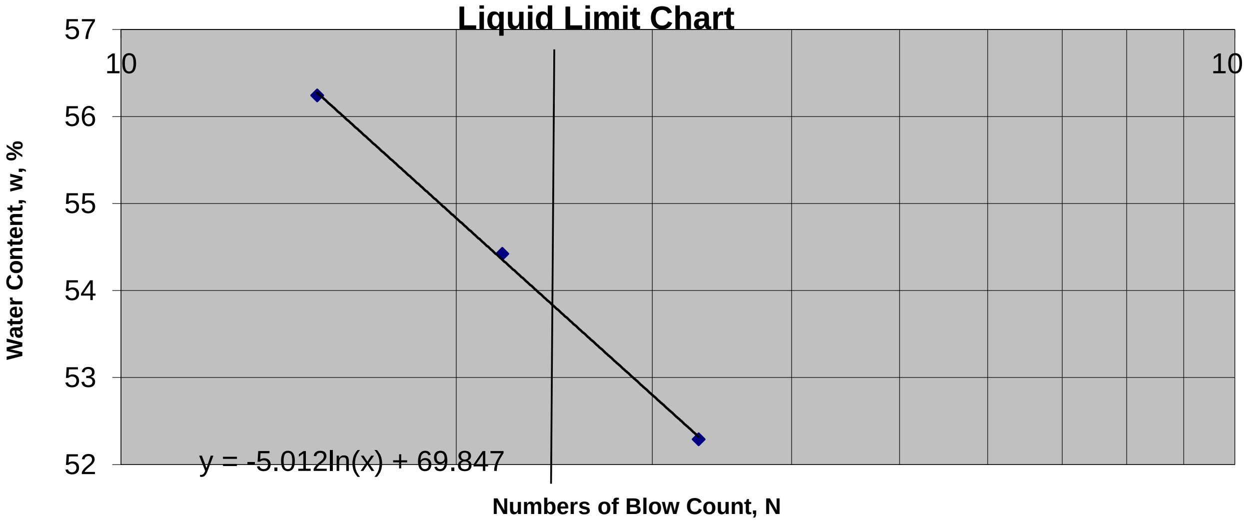
ATTERBERG LIMITS
LS-703/704

CLIENT:	Regional Group	FILE NO.:	PG6260
PROJECT:	County Rd 29 & Strathbury St	DATE SAMPLED:	28-Nov
LOCATION:	BH12-22 SS3	DATE REPORTED:	6-Dec

CAN NO.	**33**	**18**	**35**				
WT. OF CAN	4.37	8.69	4.38				
WT. OF SOIL & CAN	14.26	16.55	13.35				
WT. OF DRY SOIL & CAN	10.70	13.78	10.27				
WT. OF MOISTURE	3.56	2.77	3.08				
WT. OF DRY SOIL & CAN	6.33	5.09	5.89				
WATER CONTENT, w, %	**56.24**	**54.42**	**52.29**				
NO. OF BLOWS, N	15	22	33				

CAN NO.	**3**	**13**
WT. OF CAN	19.39	19.28
WT. OF SOIL & CAN	27.29	28.28
WT. OF DRY SOIL & CAN	25.57	26.31
WT. OF MOISTURE	1.72	1.97
WT. OF DRY SOIL & CAN	6.18	7.03
WATER CONTENT, w, %	**27.83**	**28.02**

RESULTS	
LIQUID LIMIT	**54**
PLASTIC LIMIT	**28**
PLASTICITY INDEX	**26**



TECHNICIAN: CS	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

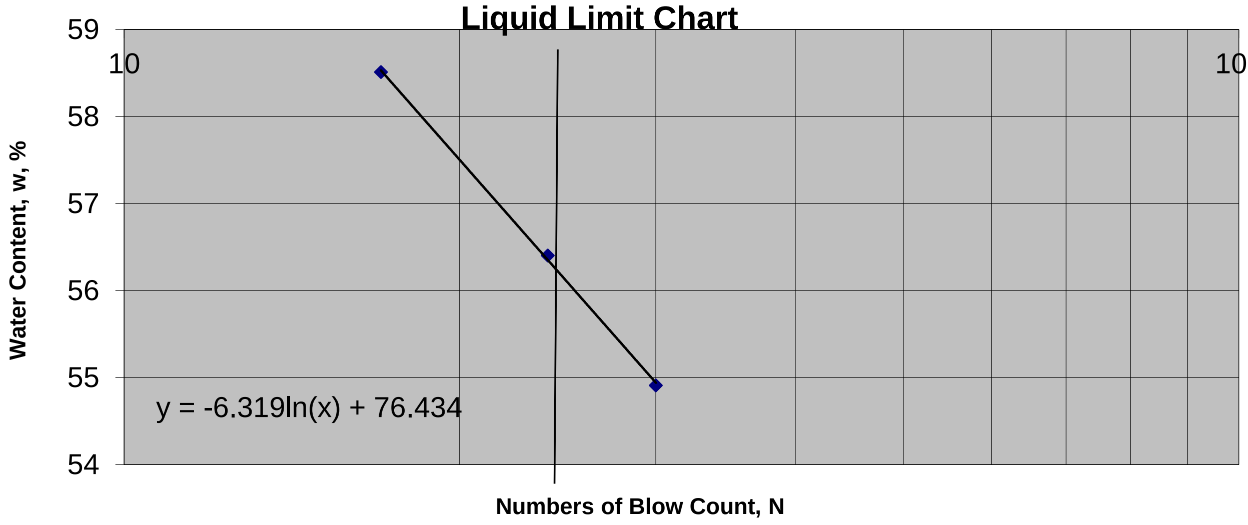
ATTERBERG LIMITS
LS-703/704

CLIENT:	Regional Group	FILE NO.:	PG6260
PROJECT:	County Rd 29 & Strathbury St	DATE SAMPLED:	28-Nov
LOCATION:	BH16-22 SS3	DATE REPORTED:	6-Dec

CAN NO.	**2**	**3**	**4**				
WT. OF CAN	8.69	8.70	8.71				
WT. OF SOIL & CAN	19.77	17.13	19.43				
WT. OF DRY SOIL & CAN	15.68	14.09	15.63				
WT. OF MOISTURE	4.09	3.04	3.80				
WT. OF DRY SOIL & CAN	6.99	5.39	6.92				
WATER CONTENT, w, %	**58.51**	**56.4**	**54.91**				
NO. OF BLOWS, N	17	24	30				

CAN NO.	**15**	**12**
WT. OF CAN	19.91	16.73
WT. OF SOIL & CAN	27.98	25.05
WT. OF DRY SOIL & CAN	26.19	23.21
WT. OF MOISTURE	1.79	1.84
WT. OF DRY SOIL & CAN	6.28	6.48
WATER CONTENT, w, %	**28.5**	**28.4**

RESULTS	
LIQUID LIMIT	**57**
PLASTIC LIMIT	**28**
PLASTICITY INDEX	**29**

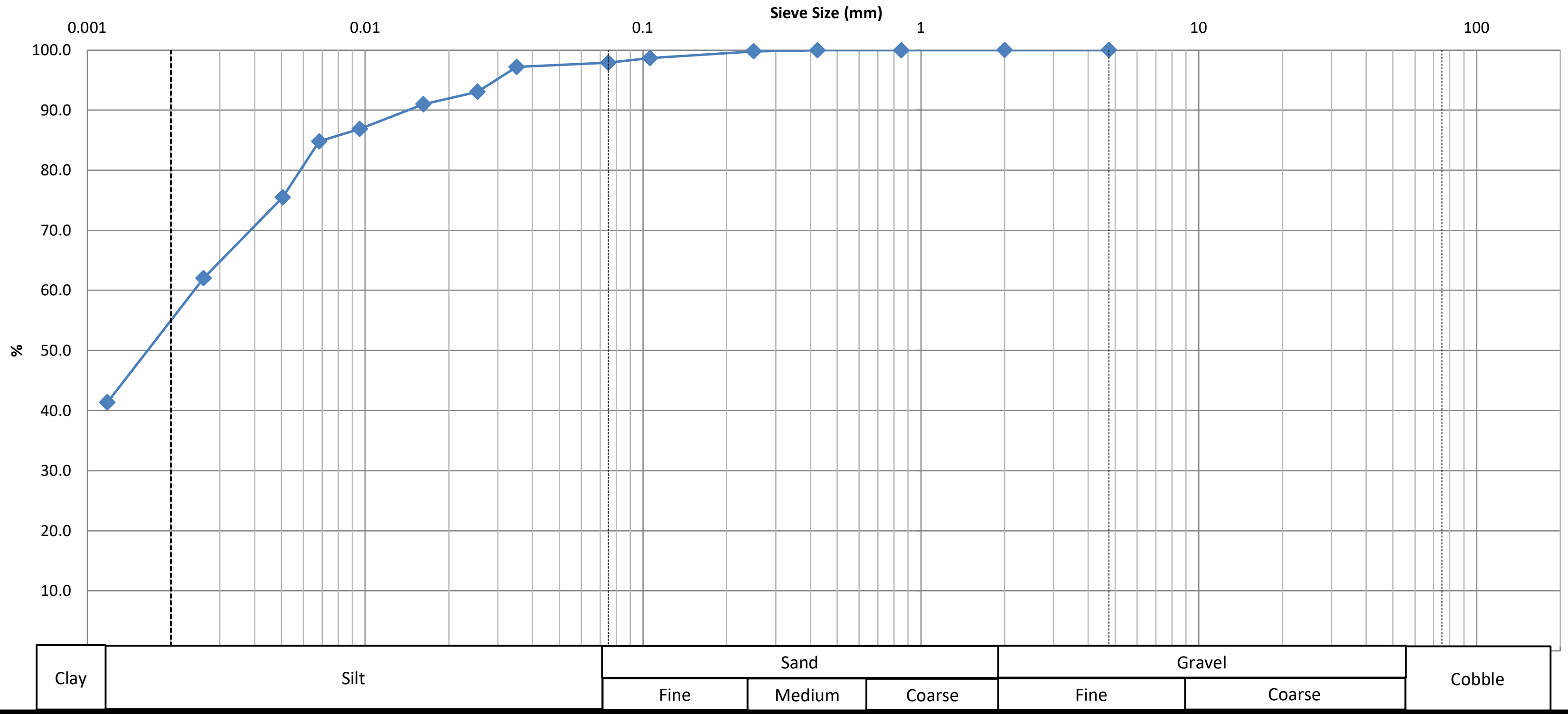


TECHNICIAN: CS	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Regional Group (Novatech)	DEPTH:	5.0' - 7.0'	FILE NO:	PG6260
CONTRACT NO.:		BH OR TP No.:	BH13-22 SS3	LAB NO:	41375
PROJECT:	County Road 29 & Strathbury Street			DATE RECEIVED:	1-Dec-22
				DATE TESTED:	6-Dec-22
DATE SAMPLED:	Nov 28-29			DATE REPORTED:	13-Dec-22
SAMPLED BY:	JP			TESTED BY:	DK/CS



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		38.6					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				0.0	2.1	43.9	54.0

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

HYDROMETER
LS-702 ASTM-422

CLIENT:	Regional Group (Novatech)	DEPTH:	5.0' - 7.0'	FILE NO.:	PG6260
PROJECT:	County Road 29 & Strathbury Street	BH OR TP No.:	BH13-22 SS3	DATE SAMPLE	Nov 28-29
LAB No. :	41375	TESTED BY:	DK/CS	DATE RECEIVE	1-Dec-22
SAMPLED BY:	JP	DATE REPT'D:	13-Dec-22	DATE TESTED:	6-Dec-22

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
108.3	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	42.19	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	1.09	AIR DRY	119.10	69.10
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	108.30	58.30
		CORRECTED	0.844	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**0.0**	0.0	**100.0**
2.0	**0.0**	0.0	**100.0**
Pan	**108.3**		
0.850	**0.01**	0.0	**100.0**
0.425	**0.02**	0.0	**100.0**
0.250	**0.09**	0.2	**99.8**
0.106	**0.66**	1.3	**98.7**
0.075	**1.05**	2.1	**97.9**
Pan	**1.09**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	7:31	53.0	6.0	23.0	0.0351	97.2	**97.2**
2	7:32	51.0	6.0	23.0	0.0254	93.1	**93.1**
5	7:35	50.0	6.0	23.0	0.0162	91.0	**91.0**
15	7:45	48.0	6.0	23.0	0.0096	86.9	**86.9**
30	8:00	47.0	6.0	23.0	0.0068	84.8	**84.8**
60	8:30	42.5	6.0	23.0	0.0051	75.5	**75.5**
250	11:40	36.0	6.0	23.0	0.0026	62.0	**62.0**
1440	7:30	26.0	6.0	23.0	0.0012	41.4	**41.4**

Moisture = 38.6%

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

Linear Shrinkage
ASTM D4943-02

CLIENT:	Regional Group (Novatech)	DEPTH	5.0' - 7.0'	FILE NO.:	PG6260
PROJECT:	County Road 29 & Strathbury Street	BH OR TP No:	BH14-22 SS3	DATE SAMPLED	Nov 28-29
LAB No:	41376	TESTED BY:	CP/CS	DATE RECEIVED	1-Dec
SAMPLED BY:	JP	DATE REPORTED:	3-Dec-22	DATE TESTED	6-Dec

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(5)	Calibration (Two Trials)	Tin NO.(x22)	
Tare	5.13	Tin	4.86	4.86
Soil Pat Wet + Tare	67.96	Tin + Grease	5.74	5.13
Soil Pat Wet	62.83	Glass	48.97	48.97
Soil Pat Dry + Tare	45.55	Tin + Glass + Water	91.57	91.5
Soil Pat Dry	40.42	Volume	36.86	37.4
Moisture	**55.44**	**Average Volume**	**37.13**	

Soil Pat + String	40.68
Soil Pat + Wax + String in Air	46.31
Soil Pat + Wax + String in Water	16.43
Volume Of Pat (Vdx)	29.88

RESULTS:

Shrinkage Limit	**21.86**
Shrinkage Ratio	**1.716**
Volumetric Shrinkage	**57.637**
Linear Shrinkage	**14.075**

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 54720

Report Date: 27-May-2022

Order Date: 20-May-2022

Project Description: PG6260

	Client ID:	BH2-22 SS3	-	-	-
	Sample Date:	19-May-22 09:00	-	-	-
	Sample ID:	2221637-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.06	-	-	-
Resistivity	0.10 Ohm.m	87.1	-	-	-
Anions					
Chloride	5 ug/g dry	7	-	-	-
Sulphate	5 ug/g dry	23	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2A TO 4B – SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG6260-1 - TEST HOLE LOCATION PLAN

DRAWING PG6260-2 – PERMISSIBLE GRADE RAISE RESTIRCTION PLAN

DRAWING PG6260-3 – TREE PLANTING SETBACK PLAN

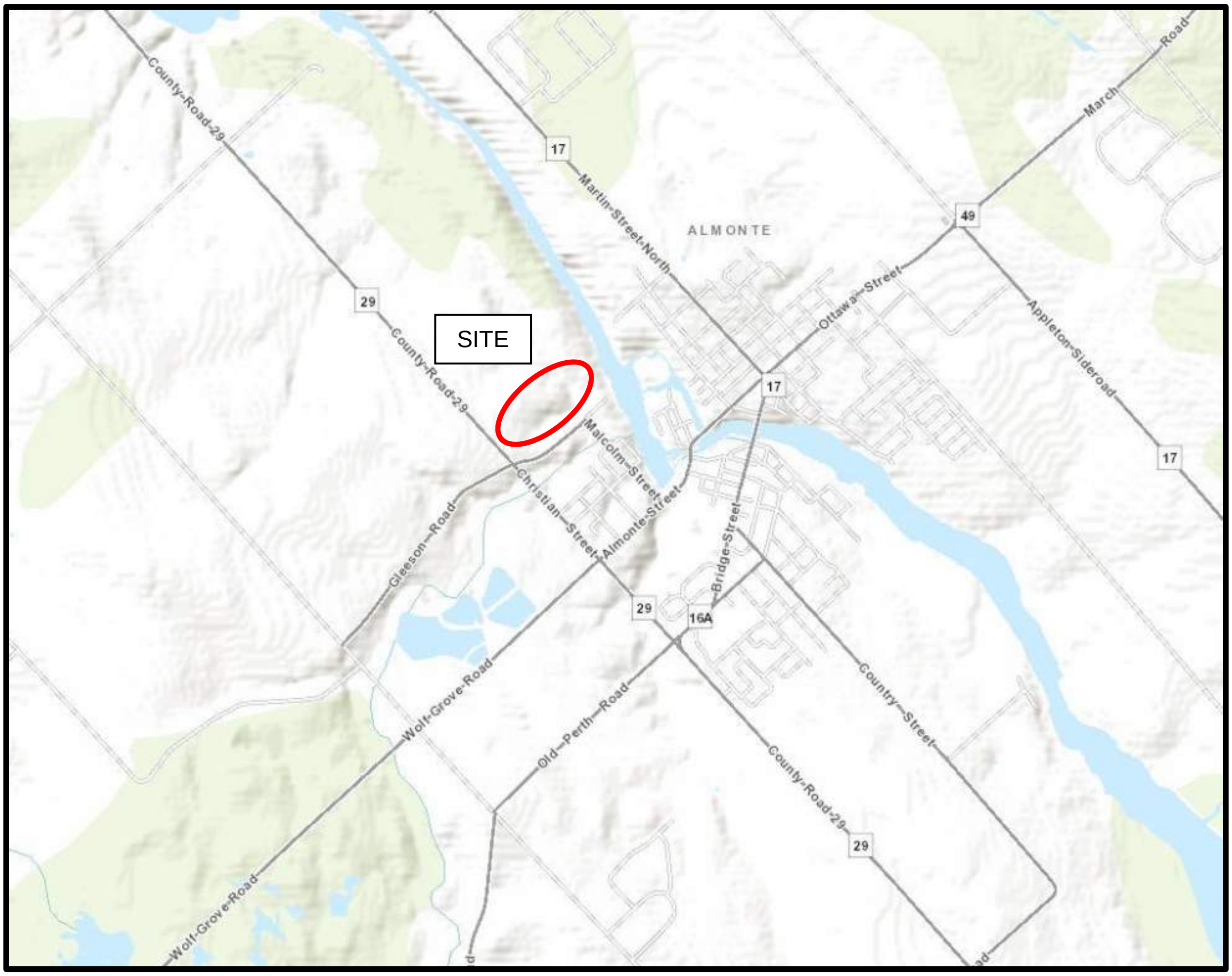


FIGURE 1

KEY PLAN

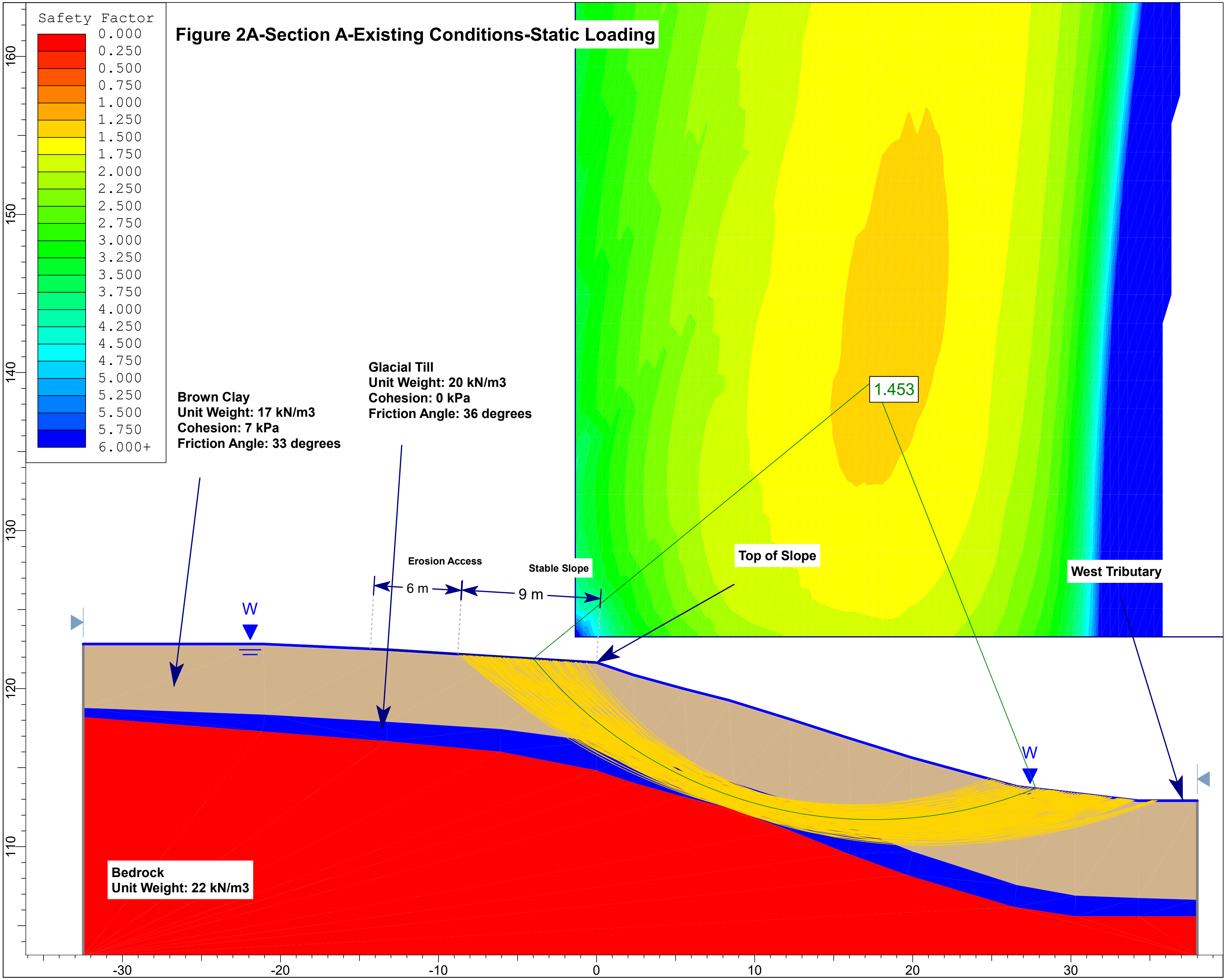


Figure 2A-Section A-Existing Conditions-Static Loading

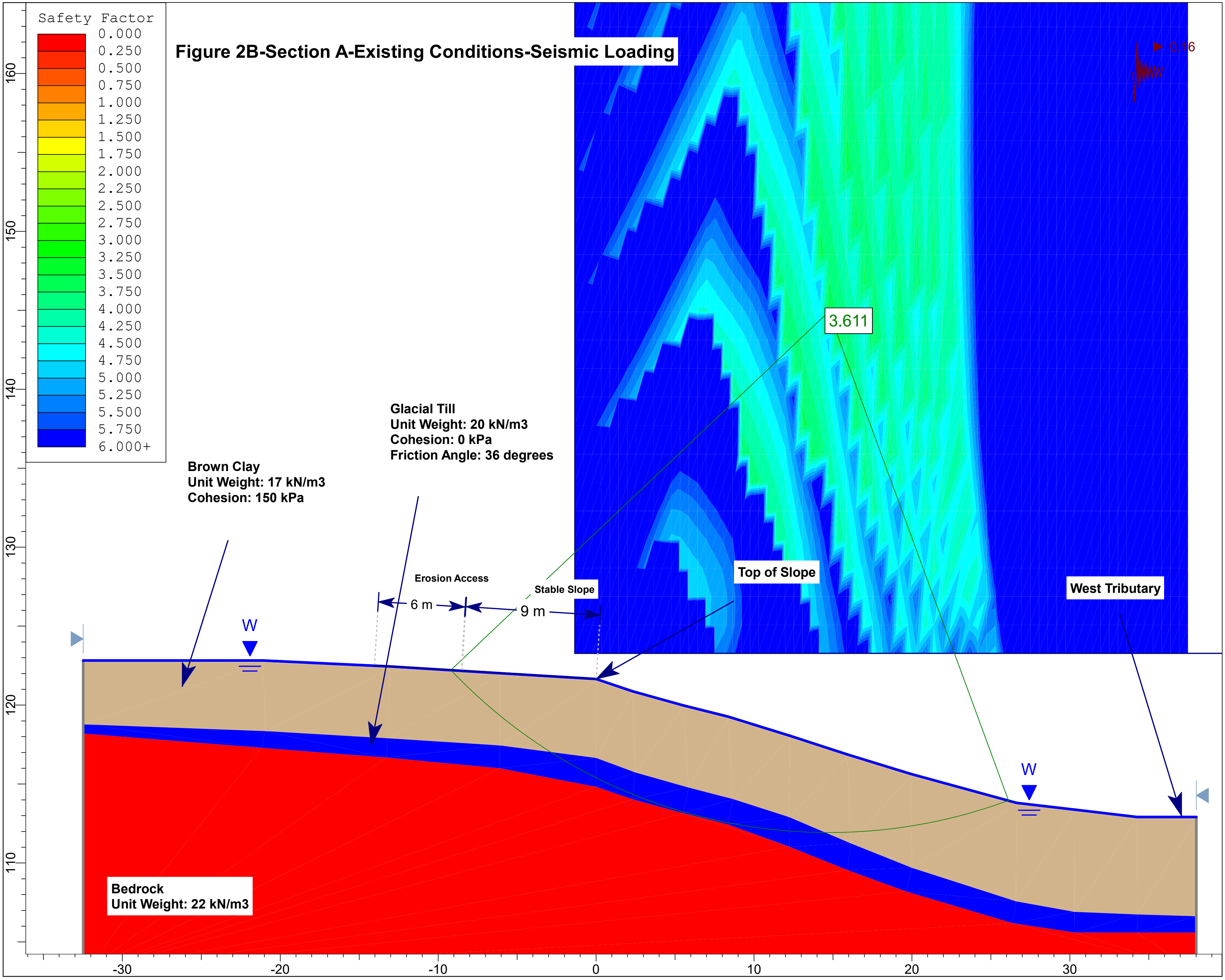


Figure 2B-Section A-Existing Conditions-Seismic Loading

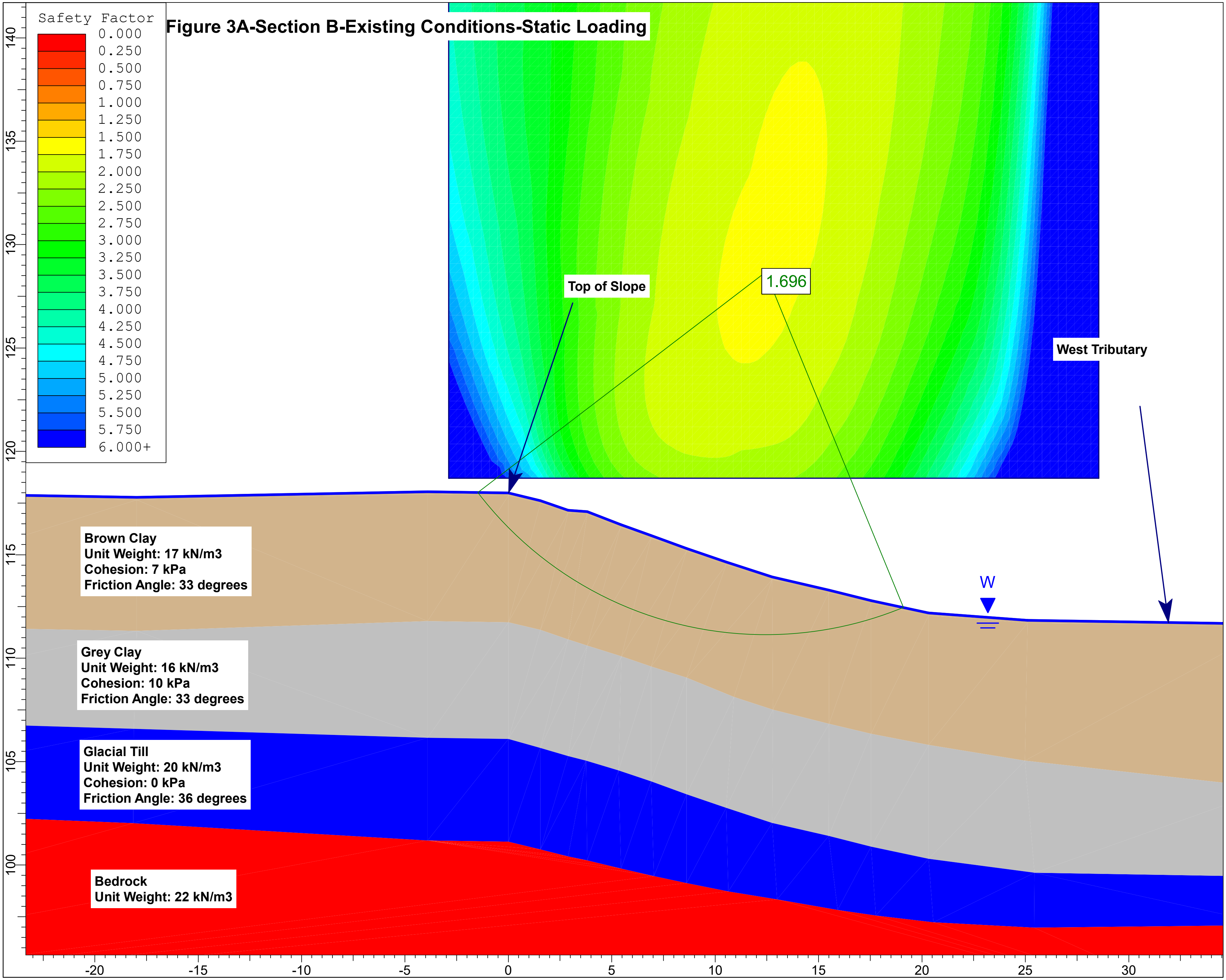


Figure 3A-Section B-Existing Conditions-Static Loading

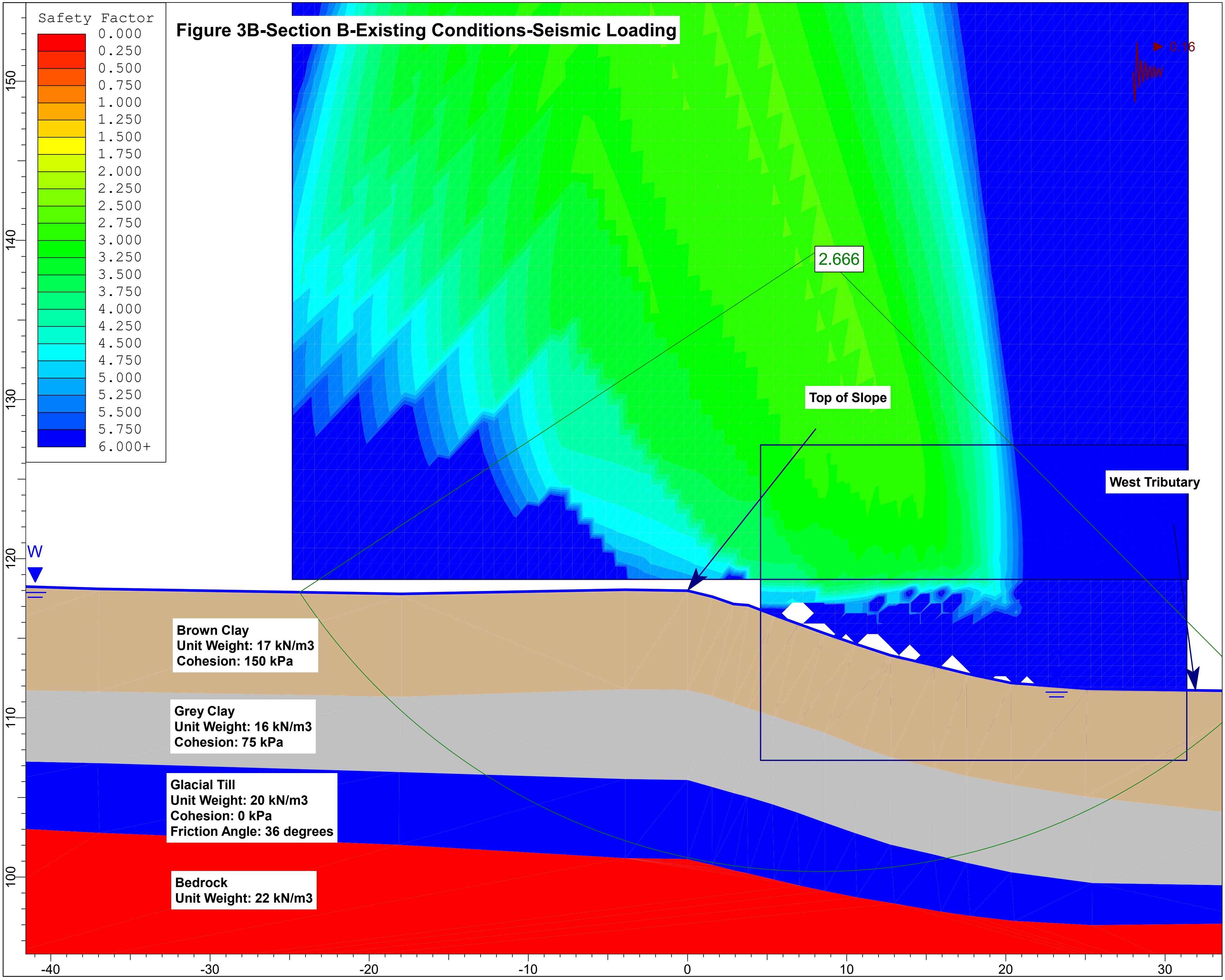


Figure 3B-Section B-Existing Conditions-Seismic Loading

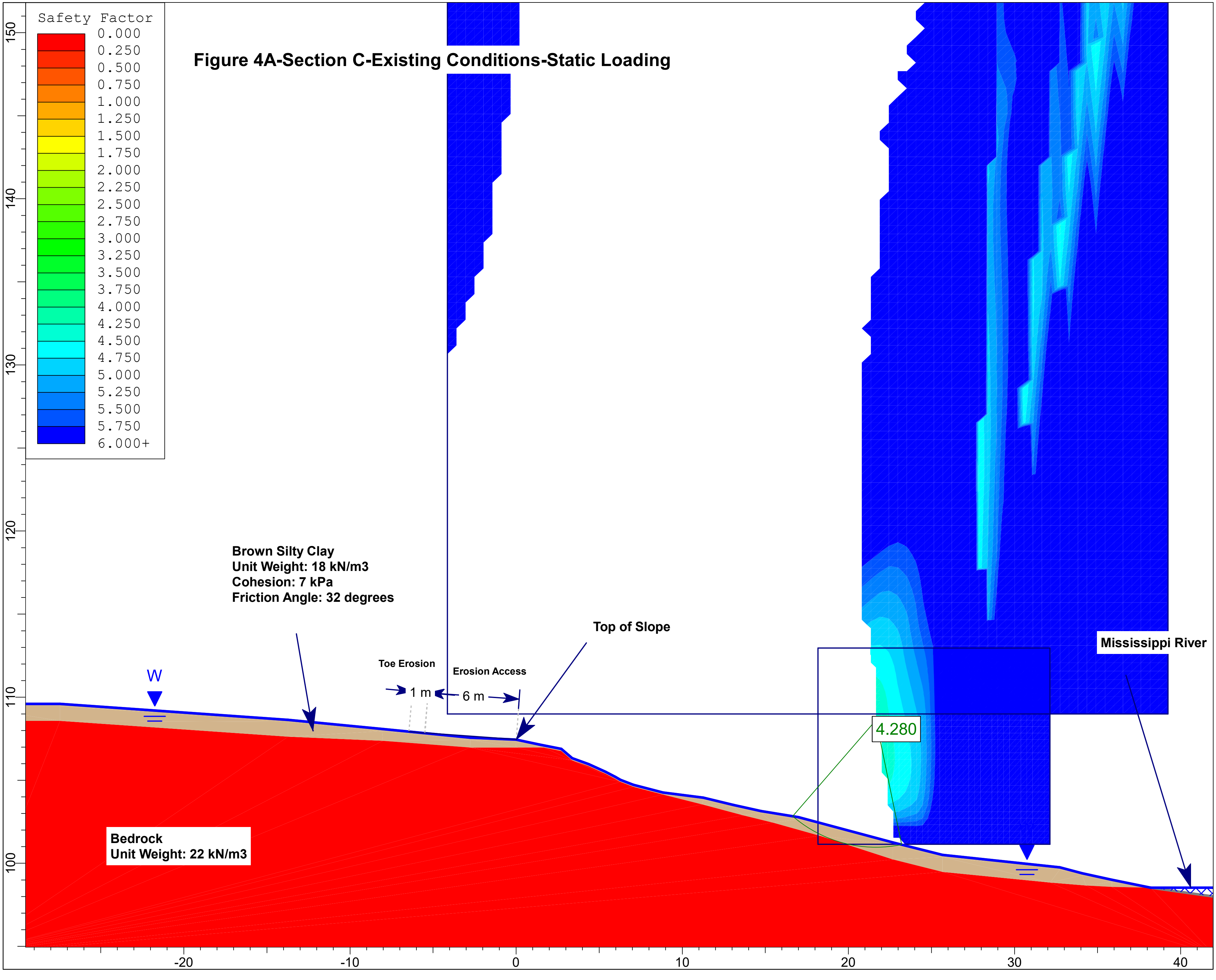


Figure 4A-Section C-Existing Conditions-Static Loading

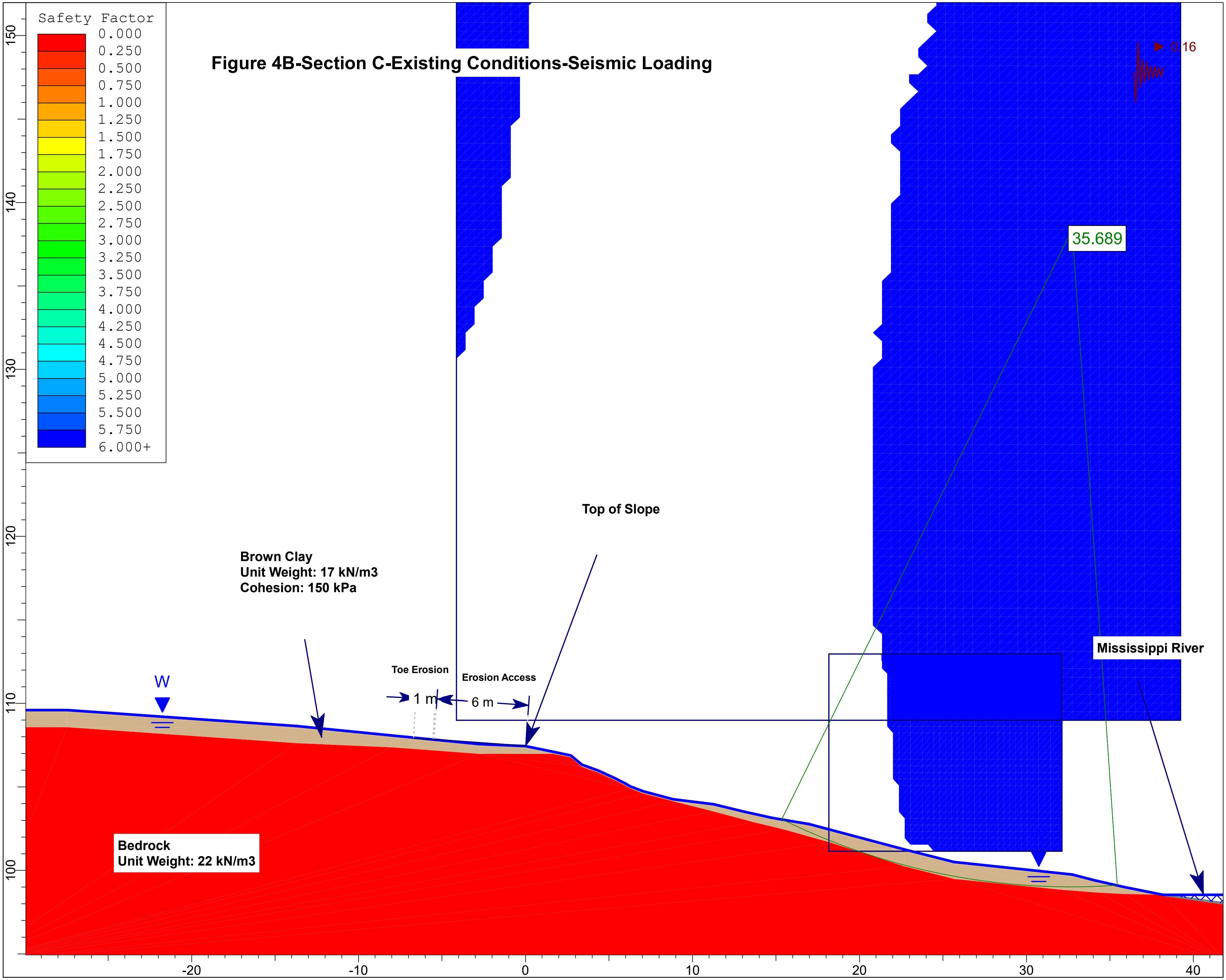
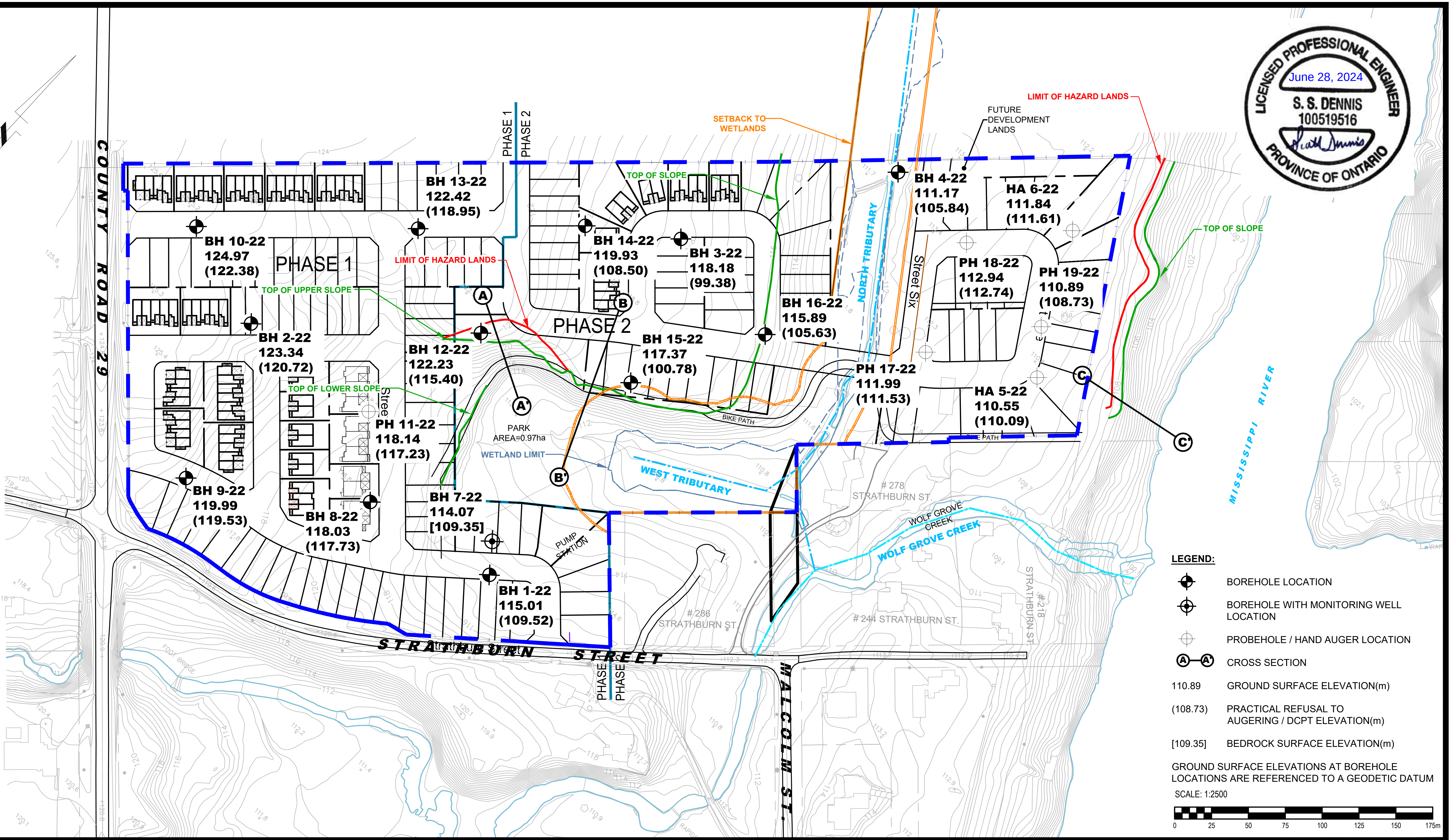


Figure 4B-Section C-Existing Conditions-Seismic Loading



June 28, 2024
S. S. DENNIS
100519516

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	PROBEHOLE / HAND AUGER LOCATION
Ⓐ—Ⓐ'	CROSS SECTION
110.89	GROUND SURFACE ELEVATION(m)
(108.73)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION(m)
[109.35]	BEDROCK SURFACE ELEVATION(m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:2500

0 25 50 75 100 125 150 175m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
3	UPDATED CONCEPTUAL PLAN TO DRAWING	28/06/2024	SD
2	UPDATED SETBACK TO WETLANDS ADDED TO DRAWING	24/01/2023	DP
1	UPDATED CONCEPTUAL PLAN TO DRAWING	10/01/2023	DP

STRATHBURN ALMONTE REGIONAL INC.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - BROWN PROPERTIES
COUNTY ROAD NO. 29 AND STRATHBURN STREET

ALMONTE, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2500	Date: 01/2023
Drawn by: JM	Report No.: PG6260-1
Checked by: NS	Dwg. No.: PG6260-1
Approved by: DP	Revision No.: 3

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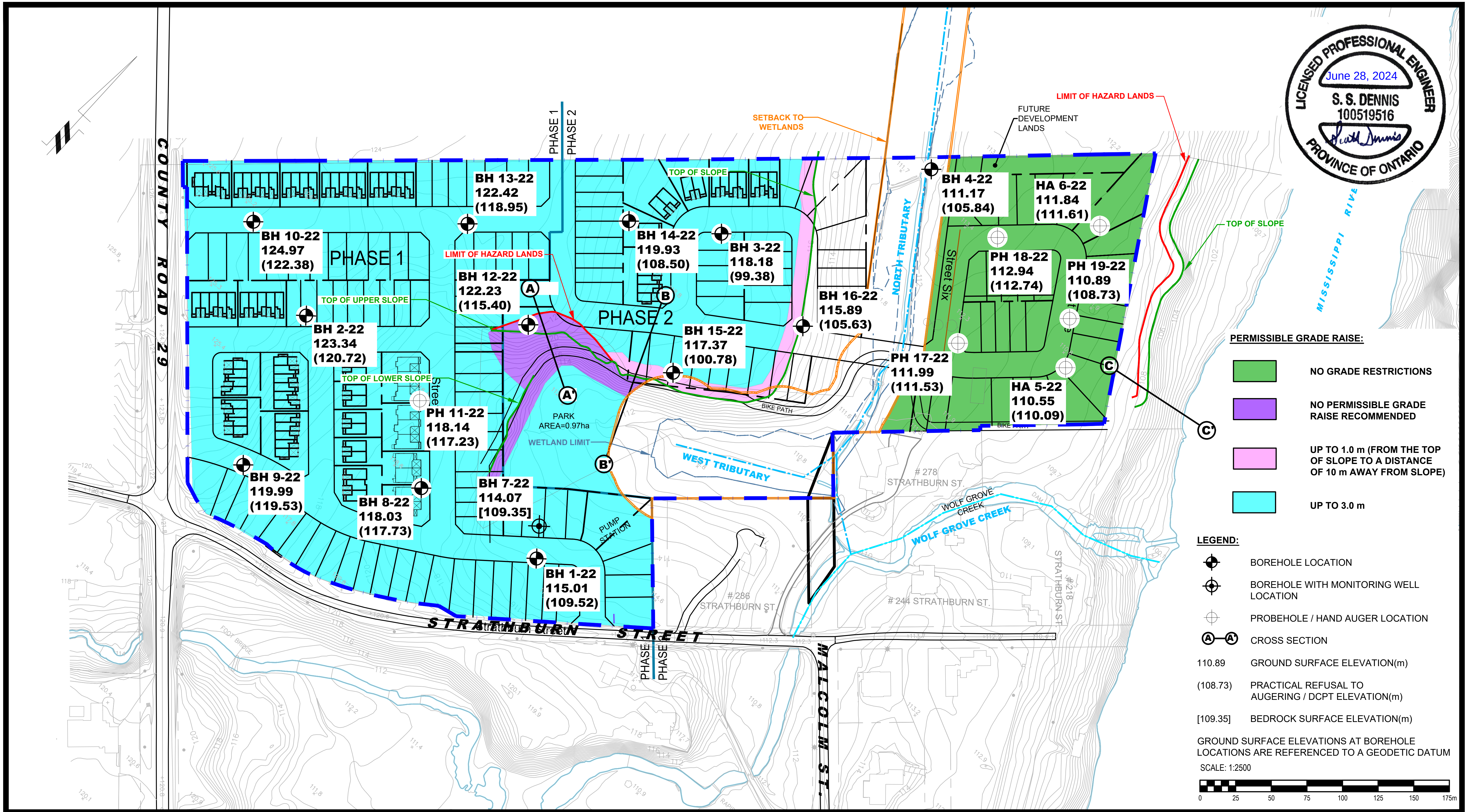

June 28, 2024
S. S. DENNIS
100519516

PERMISSIBLE GRADE RAISE:

🟩	NO GRADE RESTRICTIONS
🟪	NO PERMISSIBLE GRADE RAISE RECOMMENDED
🟪	UP TO 1.0 m (FROM THE TOP OF SLOPE TO A DISTANCE OF 10 m AWAY FROM SLOPE)
🟦	UP TO 3.0 m

LEGEND:

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⊕	BOREHOLE WITH MONITORING WELL LOCATION
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0 25 50 75 100 125 150 175m

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STRATHBURN ALMONTE REGIONAL INC.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - BROWN PROPERTIES
COUNTY ROAD NO. 29 AND STRATHBURN STREET

ALMONTE, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:2500	Date: 01/2023
Drawn by: JM	Report No.: PG6260-2
Checked by: NS	Dwg. No.: PG6260-2
Approved by: DP	Revision No.: 3

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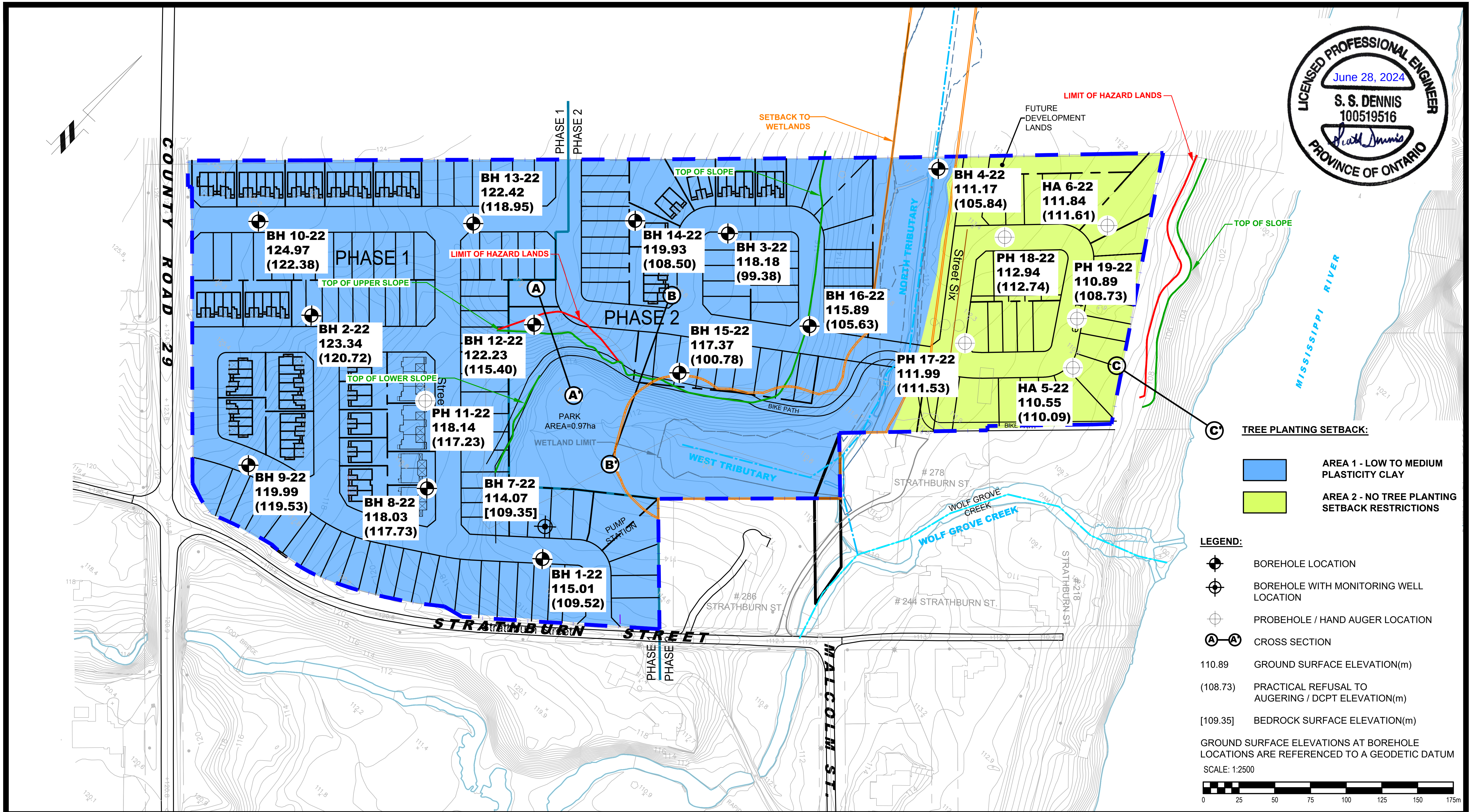

LICENSED PROFESSIONAL ENGINEER
June 28, 2024
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

TREE PLANTING SETBACK:

🟦	AREA 1 - LOW TO MEDIUM PLASTICITY CLAY
🟩	AREA 2 - NO TREE PLANTING SETBACK RESTRICTIONS

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	PROBEHOLE / HAND AUGER LOCATION
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GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:2500

0 25 50 75 100 125 150 175m

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1	UPDATED CONCEPTUAL PLAN TO DRAWING	10/01/2023	DP

STRATHBURN ALMONTE REGIONAL INC.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - BROWN PROPERTIES
COUNTY ROAD NO. 29 AND STRATHBURN STREET

ALMONTE, ONTARIO

Title:

TREE PLANTING SETBACK PLAN

Scale: 1:2500	Date: 01/2023
Drawn by: JM	Report No.: PG6260-2
Checked by: NS	Dwg. No.: **PG6260-3**
Approved by: DP	Revision No.: 3

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