Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

patersongroup

Geotechnical Investigation

National Capital Business Park
4055 & 4120 Russell Road
Ottawa, Ontario

Prepared For

Avenue 31

March 18, 2020

Report: PG4854-1
Revision 3

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Avenue 31 to conduct a geotechnical investigation for the proposed National Capital Business Park to be located at 4055 and 4120 Russell Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the investigation was to:

- ❑ determine the subsoil and groundwater conditions at this site by means of boreholes.

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

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Based on the available drawings, the proposed development at the subject site will consist of 6 commercial structures (Buildings A through F) with footprints ranging from approximately 1,500 m^2 to 65,000 m^2. The proposed buildings will be surrounded by asphalt paved access lanes and parking areas with landscaped margins.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was conducted during the period of August 28 through September 4, 2019. At that time, a total of 16 boreholes were advanced to a maximum depth of 10.2 m. The boreholes were distributed in a manner to provide general coverage of the subject site taking into consideration existing site features and underground utilities. The locations of the test holes are shown on Drawing PG4854-1 - 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 test hole procedures consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at BH 10. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the 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 BH 1, BH 3, BH 6, BH 9, BH 13, BH 14, and BH 15, and flexible piezometers were installed in the remaining boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site 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 Annis O'Sullivan Vollebekk, Ltd. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG4854-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the boreholes and visually examined in our laboratory to review the field logs.

3.4 Analytical Testing

Two (2) soil samples were submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of 2 properties, 4055 and 4120 Russell Road, which are located to the east and west of Russell Road, respectively, and have a combined footprint of approximately 100 acres. The subject site is bordered by the Highway 417 to the east, Hunt Club Road to the South, commercial properties to the west and northwest, and an electrical substation to the northeast.

The site consists mostly of agricultural lands with a farm house, barn, and grain silos fronting onto the east side of Russell Road. A drainage channel also runs approximately east-west through the southeast portion of the site. The existing ground surface across the site is generally level at approximate geodetic elevation 69 to 70 m, with the exception of the southwest portion of the site which slopes up to geodetic elevation 79 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of a 100 to 250 mm thickness of topsoil underlain by 0.3 to 3 m of loose to compact, brown silty sand. However, in the southwestern portion of the site, the topsoil was generally underlain by fill extending to approximate depths of 0.8 to 1.5 m. The fill was observed to vary from a silty sand to silty clay with some gravel.

A silty clay deposit was generally encountered underlying the silty sand and/or fill, extending to depths varying from 2.1 m in the central portion of the site, to 9.5 m at the north end of the site, and 5.6 m in the southwest portion of the site. It should also be noted that the silty clay deposit was not encountered in BH 3 and BH 4. The silty clay generally had an upper crust consisting of a hard to stiff, brown silty clay, becoming a stiff to firm, grey silty clay at approximate depths of 1.5 to 5 m.

A glacial till deposit was generally encountered underlying the silty clay, consisting of a compact to dense, grey silty clay with sand, gravel, cobbles, and boulders.

Bedrock

Practical refusal of the augers or DCPT were encountered at depths ranging from 2.3 m in the central portion of the site, to 10.3 m at the north end of the site, and 6.2 m at the southwest end of the site.

Based on available geological mapping, bedrock in the area of the subject site consists of shale of the Carlsbad Formation with overburden drift thicknesses between 2 to 10 m depth.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells on September 18, 2019 and in the standpipes on September 27, 2019. The measured groundwater level (GWL) readings are presented in Table 1 below and in the Soil Profile and Test Data sheets in Appendix 1.

It is important to note that groundwater readings at the piezometers can be influenced by water perched within the borehole backfill material. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on the observations of the soil samples, the long-term groundwater table can be expected between 1.5 to 3 m depth across the majority of the site, with the exception of the southwest end of the site where the long-term groundwater table can be expected between 4.5 to 5.5 m depth.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Measured Groundwater Levels

Test Hole Location	Ground Surface	Groundwater Level		Date
		Depth (m)	Elevation (m)	
BH 1*	69.48	2.48	67.00	September 18, 2019
BH 2	69.43	Inaccessible	-	September 27, 2019
BH 3*	71.57	3.87	67.70	September 18, 2019
BH 4	67.49	Inaccessible	-	September 27, 2019
BH 5	68.91	Dry	-	September 27, 2019
BH 6*	70.60	1.01	69.59	September 18, 2019
BH 7	71.39	2.19	69.20	September 27, 2019

Table 1 - Measured Groundwater Levels

Test Hole Location	Ground Surface	Groundwater Level		Date
		Depth (m)	Elevation (m)	
BH 8	68.59	Dry	-	September 27, 2019
BH 9*	70.95	1.17	69.78	September 18, 2019
BH 10	70.69	Blocked	-	September 27, 2019
BH 11	66.78	Blocked	-	September 27, 2019
BH 12	70.67	1.35	69.32	September 27, 2019
BH 13*	70.20	Dry	-	September 27, 2019
BH 14*	79.45	5.47	73.98	September 18, 2019
BH 15*	79.23	1.36	77.87	September 18, 2019
BH 16	78.64	Dry	-	September 18, 2019

Note: - * Denotes borehole instrumented with a 51 mm diameter monitoring well.
- The ground surface elevations at the borehole locations are referenced to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed buildings be constructed with conventional shallow foundations bearing on the undisturbed, stiff silty clay and/or undisturbed, compact silty sand.

Due to the presence of a silty clay layer, the proposed development will be subjected to grade raise restrictions. The permissible grade raise recommendations are discussed in Subsection 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

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

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

Fill Placement

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

Site excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Site excavated soils are not suitable for use as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

5.3 Foundation Design

Bearing Resistance Values

For design purposes, strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed on an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**.

Further, footings placed on an undisturbed, compact silty sand bearing surface can be designed using a bearing resistance value at SLS of **100 kPa** and a factored bearing resistance value at ULS of **150 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS.

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

An undisturbed soil bearing surface consists of 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.

Footings designed using the above noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a stiff silty clay or compact silty sand above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Settlement and Permissible Grade Raise

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

Our permissible grade raise recommendations for the subject site are presented in Drawing PG4854-2 - Permissible Grade Raise Plan in Appendix 2.

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

5.4 Design for Earthquakes

The proposed site can be taken as seismic site response **Class C** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A) for foundations considered at this site. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Slab on Grade Construction

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

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

5.6 Pavement Design

Car only parking areas, access lanes and heavy truck parking areas are anticipated at this site. The proposed pavement structures are shown in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

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

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structures at the subject site. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures. The pipe should have a positive outlet, such as a gravity connection to the catch basins or running drainage ditches.

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

6.2 Protection of Footings and Slabs Against Frost Action

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

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

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

6.4 Pipe Bedding and Backfill

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

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

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

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

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

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

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

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

6.6 Winter Construction

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

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the 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 samples indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to very aggressive corrosive environment.

6.8 Slope Stability Analysis

The slope conditions were reviewed by Paterson field personnel on December 5, 2019 for the slope stability assessment. Two slope cross-sections were studied as the worst case scenarios for the subject site. The cross section locations are presented on Drawing PG4854-3 - Limit of Hazard Lands attached to the current report.

The existing slope located in the southwestern portion of the site has an approximate height of 5 to 6 m with an incline of 4H:1V to 5H:1V. The slope was observed to be grass covered with occasional bushes and trees. A watercourse is not present at the base of this slope, and no evidence of active erosion was noted along the slope.

The existing slopes of the confined valley corridor located within the southeast portion of the site were also reviewed. The upper valley corridor walls were observed to be well vegetated and stable with no signs of active erosion. A 1 to 2 m wide watercourse was observed at the base of the slope, with an approximate 100 mm to 500 mm depth and minimal flow.

A slope stability analysis was carried out to determine the required construction setback from the top of the slope based on a factor of safety of 1.5. Erosional and access allowances were also considered in the determination of limits of hazard lands for slopes, where a water course is present, and are discussed in the following sections. The limit of hazard lands setbacks and top of slope are shown on Drawing PG4854-3 - Limit of Hazard Lands attached to the current report.

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 0.16g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

Two (2) slope cross-sections (Sections A and B) were studied as the worst case scenarios. The cross section locations are presented on Drawing PG4854-3 - Limit of Hazard Lands 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 2 through 5 in Appendix 2, and are based on the topographic data provided on Drawing PG4854-1 - Test Hole Location Plan in Appendix 2.

Stable Slope Allowance

The static analysis results for slope sections A and B are presented in Figures 2 and 4, respectively, provided in Appendix 2. The factor of safety for the slopes was greater than 1.5 for the slope sections analysed.

The results of the analyses with seismic loading are shown in Figures 3 and 5 presented in Appendix 2. The results indicate that the factor of safety for the sections are greater than 1.1. Based on these results, the slopes are considered to be stable under seismic loading.

As the slopes were determined to be stable under static and seismic conditions for the sections analyzed, a stable slope allowance is not considered to be required for the limit of hazard lands setback.

Toe Erosion and Erosion Access Allowance

For the 5 to 6 m slope in the southwestern portion of the site (Section A), given that no watercourse is present near the toe of the slope and no signs of active erosion were observed, toe erosion and erosion access allowances are not considered to be required for this slope.

The slopes along the valley corridor walls in the vicinity of the watercourse (Section B) were generally observed to be vegetated with grass, small brush, and occasional trees. Further, flow from the creek in the watercourse at the base of the slopes was observed to be minimal. In consideration of these observations, a toe erosion allowance of 2 m is recommended for the slopes in the vicinity of the watercourse.

A 6 m erosion access allowance is also recommended to be applied from the top of slope for the slopes adjacent to the existing watercourse, to allow for future maintenance of these slopes.

Limit of Hazard Lands

The limit of hazard lands setback lines for the proposed development are presented on Drawing PG4854-3 - Limit of Hazard Lands in Appendix 2. The limit of hazard lands lines along the watercourse consist of a 6 m erosion access allowance and 2 m toe erosion allowance, taken from the top of slope.

No hazard lands are required for the approximately 5 to 6 m slope in the southwestern portion of the site. However, once available, grading plans should be reviewed in order to evaluate potential impacts on this existing slope from the proposed development.

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

It is recommended that additional boreholes be completed at the site once detailed development drawings are available.

Following completion of the additional boreholes, a final Geotechnical Investigation Report would be prepared for the subject site.

8.0 Statement of Limitations

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

A geotechnical investigation is a limited sampling of a site. It is recommended that additional boreholes be completed at the site once detailed development drawings are available.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Avenue 31 or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Mar. 18, 2020
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott S. Dennis, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❏ Avenue 31 (3 copies)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Commercial / Industrial Complex
4055 and 4120 Russell Road, Ottawa, Ontario

DATUM Ground surface elevations provided Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 August 28

FILE NO. PG4854

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20	AU	1		
Brown **SILTY SAND**	0.30				
Brown **SILTY CLAY**		SS	2	38	5
		SS	3	100	5
		SS	4	100	4
- grey by 3.0m depth		SS	5	100	3
		SS	6	100	3
	5.18	SS	7	62	6
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders		SS	8	43	8
	6.88	SS	9	100	50+

DEPTH (m)	ELEV. (m)
0	69.48
1	68.48
2	67.48
3	66.48
4	65.48
5	64.48
6	63.48

End of Borehole

Practical refusal to augering at 6.88m depth

(GWL @ 2.48m - Sept. 18, 2019)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Commercial / Industrial Complex
4055 and 4120 Russell Road, Ottawa, Ontario

DATUM Ground surface elevations provided Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4854

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger

DATE 2019 August 28

SOIL DESCRIPTION	Stratum bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.43
TOPSOIL	0.15						
Loose, brown **SILTY SAND**	1.01	AU	1				
Very stiff to stiff, brown **SILTY CLAY**		SS	2	83	5	1	68.43
		SS	3	100	4	2	67.43
		SS	4	100	2	3	66.43
- grey by 3.8m depth	5.20	SS	5	100	2	4	65.43
		SS	6	100	2	5	64.43
GLACIAL TILL: Grey silty clay with gravel, cobbles and boulders	6.91	SS	7	17	15	6	63.43
		SS	8	100	55		
		SS	9	100	50+		
End of Borehole							

Practical refusal to augering at 6.91m depth

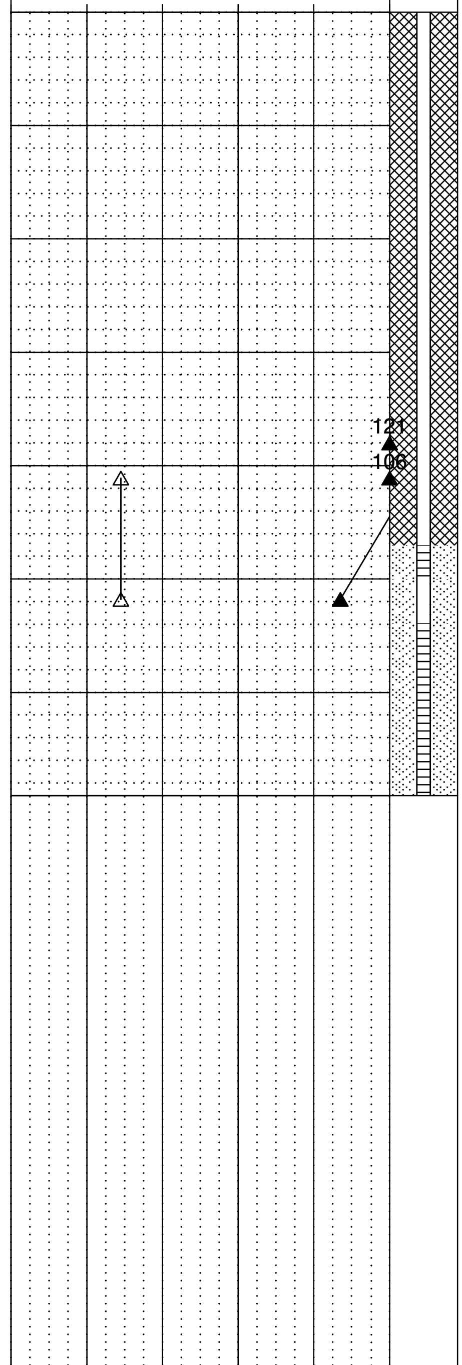
(Piezometer inaccessible - Sept. 7, 2019)

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

○ **Water Content %**

20 40 60 80

Piezometer Construction



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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Commercial / Industrial Complex
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DATUM Ground surface elevations provided Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4854

REMARKS

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger

DATE 2019 August 28

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.57
Brown **SILTY SAND** with gravel	0.28	AU	1				
Loose, brown **SILTY SAND**		SS	2	50	8	1	70.57
		SS	3	58	10	2	69.57
	2.90	SS	4	54	2		
GLACIAL TILL: Grey silty sand with gravel, cobbles, boulders, trace clay		SS	5	42	12	3	68.57
		SS	6	54	24	4	67.57
	4.88	SS	7	67	50+		

End of Borehole

Practical refusal to augering at 4.88m depth

(GWL @ 3.87m - Sept. 18, 2019)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

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FILE NO. PG4854

REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger **DATE** 2019 August 29

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	67.49		
TOPSOIL 0.18		AU	1						
Loose to compact, brown **SILTY SAND**		SS	2	96	8	1	66.49		
2.13		SS	3	100	21	2	65.49		
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders		SS	4	83	17				
3.12		SS	5	100	50+	3	64.49		
End of Borehole Practical refusal to augering at 3.12m depth (Piezometer inaccessible - Sept. 7, 2019)									

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

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FILE NO. PG4854

REMARKS

HOLE NO. BH 5

BORINGS BY CME 55 Power Auger

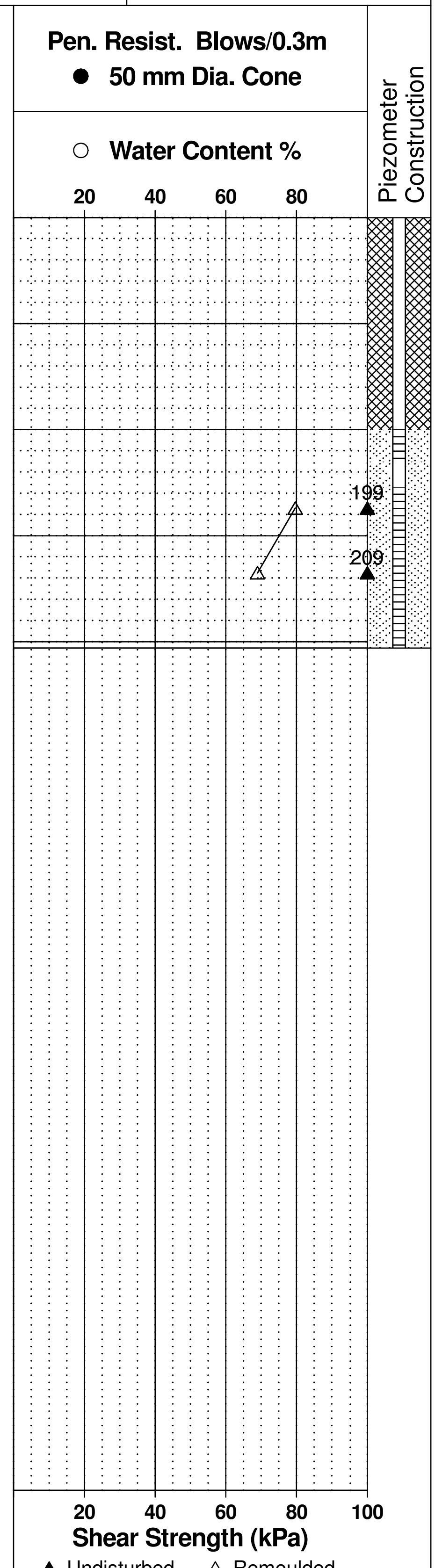
DATE 2019 August 29

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.91
TOPSOIL	0.13	AU	1				
Compact, brown **SILTY SAND**	1.37	SS	2	62	11	1	67.91
Very stiff to hard, brown **SILTY CLAY**	3.66	SS	3	100	7	2	66.91
						3	65.91
GLACIAL TILL: Brown silty clay with gravel, some cobbles, boulders	4.06	SS	4	60	50+	4	64.91

End of Borehole

Practical refusal to augering at 4.06m depth

(Piezometer inaccessible - Sept. 7, 2019)



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SOIL PROFILE AND TEST DATA

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Proposed Commercial / Industrial Complex
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FILE NO. PG4854

REMARKS

HOLE NO. BH 6

BORINGS BY CME 55 Power Auger **DATE** 2019 August 29

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	70.60
TOPSOIL	0.20	AU	1				
Compact, brown **SILTY SAND**	1.37	SS	2	75	10	1	69.60
Grey **SILTY CLAY**	2.13	SS	3	92	5	2	68.60
GLACIAL TILL: Grey silty sand with gravel, cobbles, boulders	2.29						

End of Borehole

Practical refusal to augering at 2.29m depth

(GWL @ 1.01m - Sept. 18, 2019)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

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FILE NO. PG4854

REMARKS

HOLE NO. BH 7

BORINGS BY CME 55 Power Auger

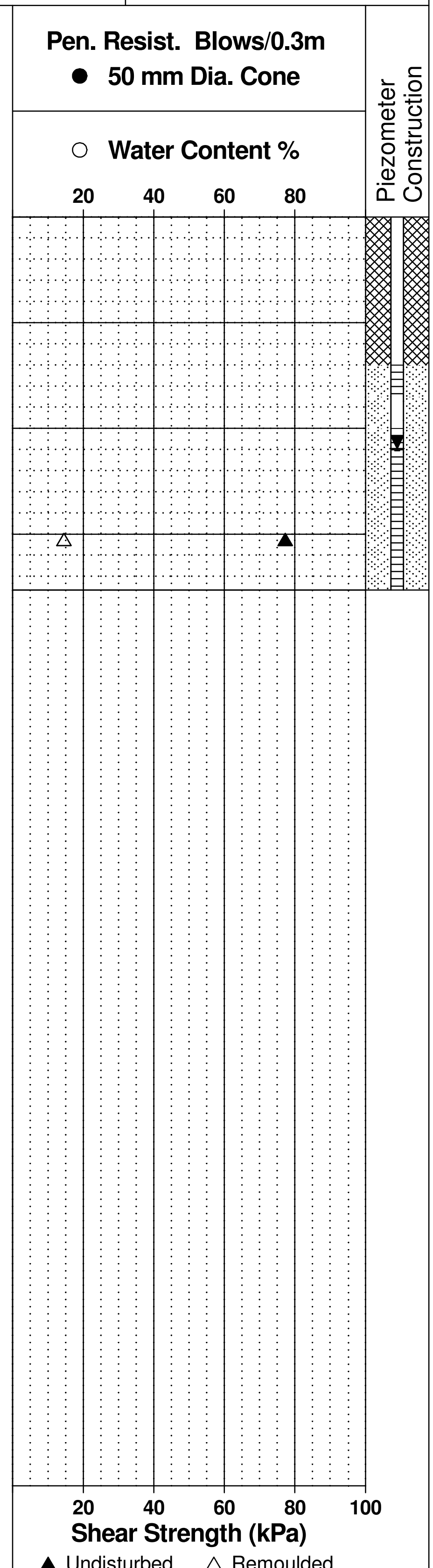
DATE 2019 August 29

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.39
TOPSOIL 0.20	AU	1				
Loose, brown **SILTY SAND**	SS	2	67	6	1	70.39
	SS	3	79	8	2	69.39
2.29						
Stiff, grey **SILTY CLAY**	SS	4	96	1	3	68.39
3.53						

End of Borehole

Practical refusal to augering at 3.53m depth

(GWL @ 2.19m - Sept. 27, 2019)



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SOIL PROFILE AND TEST DATA

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REMARKS

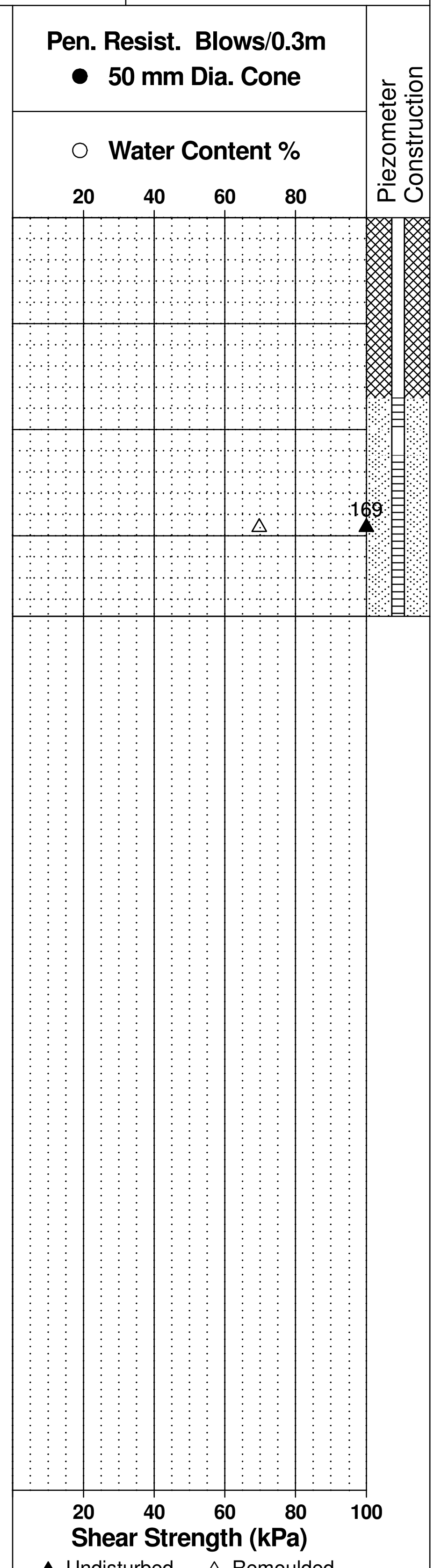
BORINGS BY CME 55 Power Auger

DATE 2019 August 29

FILE NO. PG4854

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.59
TOPSOIL 0.23		AU	1				
Very stiff, brown **SILTY CLAY**		SS	2	75	8	1	67.59
- grey by 1.5m depth		SS	3	100	4	2	66.59
2.90						3	65.59
GLACIAL TILL: Grey silty clay, trace gravel, cobbles and boulders 3.76		SS	4	75	3		
End of Borehole							
Practical refusal to augering at 3.76m depth							
(Piezometer dry - Sept. 27, 2019)							



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FILE NO. PG4854

REMARKS

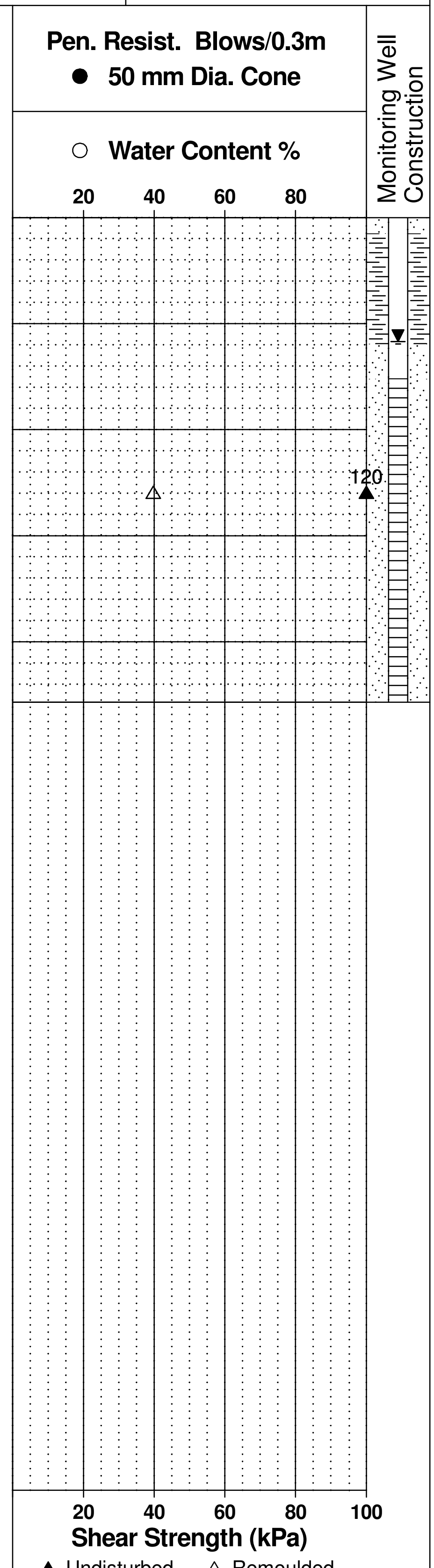
HOLE NO. BH 9

BORINGS BY CME 55 Power Auger **DATE** 2019 August 29

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	70.95
TOPSOIL	0.25	AU	1				
Loose, brown SILTY SAND	1.52	SS	2	46	9	1	69.95
Very stiff to stiff, grey SILTY CLAY		SS	3	75	3	2	68.95
		SS	4	100	1	3	67.95
		SS	5	100	1	4	66.95
	4.57	SS	6	100	1		

End of Borehole

(GWL @ 1.17m - Sept. 18, 2019)



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SOIL PROFILE AND TEST DATA

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FILE NO. **PG4854**

REMARKS

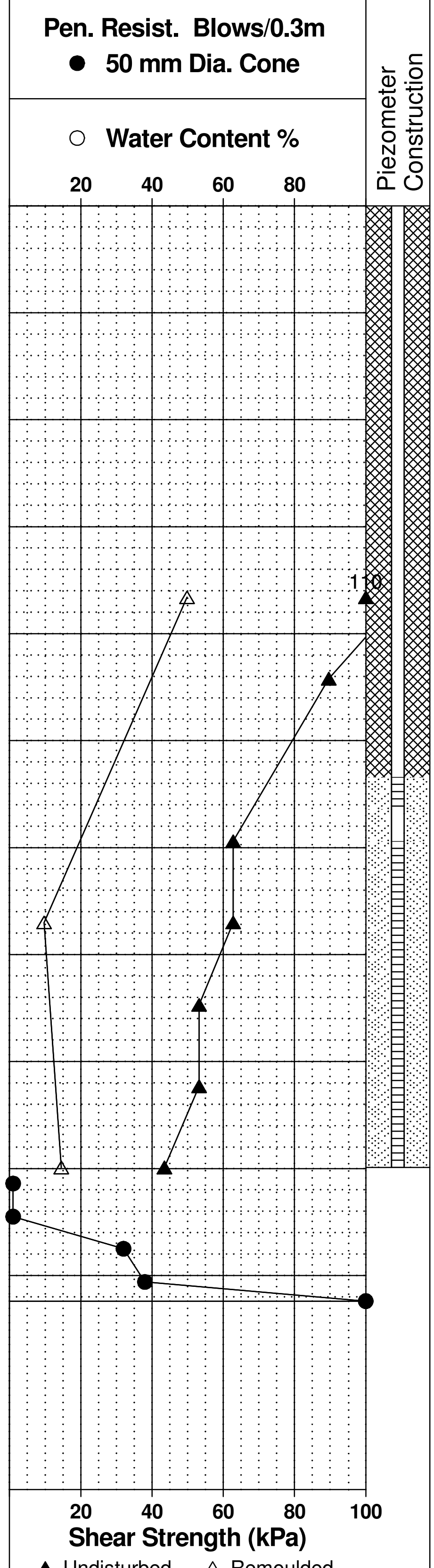
HOLE NO. **BH10**

BORINGS BY CME 55 Power Auger

DATE 2019 August 29

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.15	AU	1		
Compact, brown **SILTY SAND**	1.52	SS	2	54	10
Very stiff to stiff, grey **SILTY CLAY** - firm by 8.4m depth	8.99	SS	3	83	6
		SS	4	100	2
		SS	5	100	1
Dynamic Cone Penetration Test commenced at 8.99m depth.	10.24				
End of Borehole Practical DCPT refusal at 10.24m depth (Piezometer blocked - Sept. 27, 2019)					

DEPTH (m)	ELEV. (m)
0	70.69
1	69.69
2	68.69
3	67.69
4	66.69
5	65.69
6	64.69
7	63.69
8	62.69
9	61.69
10	60.69



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Commercial / Industrial Complex
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DATUM Ground surface elevations provided Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. **PG4854**

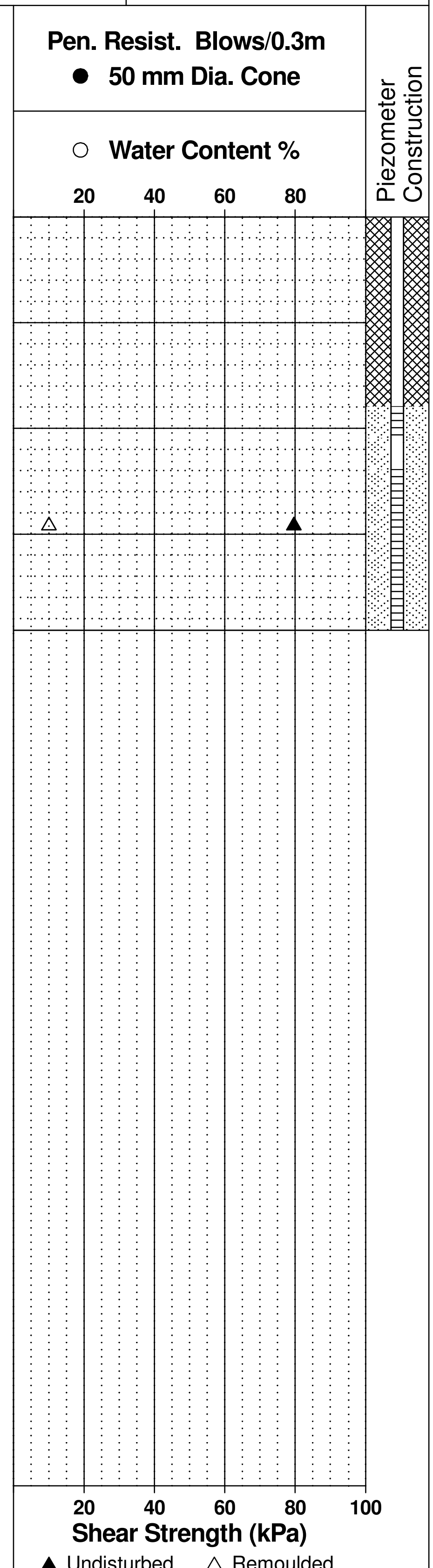
REMARKS

HOLE NO. **BH11**

BORINGS BY CME 55 Power Auger

DATE 2019 August 30

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	66.78
TOPSOIL (0.15)	AU	1				
Very stiff to stiff, brown **SILTY CLAY**	SS	2	88	8	1	65.78
- grey by 1.5m depth	SS	3	92	4	2	64.78
(3.05)					3	63.78
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles, boulders	SS	4	100	12		
(3.91)	SS	5	0	50+		
End of Borehole						
Practical refusal to augering at 3.91m depth						
(Piezometer blocked - Sept. 27, 2019)						



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SOIL PROFILE AND TEST DATA

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FILE NO. PG4854

REMARKS

HOLE NO. BH12

BORINGS BY CME 55 Power Auger

DATE 2019 August 30

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	70.67
TOPSOIL	0.10	AU	1				
Very stiff to stiff, brown **SILTY CLAY**		SS	2	79	8	1	69.67
- grey by 1.5m depth		SS	3	100	5	2	68.67
		SS	4	100	5		
	3.81	SS	5	100	4	3	67.67
GLACIAL TILL: Brown silty clay with gravel, cobbles and boulders		SS	6	50	6	4	66.67
		SS	7	54	18	5	65.67
						6	64.67
	7.32	SS	8	100	12	7	63.67

End of Borehole

Practical refusal to augering at 7.32m depth

(GWL @ 1.35m - Sept. 27, 2019)

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

○ Water Content %
20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Commercial / Industrial Complex
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FILE NO. PG4854

REMARKS

HOLE NO. BH13

BORINGS BY CME 55 Power Auger

DATE 2019 August 30

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	70.20
TOPSOIL 0.08		AU	1				
Very stiff, brown **SILTY CLAY**		SS	2	100	8	1	69.20
1.52							
GLACIAL TILL: Brown silty clay with gravel, cobbles and boulders		SS	3	100	5	2	68.20
		SS	4	62	7	3	67.20
		SS	5	38	7	4	66.20
		SS	6	33	15		
4.57							
End of Borehole							
Practical refusal to augering at 4.57m depth							
(GWL @ 0.85m - Sept. 18, 2019)							

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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REMARKS

BORINGS BY CME 55 Power Auger **DATE** 2019 September 4

FILE NO. PG4854

HOLE NO. BH14

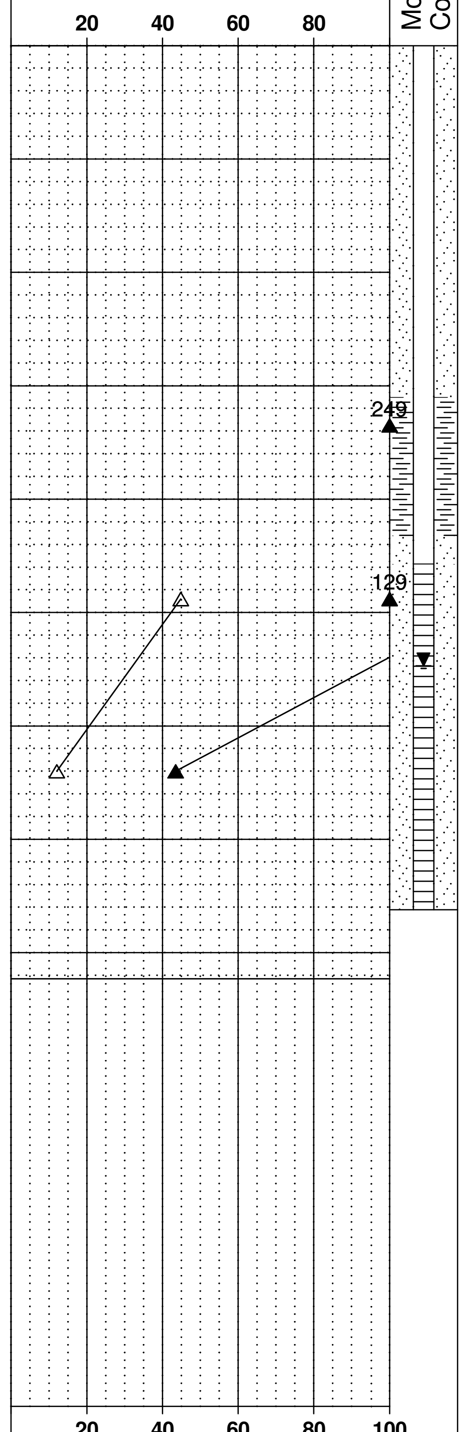
SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.20
FILL: Brown silty clay with sand, some gravel	1.52
Hard to very stiff, brown **SILTY CLAY** - stiff to firm and grey by 5.3m depth	7.92
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	8.23
End of Borehole (GWL @ 5.47m - Sept. 18, 2019)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	12	17
SS	3	96	15
SS	4	100	9
SS	5	100	7
SS	6	100	4
SS	7	100	1
SS	8	100	W
SS	9	100	W
SS	10	100	1
SS	11	10	2

DEPTH (m)	ELEV. (m)
0	79.45
1	78.45
2	77.45
3	76.45
4	75.45
5	74.45
6	73.45
7	72.45
8	71.45

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

Monitoring Well Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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FILE NO. PG4854

REMARKS

HOLE NO. BH15

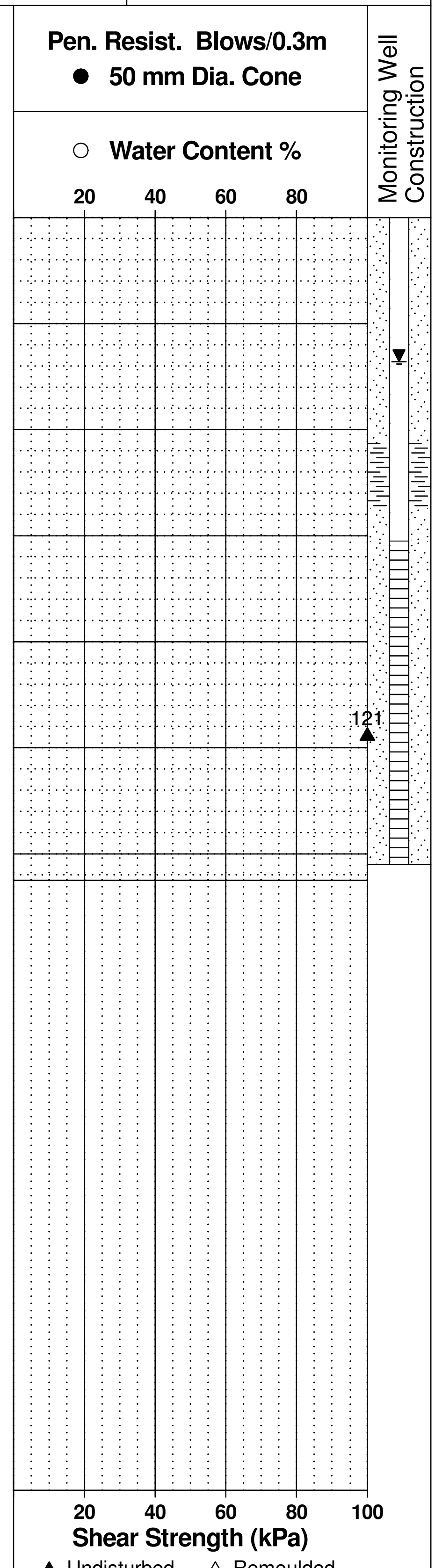
BORINGS BY CME 55 Power Auger **DATE** 2019 September 4

SOIL DESCRIPTION	Depth of layer base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	79.23
TOPSOIL	0.10						
FILL: Brown silty sand, trace gravel	0.76	SS	1	67	7		
Very stiff, brown **SILTY CLAY**		SS	2	75	13	1	78.23
		SS	3	92	11	2	77.23
		SS	4	100	9		
		SS	5	100	5	3	76.23
		SS	6	100	4	4	75.23
- grey by 4.8m depth	5.64	SS	7	100	2	5	74.23
GLACIAL TILL: Grey silty sand with gravel, cobbles, boulders	6.25	SS	8	67	50	6	73.23
		SS	9	60	50+		

End of Borehole

Practical refusal to augering at 6.25m depth

(GWL @ 1.36m - Sept. 18, 2019)



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SOIL PROFILE AND TEST DATA

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FILE NO. PG4854

REMARKS

HOLE NO. BH16

BORINGS BY CME 55 Power Auger **DATE** 2019 September 4

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	78.64
TOPSOIL 0.20	AU	1				
Hard, brown **SILTY CLAY,** trace sand	SS	2	88	19	1	77.64
	SS	3	100	15	2	76.64
	SS	4	100	9	3	75.64
3.81						
GLACIAL TILL: Brown silty sand with ground rock 3.91	SS	5	100	50+		

End of Borehole

Practical refusal to augering at 3.91m depth

(Piezometer dry - Sept. 27, 2019)

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

249

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

Piezometer Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

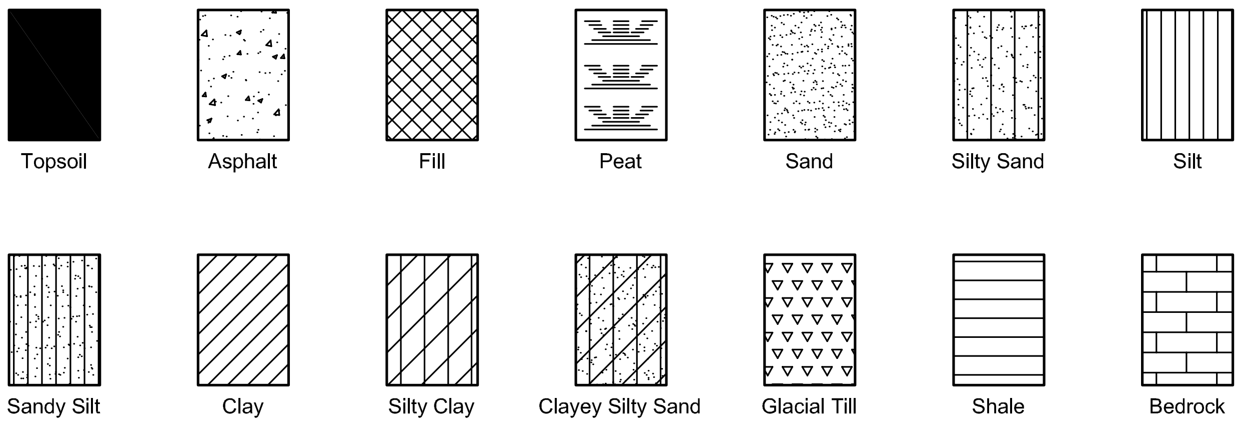
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

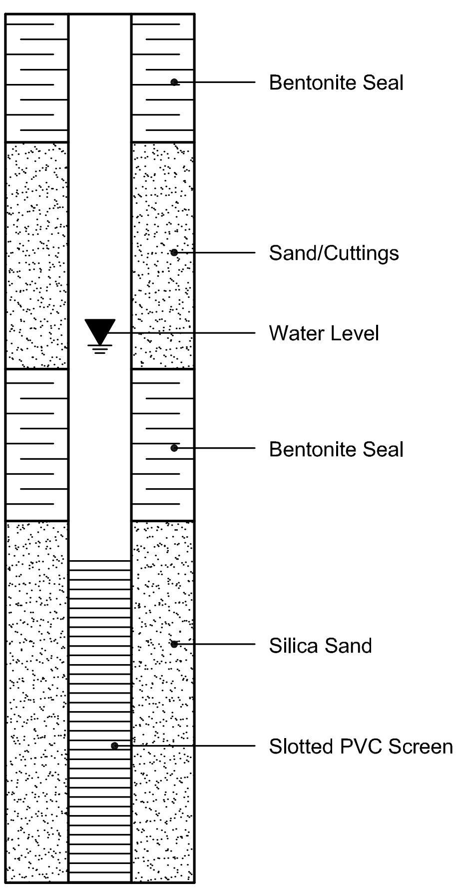
SYMBOLS AND TERMS (continued)

STRATA PLOT

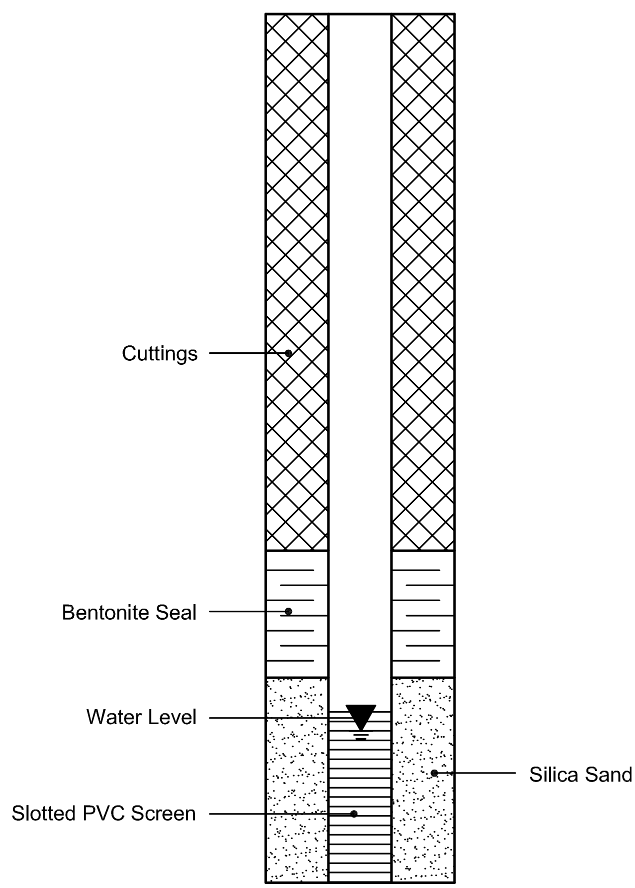


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 27100

Report Date: 03-Oct-2019

Order Date: 30-Sep-2019

Project Description: PG4854

Client ID:		BH2 5'-7'	BH3 5'-7'	-	-
Sample Date:		27-Sep-19 11:00	27-Sep-19 11:00	-	-
Sample ID:		1940105-01	1940105-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.8	77.0	-	-
General Inorganics					
pH	0.05 pH Units	6.72	6.79	-	-
Resistivity	0.10 Ohm.m	18.3	64.3	-	-
Anions					
Chloride	5 ug/g dry	199	14	-	-
Sulphate	5 ug/g dry	71	34	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 5 - SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG4854-1 - TEST HOLE LOCATION PLAN

DRAWING PG4854-2 - PERMISSIBLE GRADE RAISE PLAN

DRAWING PG4854-3 - LIMIT OF HAZARD LANDS

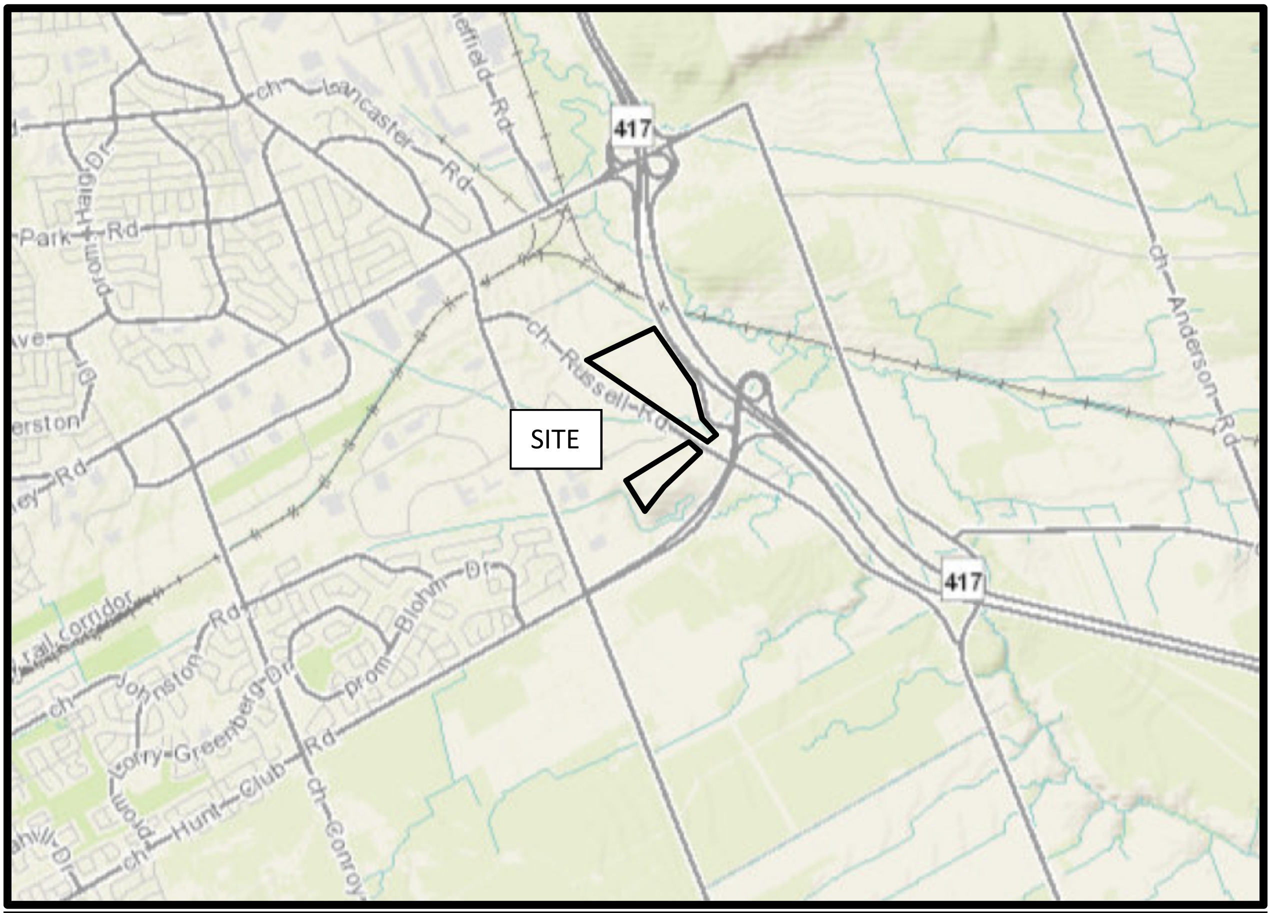


FIGURE 1

KEY PLAN

patersongroup

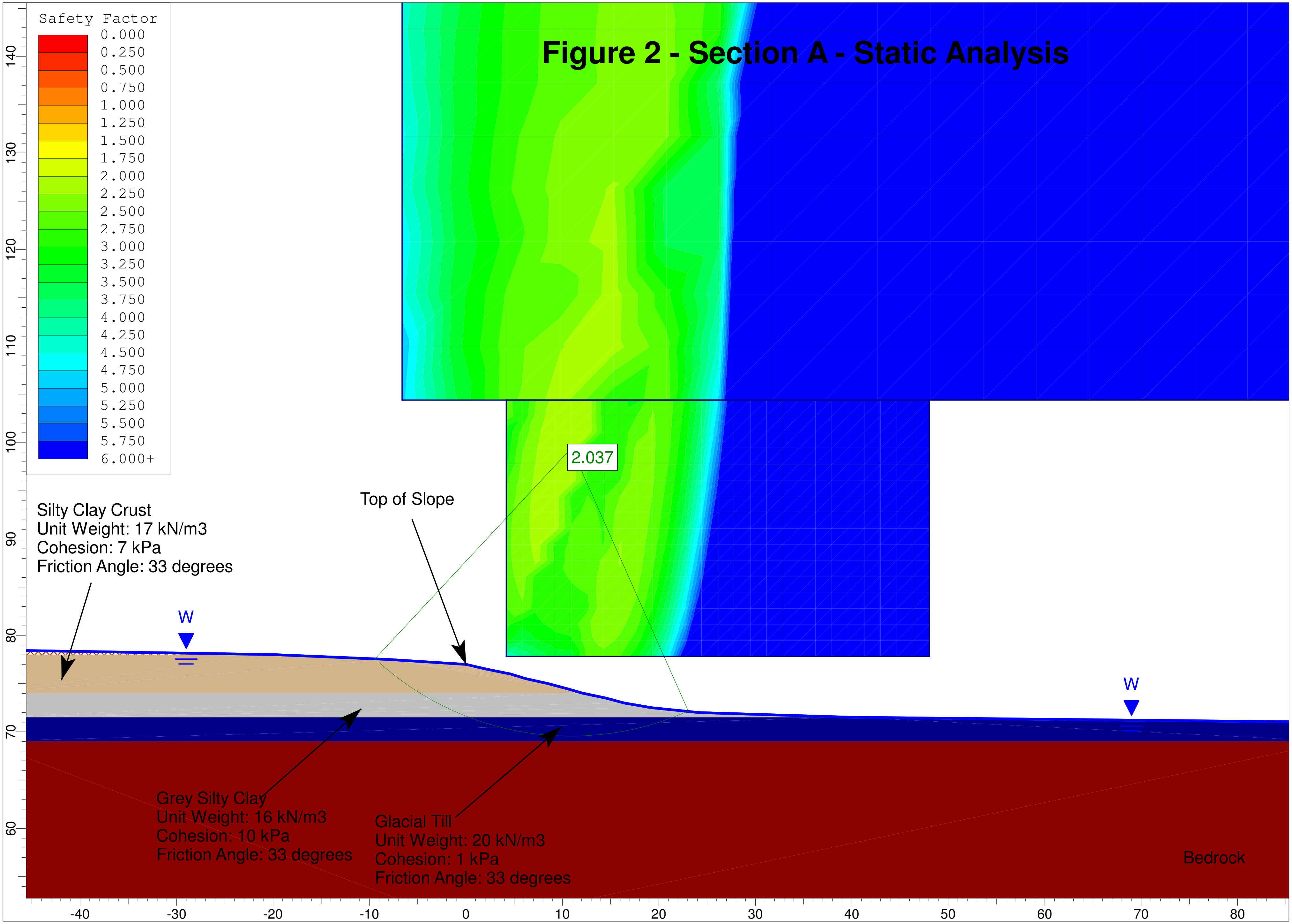
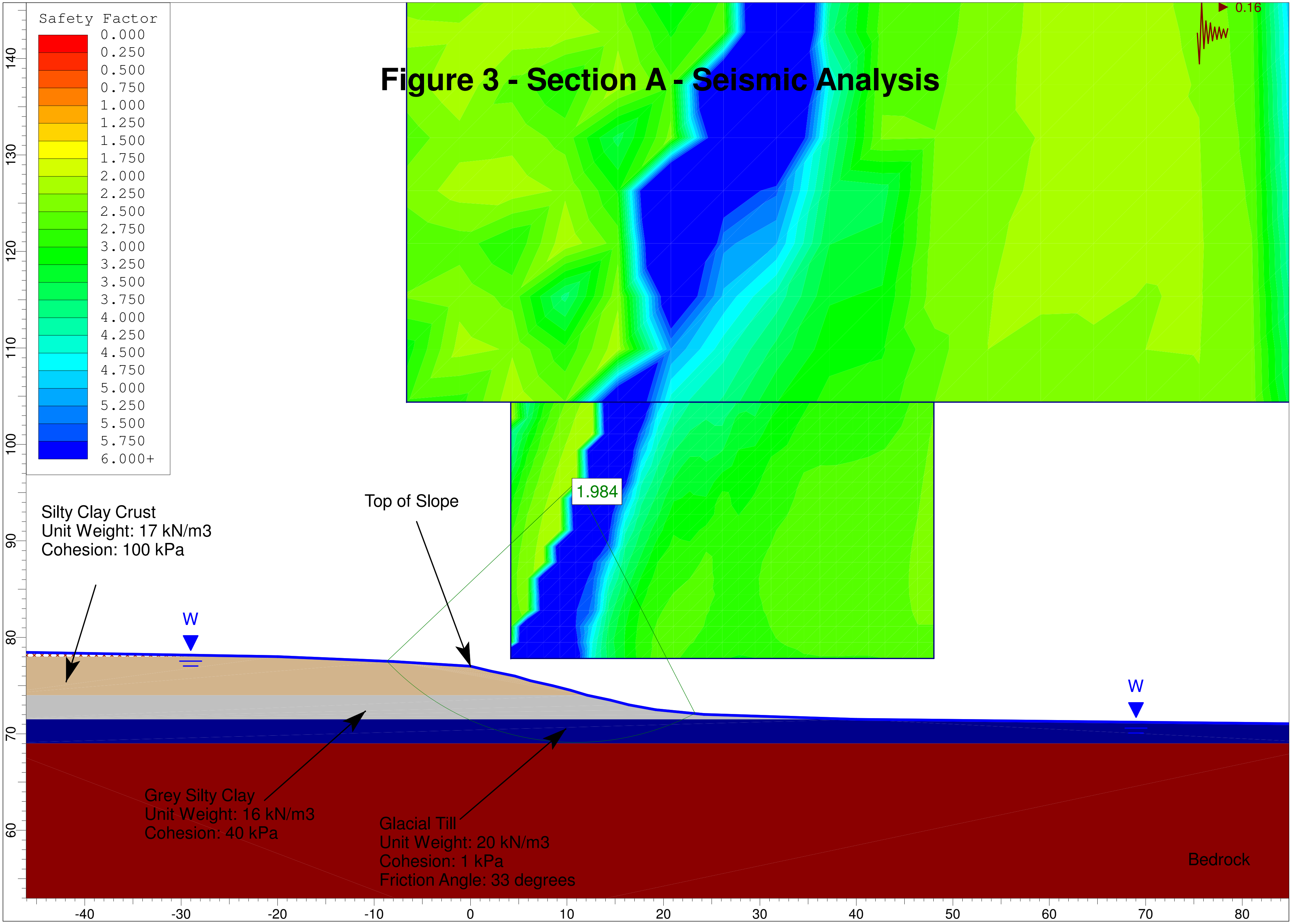


Figure 2 - Section A - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
2.037
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
Top of Slope
W
W
Grey Silty Clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 33 degrees
Bedrock
140
130
120
110
100
90
80
70
60
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80

Figure 3 - Section A - Seismic Analysis



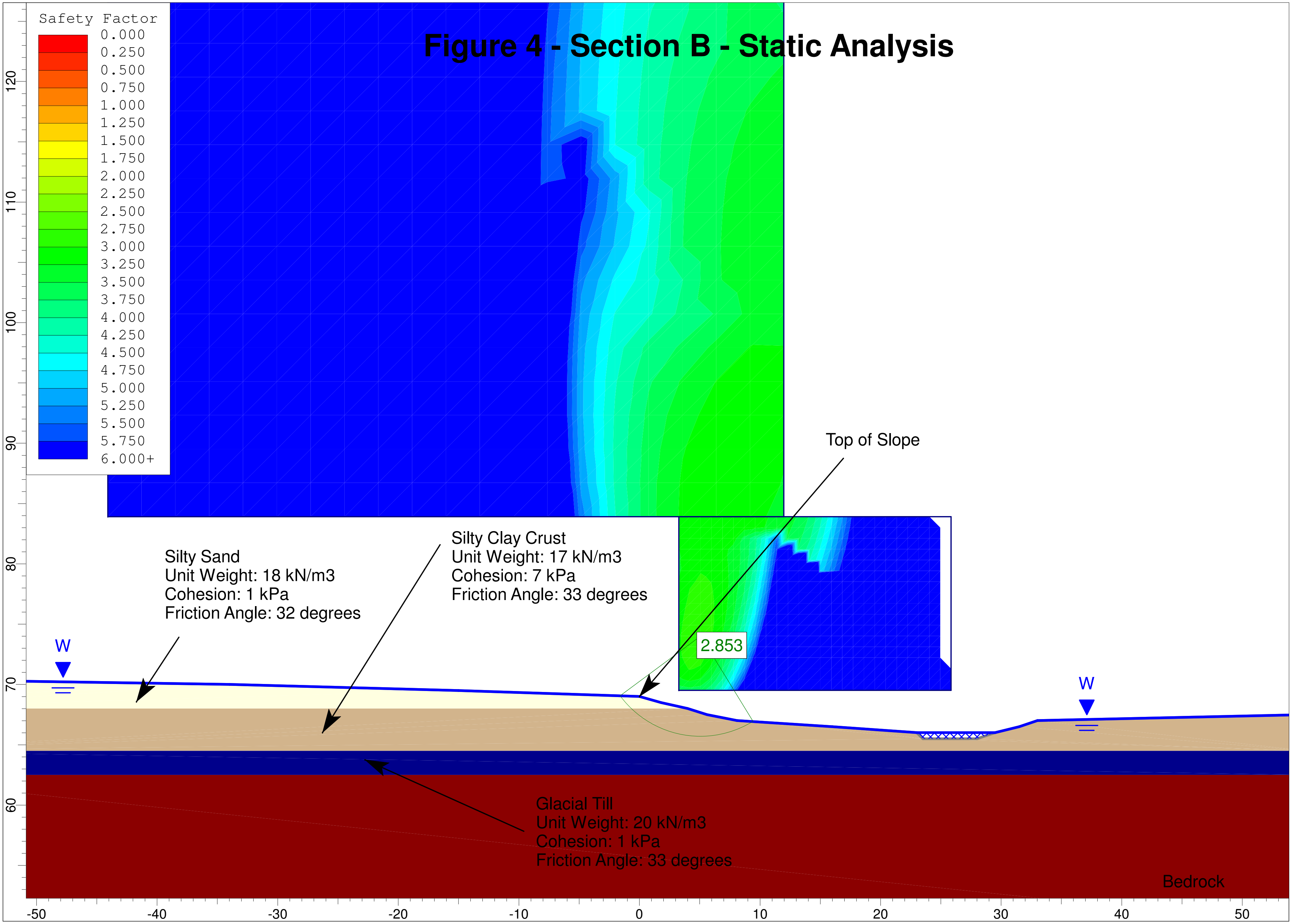


Figure 4 - Section B - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Top of Slope
Silty Sand
Unit Weight: 18 kN/m3
Cohesion: 1 kPa
Friction Angle: 32 degrees
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
2.853
W
W
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 33 degrees
Bedrock
120
110
100
90
80
70
60
-50
-40
-30
-20
-10
0
10
20
30
40
50

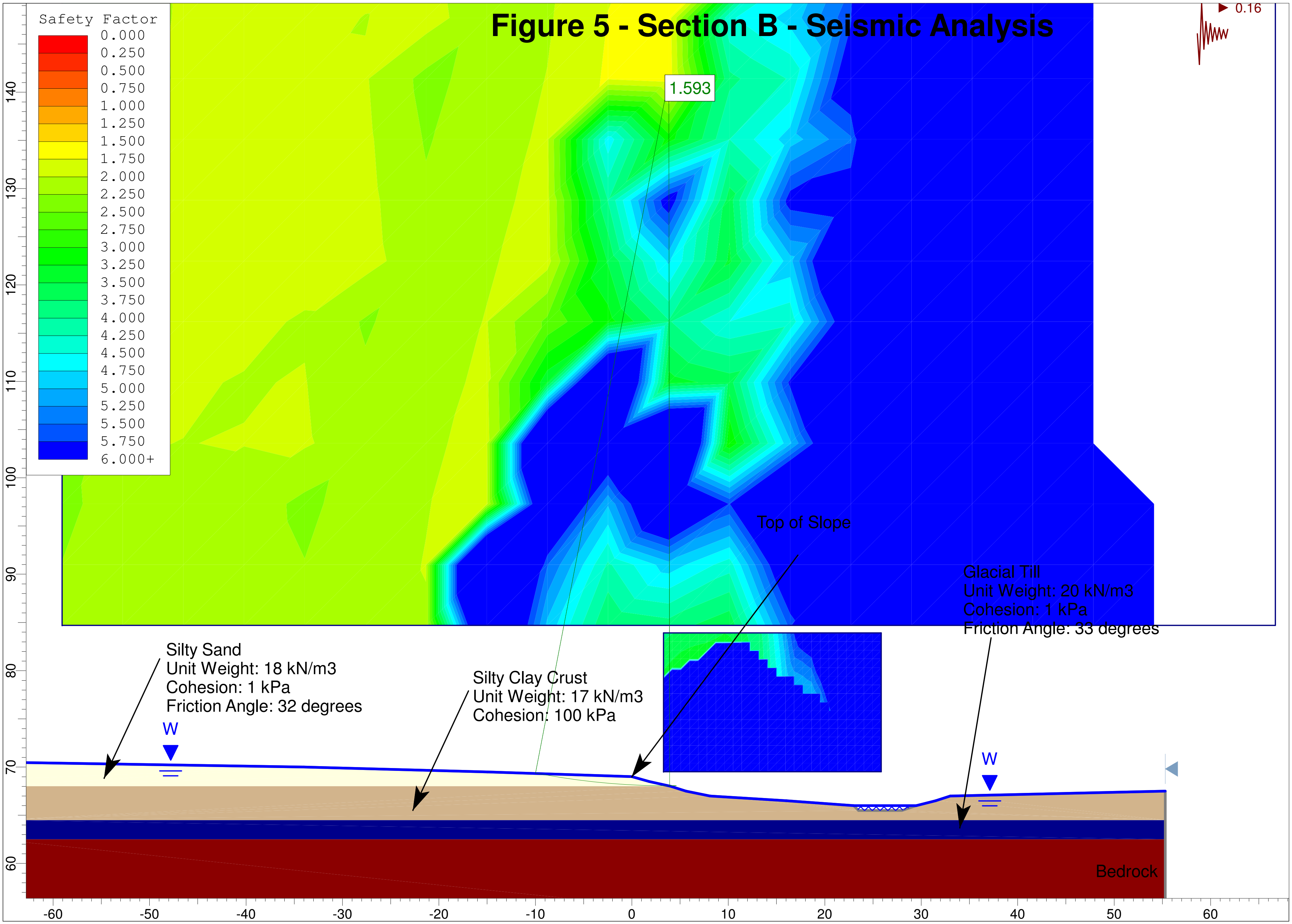
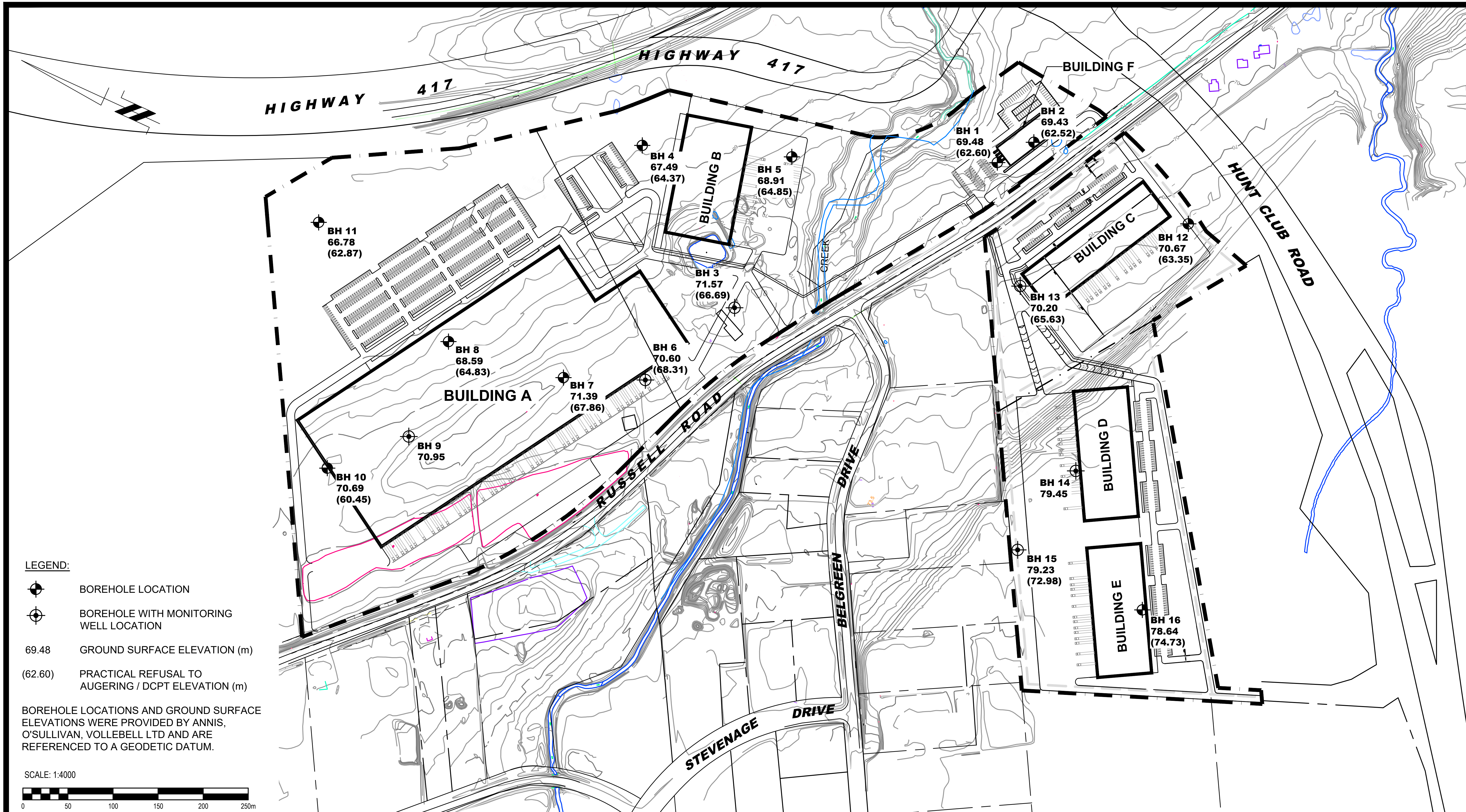


Figure 5 - Section B - Seismic Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.593
0.16
Top of Slope
Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 33 degrees
Silty Sand
Unit Weight: 18 kN/m3
Cohesion: 1 kPa
Friction Angle: 32 degrees
Silty Clay Crust
Unit Weight: 17 kN/m3
Cohesion: 100 kPa
W
W
Bedrock
140
130
120
110
100
90
80
70
60
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60



LEGEND:

- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- 69.48 GROUND SURFACE ELEVATION (m)
- (62.60) PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS WERE PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBELL LTD AND ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:4000

0 50 100 150 200 250m

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consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
2	UPDATED TO LATEST CONCEPTUAL PLAN	18/03/2020	SD
1	UPDATED NEW BASE PLAN	13/01/2020	SD

AVENUE 31

GEOTECHNICAL INVESTIGATION - NATIONAL CAPITAL BUSINESS PARK
4055 AND 4120 RUSSELL ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:4000	Date: 10/2019
Drawn by: YA	Report No.: PG4854-1
Checked by: SD	Dwg. No.: **PG4854-1**
Approved by: DJG	Revision No.: 2

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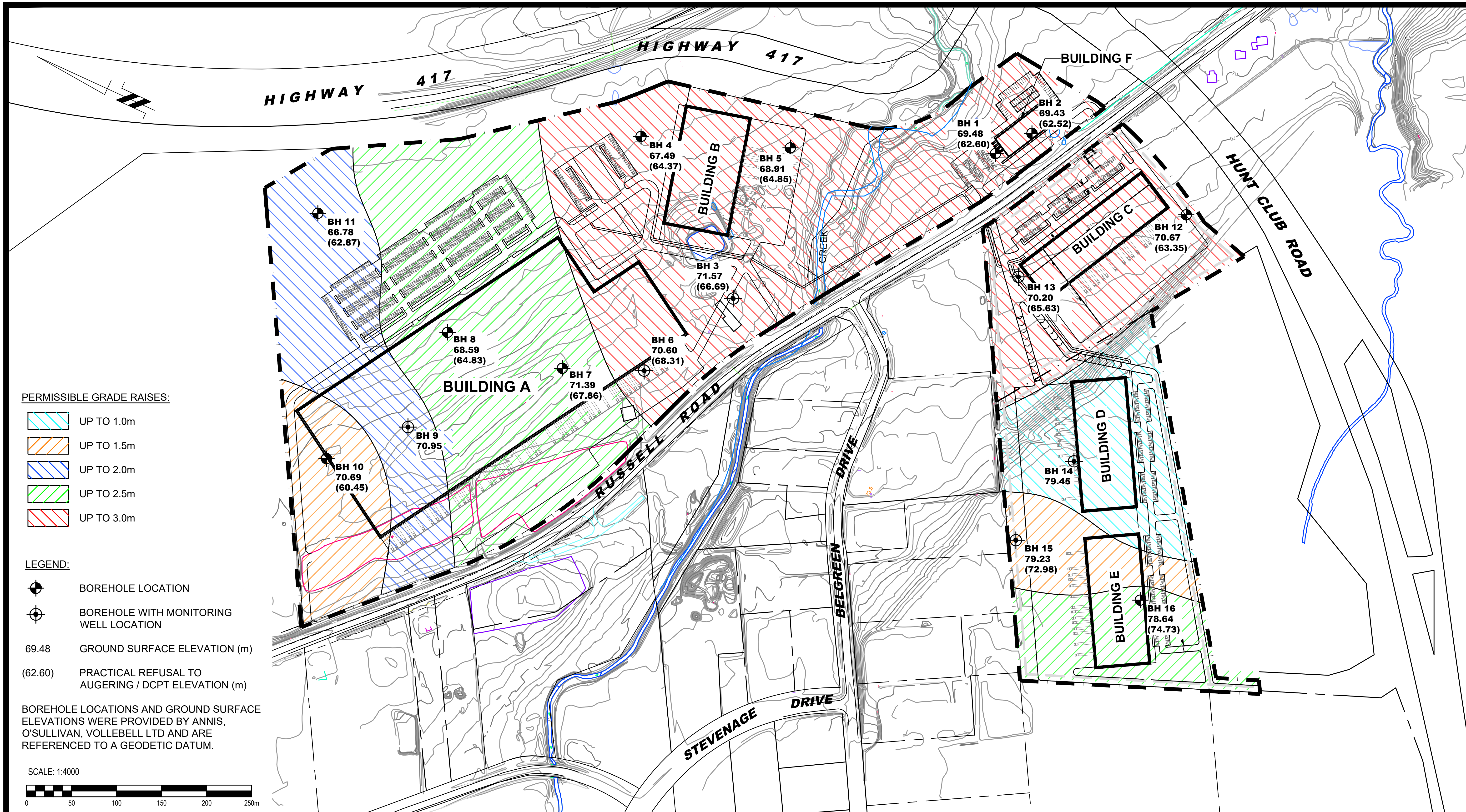

PERMISSIBLE GRADE RAISES:

- UP TO 1.0m
- UP TO 1.5m
- UP TO 2.0m
- UP TO 2.5m
- UP TO 3.0m

LEGEND:

- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- 69.48 GROUND SURFACE ELEVATION (m)
- (62.60) PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS WERE PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBELL LTD AND ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:4000

0 50 100 150 200 250m

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consulting engineers

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Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
2	UPDATED TO LATEST CONCEPTUAL PLAN	18/03/2020	SD
1	UPDATED NEW BASE PLAN	13/01/2020	SD

AVENUE 31
GEOTECHNICAL INVESTIGATION - NATIONAL CAPITAL BUSINESS PARK
4055 AND 4120 RUSSELL ROAD
OTTAWA, ONTARIO

Title:
PRELIMINARY PERMISSIBLE GRADE RAISE PLAN

Scale: 1:4000	Date: 10/2019
Drawn by: MPG	Report No.: PG4854-1
Checked by: SD	Dwg. No.: PG4854-2
Approved by: DJG	Revision No.: 2

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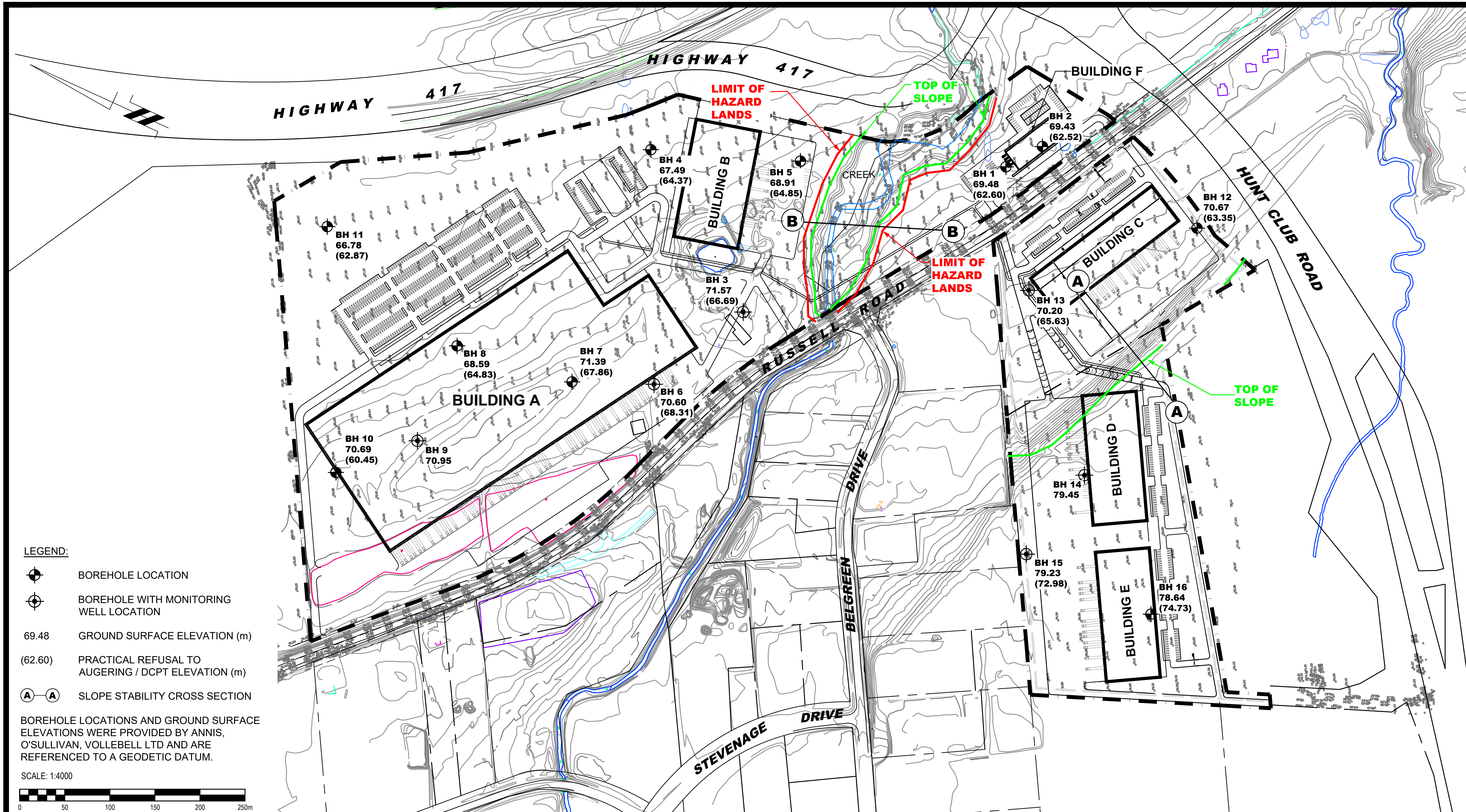

LEGEND:

- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- 69.48 GROUND SURFACE ELEVATION (m)
- (62.60) PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)
- A—A SLOPE STABILITY CROSS SECTION

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS WERE PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBELL LTD AND ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:4000

0 50 100 150 200 250m

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Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
2	UPDATED TO LATEST CONCEPTUAL PLAN	18/03/2020	SD
1	UPDATED NEW BASE PLAN ADDED LIMIT OF HAZARD LANDS	13/01/2020	SD

AVENUE 31

GEOTECHNICAL INVESTIGATION - NATIONAL CAPITAL BUSINESS PARK
4055 AND 4120 RUSSELL ROAD

OTTAWA, ONTARIO

Title:

LIMIT OF HAZARD LANDS

Scale:	1:4000	Date:	01/2020
Drawn by:	YA	Report No.:	PG4854-1
Checked by:	SD	Dwg. No.:	PG4854-3
Approved by:	DJG	Revision No.:	2

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