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Geotechnical Investigation

Proposed Multi-Storey Building
1040 Somerset Street West
Ottawa, Ontario

Prepared For

Claridge Homes

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Revision 1

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Table of Contents

PAGE
1.0 Introduction 1
2.0 Proposed Development 1
3.0 Method of Investigation
3.1 Field Investigation 2
3.2 Field Survey 3
3.3 Laboratory Testing 4
3.4 Analytical Testing 4
4.0 Observations
4.1 Surface Conditions 5
4.2 Subsurface Profile 5
4.3 Groundwater 6
5.0 Discussion
5.1 Geotechnical Assessment 7
5.2 Site Grading and Preparation 7
5.3 Foundation Design 10
5.4 Design for Earthquakes 11
5.5 Basement Slab 12
5.6 Basement Wall 13
5.7 Rock Anchor Design 15
5.8 Pavement Design 17
6.0 Design and Construction Precautions
6.1 Foundation Drainage and Backfill 19
6.2 Protection of Footings Against Frost Action 20
6.3 Excavation Side Slopes and Temporary Shoring 20
6.4 Pipe Bedding and Backfill 22
6.5 Groundwater Control 23
6.6 Winter Construction 24
6.7 Corrosion Potential and Sulphate 24
6.8 Slope Stability Analysis 25
6.9 Protection of Existing Watermain (Breezehill Avenue) 26
7.0 Recommendations 27
8.0 Statement of Limitations 28

Appendices

Appendix 1 Soil Profile and Test Data Sheets
Symbols and Terms
Analytical Testing Results

Appendix 2 Figure 1 - Key Plan
Figure 2 - Shear Wave Velocity Profile at Shot Location +22.6 m
Figure 3 - Shear Wave Velocity Profile at Shot Location -0.4 m
Figures 4 to 7 - Slope Stability Sections
Drawing PG2674-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1040 Somerset Street West, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were 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 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.

An environmental investigation was carried out in conjunction with the geotechnical program and the findings are presented under separate cover.

2.0 Proposed Development

Although drawings were not available during the preparation of this report, it is understood that the proposed development at the subject site will consist of a multi-storey building with 7 to 9 levels of underground parking. Further, the footprint of the underground parking is anticipated occupy the entire site.

The subject site is currently occupied by a single-storey commercial building which will be demolished prior to construction of the proposed multi-storey building.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation consisted of 1 borehole (BH 1-20) which was carried out on November 17, 2020 to a depth of 25.6 m below the existing ground surface. Previous investigations at this site consisted of 4 boreholes in April and May 2012 (BH 1-12 through BH 4-12), 2 boreholes on October 27, 2014 (BH 5-14 and BH 6-14), and 1 borehole on February 12, 2015 (BH 7-15). In addition, 4 boreholes (BH 1 through BH 4) were placed across the subject site as part of the environmental investigation program in May 2007. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are illustrated on Drawing PG2674-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck- or 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 at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

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

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

Rock samples were recovered in BH 1-20 using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are 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.

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

Groundwater

A 19 mm PVC groundwater monitoring well was installed in BH 1-20, BH 5-14, BH 6-14, BH 7-15, BH 1-12, BH 3-12, and BH 3 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The boreholes were surveyed with respect to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located at the southwest corner of the intersection of Somerset Street West and Breezehill Avenue North. A geodetic elevation of 63.67 m was provided by Annis, O'Sullivan, Vollebekk for this TBM. Borehole locations and ground surface elevations at the borehole locations are presented on Drawing PG2674-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is generally occupied by a single-storey commercial building, with an associated asphalt-paved access lane and parking area on the northern end of the site. The site is bordered by Somerset Street West to the north, Breezehill Avenue North to the west, a commercial property to the south, and the Trillium rail corridor to the east. A 1375 mm watermain is also located underlying Breezehill Avenue North, to the west of the subject site.

The existing ground surface across the site is relatively level at approximate geodetic elevation 63 m, however, a slope is located beyond the eastern property line, extending downward approximately 5 m to the Trillium rail corridor. .

4.2 Subsurface Profile

Overburden

The subsurface profile at the borehole locations consists of fill underlying the asphalt surface, extending to approximate depths of 3 to 4.7 m below the existing ground surface. The fill was generally observed to consist of a silty sand to sand and gravel with occasional coal, slag, glass, wood pieces, brick fragments, and concrete fragments.

Underlying the fill, a silty clay deposit was encountered, generally consisting of a very stiff to stiff, brown to grey silty clay with occasional traces of sand.

A glacial till deposit was encountered underlying the silty clay at approximate depths of 8.0 and 8.3 m in BH 1-20 and BH 1-12, respectively. The glacial till deposit was observed to consist of a silty sand to silty clay with gravel, cobbles, and boulders.

Practical refusal to augering was encountered in BH 1-12 at an approximate depth of 13.6 m below the existing ground surface.

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

Bedrock

Bedrock was cored at BH 1-20 from depths of 13.7 to 25.6 m, and consisted of limestone with interbedded shale seams. Based on the RQDs of the recovered rock core, the bedrock can be classified as good to excellent in quality.

4.3 Groundwater

Groundwater level readings were recorded at the monitoring well locations and are presented in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1-20	63.22	7.75	55.47	November 26, 2020
BH 3-12	63.27	3.63	59.64	November 11, 2014
BH 5-14	63.46	3.41	60.05	November 11, 2014
BH 6-14	63.47	3.15	60.32	November 11, 2014
BH 7-15	63.49	3.62	59.87	February 19, 2015
BH 3	-	3.83	-	June 4, 2007

Note: The ground surface elevations at the test hole locations were referenced to a TBM consisting of the top of spindle of a fire hydrant located at the southwest corner of Breezehill Avenue and Somerset Street West. A geodetic elevation of 63.67 m was provided for the TBM.

It should be noted that surface water can become perched within a recently backfilled borehole, which can lead to a higher than normal groundwater level readings.

The long-term groundwater level can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples. Based on these observations, the groundwater is expected between 5 to 6 m depth. Groundwater levels are subject to seasonal fluctuations and therefore could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed multi-storey building be founded on shallow footings placed on clean, surface sounded bedrock.

Considering the close proximity of the west embankment of the Somerset Street overpass to the northern end of the subject site, the shoring design will have to include support for the foundation west embankment. It is recommended that the foundation dimensions and location of the footing for the west embankment be determined to ensure the shoring does not encounter the existing foundation.

In addition, due to the close proximity of the adjacent 1375 mm diameter watermain, which is located less than 5 to 6 m from the west property boundary along Breezehill Avenue, additional precautions should be taken during excavation activities to ensure that the existing service is not affected. In particular, the temporary shoring system along Breezehill Avenue is recommended to consist of a secant pile wall socketed into the bedrock in order to minimize lateral movement of the shoring system.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the anticipated founding level of the proposed multi-storey building, it is expected that all existing overburden material will be excavated from within the footprint of the underground parking levels for the proposed multi-storey building.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock for the underground parking levels. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming. Furthermore, rock grinding may be considered to complete the bedrock removal along vertical surfaces and lessen the effects of over break encountered with other mechanical methods. Grinding of the bedrock also provides a better prepared surface for installation of the waterproofing system.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be 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/claims related to the blasting operations.

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

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

Vibration Considerations

Construction operations are 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 equipments could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the temporary shoring system using soldier piles, sheet piling, and/or secant piles will require the use of this equipment. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining 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). It should be noted that these guidelines are for today's 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 building.

A vibration monitoring program should be implemented during the construction of the temporary shoring system and bedrock blasting program to ensure that the neighbouring structures and utilities are not negatively impacted by the proposed building's construction.

Fill Placement

Excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement. Alternatively, an engineered fill such as an OPSS Granular A or Granular B Type II, compacted to 98% of its standard Proctor maximum dry density (SPMDD), could be placed around the proposed footings.

Watermain Monitoring Program

The following vibration monitoring program is recommended to ensure that excessive movements and vibrations do not occur at the watermain location:

❏ Install 2 inclinometers located adjacent to the 1375 mm diameter watermain and the shoring face. Daily monitoring events should be completed during the excavation program until the tiebacks are stressed and then weekly during the construction program until the foundation extends above exterior finished grade. An alert level with 3 mm of movement will require an assessment. An action level with movement greater than 6 mm will require immediate attention and possible mitigation measures. A visual inspection of the excavation side slopes will also be completed along with the inclinometer monitoring events.

❏ Periodically monitor the vibration levels within an existing valve chamber along the subject section of watermain. If the vibration monitor cannot be placed within the valve chamber, the monitor will be placed at ground surface in the immediate area of shoring works.

❏ If the vibration limits noted in Table 2 are exceeded, the site superintendent will be notified by Paterson personnel of the exceedance and the shoring/excavation operation will be stopped. The project surveyor will survey the watermain level (within the valve chamber) to ensure pipe movement has not occurred. If pipe movement is not observed based on the survey results, the shoring/excavation operation will resume.

The following vibration limits are recommended for the shoring/excavation operation to be completed adjacent to the 1375 mm diameter watermain.

Table 2 - Vibration Limits for Work Completed Adjacent to Watermain

Location of Vibration Monitor	Peak Particle Velocity (mm/s)	Frequency (Hz)
Inside the Valve Chamber	15	4 to 12
	25	>40
At Ground Surface (within 3 m of watermain)	10	4 to 12
	25	>40

Note: The values should be interpolated between 12 and 40 Hz.

Weekly reporting of our findings and recommendations will be provided to the owner and the City of Ottawa. Any mitigation measures contemplated for implementation will be discussed with the owner and City of Ottawa personnel.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **4,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

A factored bearing resistance value at ULS of **6,000 kPa**, incorporating a geotechnical resistance factor of 0.5, could be used for footings founded on limestone bedrock if the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint(s) of the footing(s). At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant.

Settlement

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

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

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2.

Field Program

The seismic array testing location was placed on the western end of the site in an approximate north-south direction as presented in Drawing PG2674-1, attached to the present report. Paterson field personnel placed 21 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 0.4 m away from the first geophone, 1.6, 11.5, and 18.5 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

The Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the Ontario Building Code (OBC) 2012, and as presented below:

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}076m/s}\right)}$$

$$V_{s30} = 2{,}076m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for foundations placed on bedrock is 2,076 m/s. Therefore, a **Site Class A** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. It is expected that the basement area will be mostly parking and the recommended pavement structure noted in Subsection 5.8 will be applicable.

However, if storage or other uses of the lower level where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone layer under the lower level floor slab.

5.6 Basement Wall

It is understood that the lower basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. A nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3) for this condition. A seismic earth pressure component will not be applicable for the foundation wall which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where soil is to be retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

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

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

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

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

Seismic Conditions

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

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

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

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

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

5.7 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 by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that 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 anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It is recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with much less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 60 and 120 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 0.3, can be used. 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 subsoils information, a **Rock Mass Rating (RMR) of 69** 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

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

Table 3 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	69 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

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

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	1.2	0.6	1.8	250
	1.9	0.8	2.7	500
	3.0	1.5	4.5	1000
125	1.1	0.5	1.6	250
	1.5	0.7	2.2	500
	2.6	1.0	3.6	1000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

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. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Structure

The proposed lower basement slab will be considered a rigid pavement structure. The following rigid pavement structure is suggested to support car parking only.

Table 5 - Recommended Rigid Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
125	**Wear Course -** Concrete slab
300 to 500	**BASE** - OPSS Granular A (thickness will depend on required pipe cover and other subfloor surfaces)
	SUBGRADE - Bedrock

It is recommended that the concrete slab to be used as a rigid pavement structure consist of category C2 concrete with a strength of 32 MPa at 28 days and air entrainment of 5 to 8 percent.

For design purposes, the pavement structure presented in the following table could be used for the design of access lanes.

Table 6 - Recommended Pavement Structure - Access Lanes

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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Existing fill, or OPSS Granular B Type I or II material placed over bedrock.	

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

Foundation Drainage

It is recommended that the portion of the proposed building foundation walls located below the long-term groundwater table (approximate geodetic elevation 58 m) be placed against a groundwater infiltration control system which is fastened to the temporary shoring system or vertical bedrock face. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which comes in contact with the proposed building's foundation walls.

For the portion of the groundwater infiltration control system installed against the vertical bedrock face, the following is recommended:

❏ Line drill the excavation perimeter.
❏ Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface, as required based on site inspection by Paterson.
❏ Place a suitable membrane against the prepared bedrock surface, such as a bentomat liner system or equivalent. The membrane liner should extend from geodetic elevation 58 m down to the footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at underside of footing level.
❏ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
❏ Pour foundation wall against the composite drainage system.

It is recommended that 100 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Subfloor Drainage

Subfloor drainage may be required to control water infiltration due to groundwater lowering within the bedrock. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at approximate 6 to 8 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Concrete Mud Slab

To lessen the potential groundwater infiltration at the base of the excavation, consideration should be given to pouring a 100 mm thick concrete mud slab using 20 MPa compressive strength concrete directly on the bedrock surface prior to pouring footings. The purpose of the concrete mud slab is to provide a uniform layer to restrict the bulk of the groundwater infiltration. The effectiveness of the concrete mud slab is dependent on pouring a uniform layer on a flat surface avoiding pits and horizontal surfaces from deeper excavations. More details can be provided once the excavation plan is available.

6.2 Protection of Footings Against Frost Action

It is expected that the underground parking levels will not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard.

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by shoring systems from the start of the excavation until the structure is backfilled. Based on the depth of the proposed structure and the proximity to property lines, it is anticipated that a temporary shoring system will be required to support the excavation.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly 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.

Rock Stabilization

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. Where the excavation extends into the bedrock, horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface.

The requirement for horizontal rock anchors will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Temporary Shoring

Temporary shoring will be required to support the overburden soils. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the shoring designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's representative prior to implementation.

On the north, east, and south sides of the excavation, the temporary shoring system may consist of a soldier pile and lagging system or interlocking steel sheet piles. However, on the west side of the excavation along Breezehill Avenue, due to the proximity of the 1375 mm diameter watermain, the temporary shoring system is recommended to consist of a secant pile wall which is socketed into the bedrock. Further, the secant pile wall design should include tie backs and walers to provide lateral support.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced.

The toe of the shoring is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is used.

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

Table 7 - Soil Parameters for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible.

The dry unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil 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 (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

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 the potential 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

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.

The rate of flow of groundwater into the excavation through the overburden and bedrock should be moderate for the expected subsurface conditions at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

Groundwater Control for Building Construction

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

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

Impacts on Neighbouring Properties

Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site mostly consist of frost susceptible materials. In 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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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 this site, whereas the resistivity is indicative of a moderate to very aggressive corrosive environment.

6.8 Slope Stability Analysis

A slope section was analysed along the east property boundary where a slope is located within the railway easement to the east of the subject site. The cross section location is presented on Drawing PG2674-1 - Test Hole Location Plan in Appendix 2.

The existing soils along the approximately 5 m high slope are noted to consist primarily of fill. The majority of the slope surface is brush covered with some construction debris.

The analysis of slope stability was carried out using SLIDE, a computer program that permits a two-dimensional slope stability analysis using several methods, including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain than the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures. An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

It should be noted that the majority of the soil within the subject site will be removed as part of the proposed building construction. The removal of the soil along the top of slope as part of the redevelopment works will increase the overall slope stability factor of safety beyond what is currently present. Also, the proposed building will not be negatively impacted by the neighbouring slope due to the proposed founding level being located well below any failure circles associated with the existing slope.

The results of the stability analysis for the existing static conditions at Section A are presented on Figure 4 in Appendix 2. The factor of safety for the slope was less than 1.5 for Section A. Figure 6 presents our analysis which includes the proposed building footprint. The factor of safety for this slope condition is slightly improved, but still less than 1.5 when the proposed building is included.

The results of the analyses including seismic loading are shown on Figures 5 and 7 for the slope section. The results indicate that the factor of safety for both conditions are greater than 1.1.

6.9 Protection of Existing Watermain (Breezehill Avenue)

Due to the close proximity of the existing watermain, which is located approximately 5 to 6 m from the west property boundary along Breezehill Avenue, extra precautions should be taken at the time of excavation. A secant pile wall socketed into the bedrock and supported with walers and tie backs is recommended to provide lateral support to the watermain during the excavation.

As an extra measure, a monitoring program is required to ensure the lateral support zone of the watermain has not been impacted. The monitoring program will consist of installation of two inclinometers between the shoring system and the existing watermain. In addition, the excavation side slope should be monitored by the geotechnical consultant on a daily basis during until tie backs are stressed and weekly until the foundation extends above exterior finished grade. An alert level for any movement greater than 3 mm should be assessed immediately. An action level for movement of 6 mm will require immediate investigation and possible mitigation measures.

Weekly reporting including inspection findings and recommendations should be provided to the owner and the City by the geotechnical consultant.

7.0 Recommendations

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

- Review of the geotechnical aspects of the excavating contractor’s shoring design, prior to construction.
- Observation of all bearing surfaces prior to the placement of concrete.
- Inspection of the foundation waterproofing and all foundation drainage systems.
- Sampling and testing of the concrete and fill materials used.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Claridge Homes 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.

Scott Dennis

LICENSED PROFESSIONAL ENGINEER
Nov. 30, 2020
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott S. Dennis, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution

☐ Claridge Homes (e-mail copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1040 Somerset Street West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 17, 2020

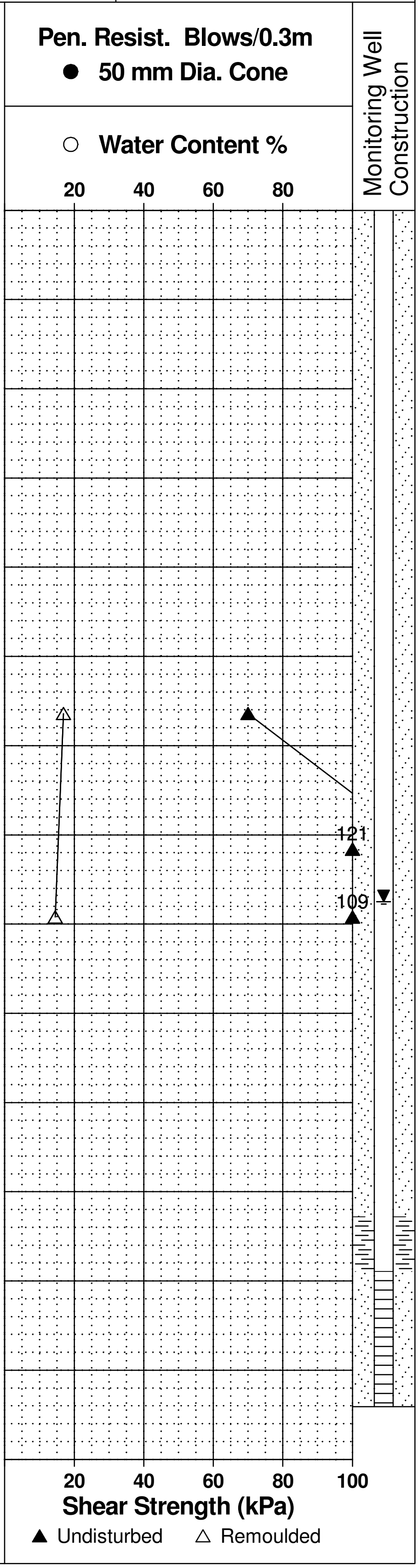
FILE NO. PG2674

HOLE NO. BH 1-20

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Grey sand and gravel	0.91
FILL: Black gravelly sand	1.37
FILL: Brown sand, some silt and gravel	2.13
FILL: Dark brown silty sand, some clay, trace gravel	3.35
FILL: Brown silty sand, some gravel, cobbles and boulders	4.65
Stiff, grey **CLAYEY SILT**	5.18
Stiff to very stiff, grey **SILTY CLAY**	8.03
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles and boulders	13.74

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	27
SS	3	17	5
SS	4	25	5
SS	5	17	22
SS	6	50	50+
SS	7	83	4
SS	8	100	2
SS	9	75	3
SS	10	21	20
SS	11		9
SS	12	33	20
SS	13	50	34
SS	14	43	33
SS	15	42	34
SS	16	100	50+

DEPTH (m)	ELEV. (m)
0	63.22
1	62.22
2	61.22
3	60.22
4	59.22
5	58.22
6	57.22
7	56.22
8	55.22
9	54.22
10	53.22
11	52.22
12	51.22
13	50.22
14	49.22



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1040 Somerset Street West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 17, 2020

FILE NO. PG2674

HOLE NO. BH 1-20

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									
						14	49.22		
		RC	1						
		RC	2	94	88				
						15	48.22		
		RC	3	100	92				
						16	47.22		
						17	46.22		
		RC	4	100	100				
						18	45.22		
		RC	5	100	90				
BEDROCK: Good to excellent quality, grey limestone with shale seams						19	44.22		
						20	43.22		
		RC	6	100	100				
						21	42.22		
		RC	7	100	100				
						22	41.22		
						23	40.22		
		RC	8	100	95				
						24	39.22		
		RC	9	100	95				
						25	38.22		
25.55									
End of Borehole (GWL @ 7.75m - Nov. 25, 2020)									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1040 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Breezehill Avenue and Somerset Street West. Geodetic elevation = 63.669m.

REMARKS

BORINGS BY CME 55 Power Auger

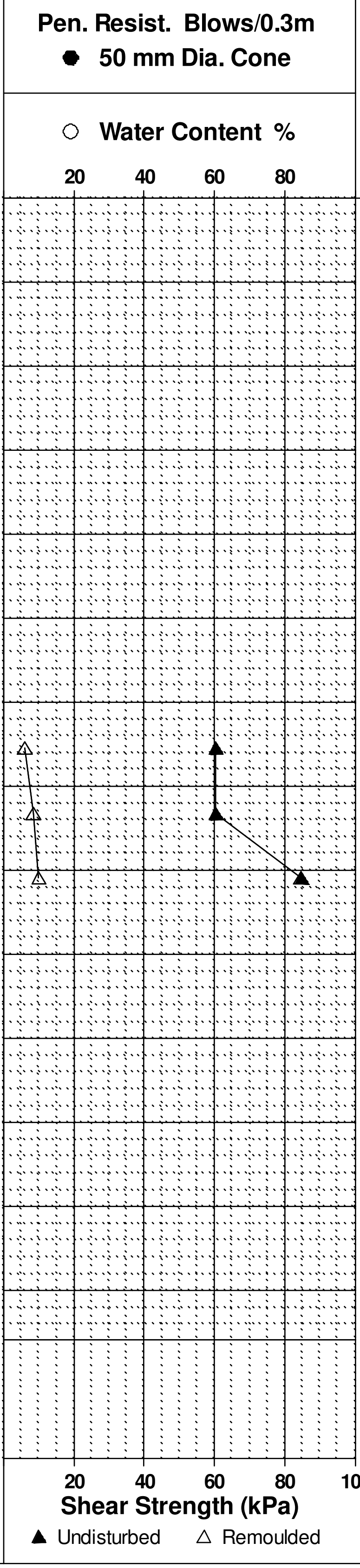
DATE April 20, 2012

FILE NO. PG2674

HOLE NO. BH 1-12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with clay and gravel	▨	SS	1	42	5
		SS	2	25	13
		SS	3	29	45
3.70		SS	4	100	50+
Very loose, grey **SILTY SAND** with clay		SS	5	83	3
5.25		SS	6	92	1
Stiff, grey **SILTY CLAY**		SS	7	100	1
		SS	8	100	P
Stiff, grey **SILTY CLAY**		SS	9	100	P
8.30		SS	10	100	P
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles		SS	11	75	P
		SS	12	8	11
		SS	13	58	4
		SS	14	50	20
		SS	15	67	50+
		SS	16	67	24
13.59		SS	17	80	50+
End of Borehole Practical refusal to augering at 13.59m depth.					

DEPTH (m)	ELEV. (m)
0	63.28
1	62.28
2	61.28
3	60.28
4	59.28
5	58.28
6	57.28
7	56.28
8	55.28
9	54.28
10	53.28
11	52.28
12	51.28
13	50.28



Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1040 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Breezehill Avenue and Somerset Street West. Geodetic elevation = 63.669m.

REMARKS

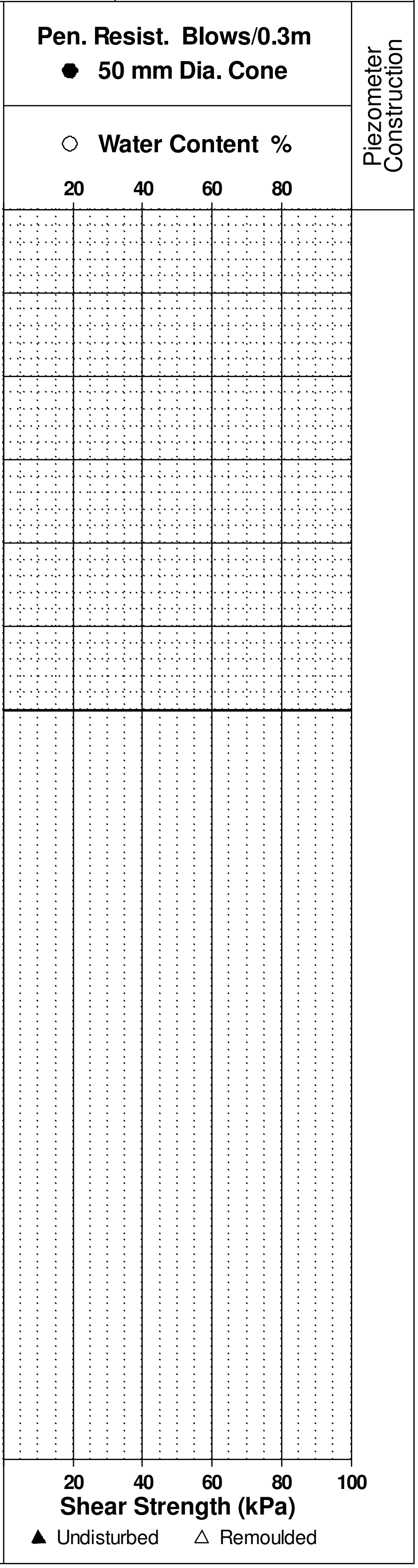
BORINGS BY CME 55 Power Auger

DATE May 3, 2012

FILE NO. PG2674

HOLE NO. BH 2-12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.28
Asphaltic concrete 0.05							
FILL: Crushed stone 0.15							
FILL: Black gravel with silty sand 1.45		AU	1			1	62.28
FILL: Brown silty sand with gravel and boulders		SS	2	25	9	2	61.28
		SS	3	33	9		
3.73		SS	5	42	0	3	60.28
Grey **SILTY CLAY** with sand		SS	5	50	3	4	59.28
		SS	6	92	3	5	58.28
6.02		SS	7	100	2		
End of Borehole						6	57.28



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1040 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Breezehill Avenue and Somerset Street West. Geodetic elevation = 63.669m.

REMARKS

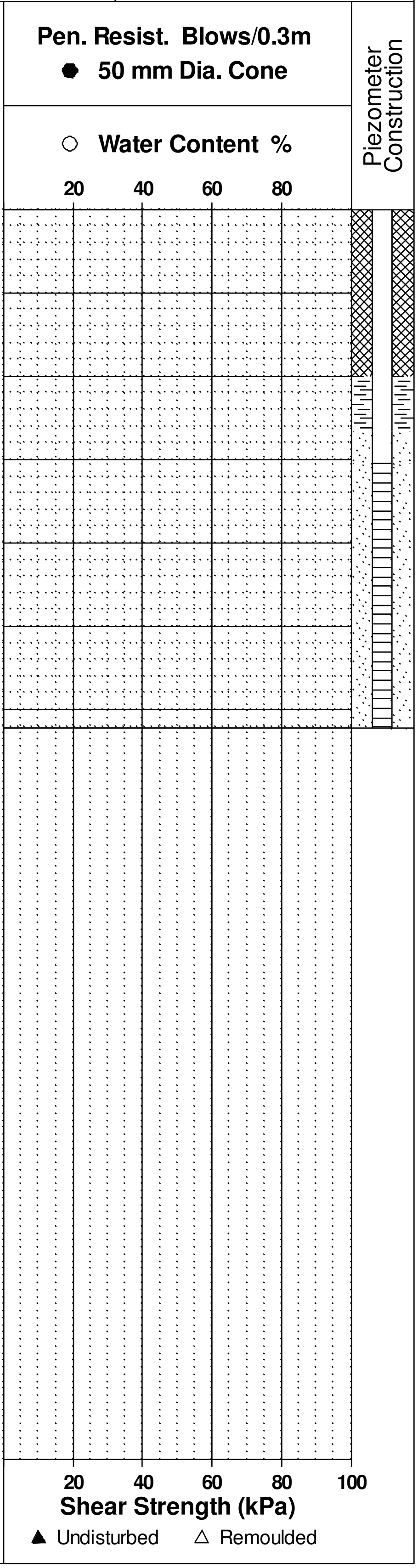
BORINGS BY CME 55 Power Auger

DATE May 3, 2012

FILE NO. PG2674

HOLE NO. BH 3-12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.27
Asphaltic concrete 0.05	▨						
FILL: Crushed stone 0.25	▨						
FILL: Brown silty sand with gravel, coal and concrete	▨	SS	1	33	29	1	62.27
	▨	SS	2	8	3	2	61.27
2.21							
FILL: Brown silty sand with gravel and cobbles	▨	SS	3	33	14	3	60.27
	▨	SS	4	42	81		
3.73							
Grey **SILTY CLAY,** trace sand	▨	SS	5	58	6	4	59.27
	▨	SS	6	67	2	5	58.27
	▨	SS	7	100	3	6	57.27
6.22							
End of Borehole							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1040 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Breezehill Avenue and Somerset Street West. Geodetic elevation = 63.669m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 3, 2012

FILE NO. PG2674

HOLE NO. BH 4-12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.29
Asphaltic concrete 0.05							
FILL: Brown silty sand with gravel, coal, slag, glass		SS	1	33	27	1	62.29
		SS	2	8	3	2	61.29
		SS	3	25	5	3	60.29
3.73		SS	4	42	16		
Grey **SILTY CLAY** with sand		SS	5	42	26	4	59.29
		SS	6	17	2	5	58.29
6.02		SS	7	100			
End of Borehole						6	57.29

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1040 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Breezehill Avenue and Somerset Street West. Geodetic elevation = 63.669m.

REMARKS

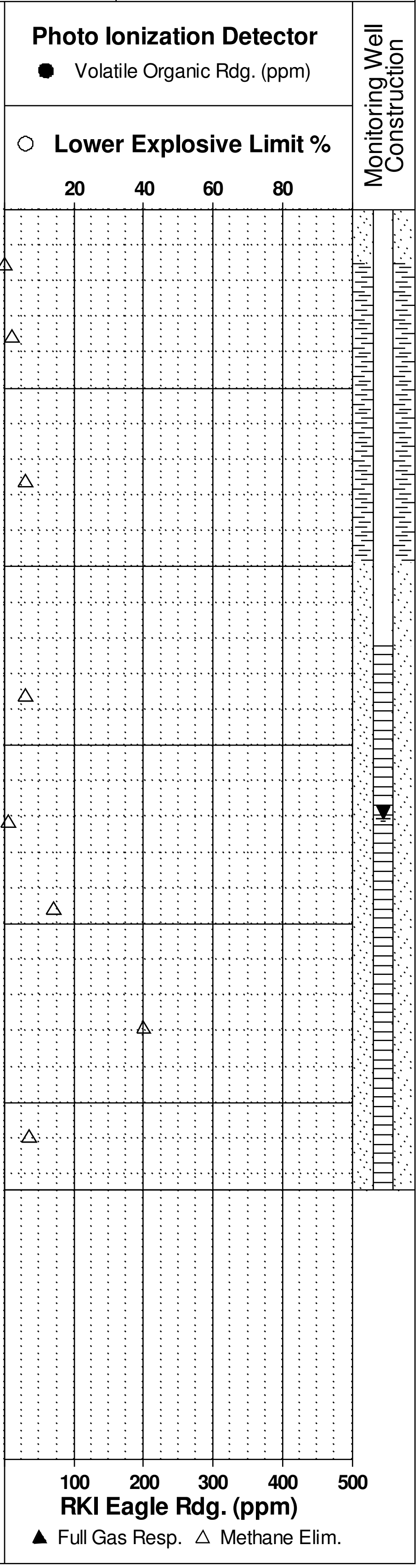
BORINGS BY Jack Hammer

DATE October 27, 2014

FILE NO. **PE2636**

HOLE NO. **BH 5-14**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.46
Concrete 0.20						
FILL: Sand and gravel	AU	1				
	SS	2	29			
					1	62.46
	SS	3	71			
					2	61.46
	SS	4	50			
					3	60.46
3.60	SS	5	50			
Brown **SILTY CLAY** 4.22	SS	6	100			
					4	59.46
Brown **CLAYEY SILT,** some sand and gravel	SS	7	100			
					5	58.46
5.49	SS	8	83			
End of Borehole						
(GWL @ 3.41m-Nov. 11, 2014)						



SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1040 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Breezehill Avenue and Somerset Street West. Geodetic elevation = 63.669m.

REMARKS

BORINGS BY Jack Hammer

DATE October 27, 2014

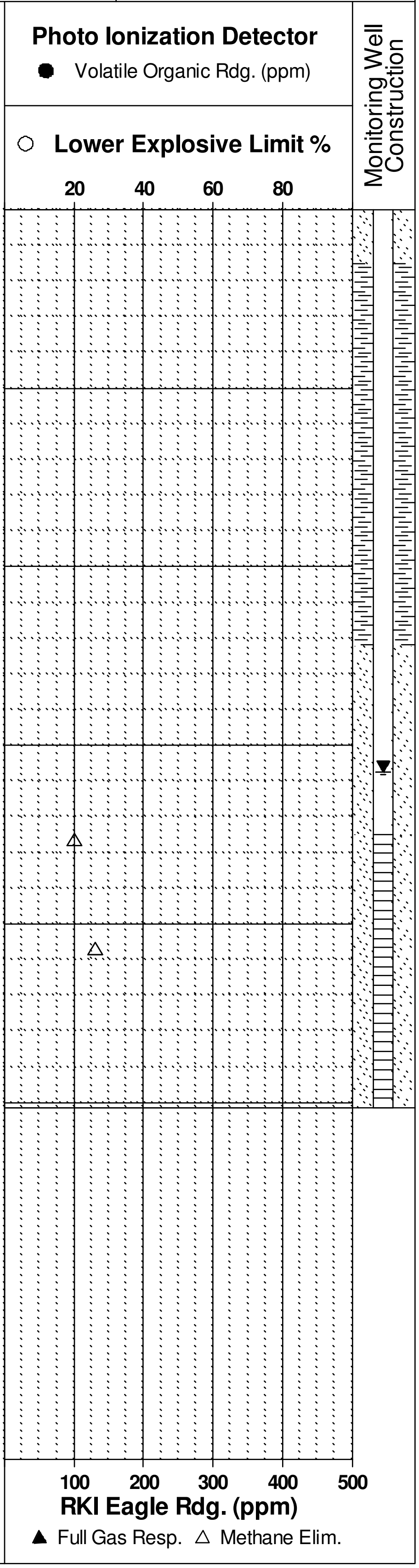
FILE NO. PE2636

HOLE NO. BH 6-14

SOIL DESCRIPTION	
GROUND SURFACE	
Concrete	0.25
FILL: Inferred cobbles and boulders	3.15
FILL: Brown silty sand with gravel	4.11
Brown **SILTY CLAY**	5.03
End of Borehole (GWL @ 3.15m-Nov. 11, 2014)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
RC	1		
RC	2		
SS	3	46	
SS	4	100	

DEPTH (m)	ELEV. (m)
0	63.47
1	62.47
2	61.47
3	60.47
4	59.47
5	58.47



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
1040 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southwest corner of Breezehill Avenue and Somerset Street West. Geodetic elevation = 63.669m.

REMARKS

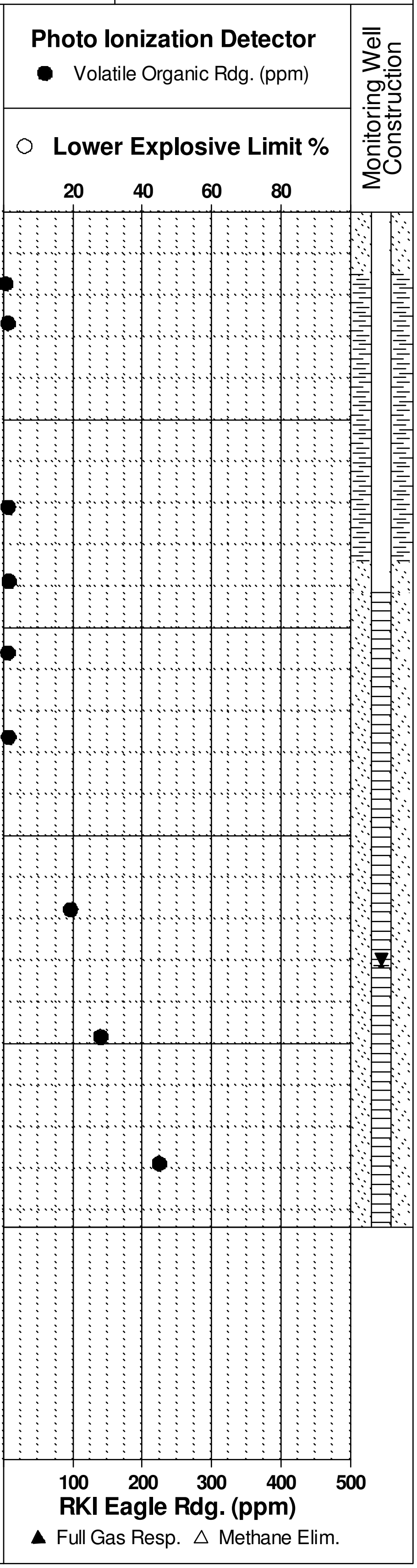
BORINGS BY Portable Drill

DATE February 12, 2015

FILE NO. PE2636

HOLE NO. BH 7-15

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.49
Concrete slab 0.23						
FILL: Crushed stone, trace sand and silt 0.46	AU	1				
	AU	2				
FILL: Blast rock, some sand and gravel, trace silt and clay					1	62.49
	AU	3				
	RC	1	25			
					2	61.49
	AU	4				
		5				
3.00					3	60.49
Brown **SILTY CLAY,** trace sand	SS	6	75			
	SS	7	100		4	59.49
- grey by 4.5m depth	SS	8	100			
4.88						
End of Borehole						
(GWL @ 3.62m - Feb. 19, 2015)						



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SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
1040 Somerset Street West
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 29, 07

FILE NO. PE1148

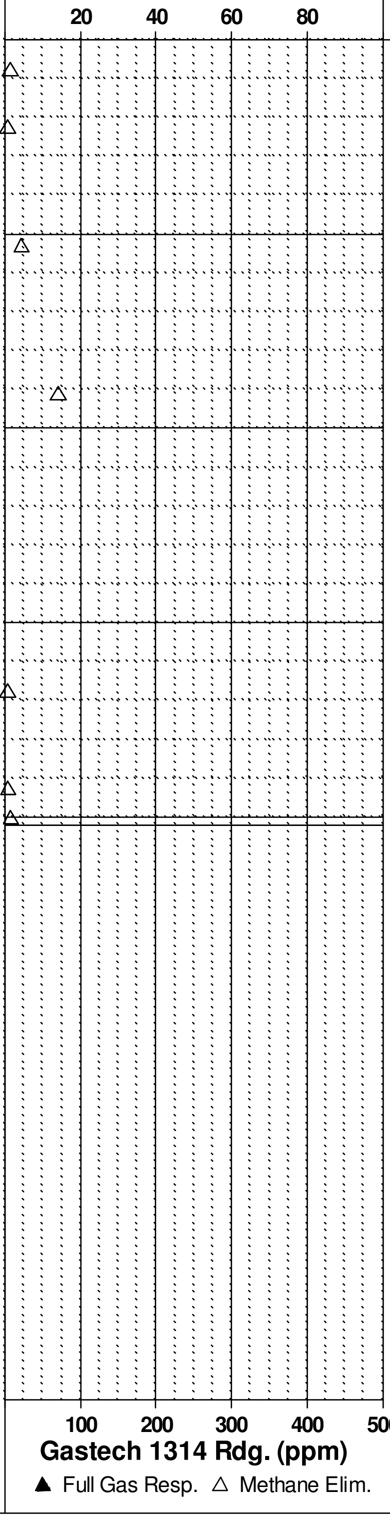
HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
FILL: Brown sandy topsoil with gravel		AU	1				
		AU	2				
0.60							
FILL: Coal		SS	3	25	28	1	
1.68							
FILL: Brown silty sand, some organic matter near the top and with some cobbles by 3.7m depth		SS	4	12	5	2	
						3	
		SS	5	25	35		
		SS	6	27	85+		
4.04		AU	7			4	
End of Borehole Practical refusal to augering @ 4.04m depth							

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

○ **Lower Explosive Limit %**

Monitoring Well Construction



Gastech 1314 Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

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

SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
1040 Somerset Street West
Ottawa, Ontario

DATUM

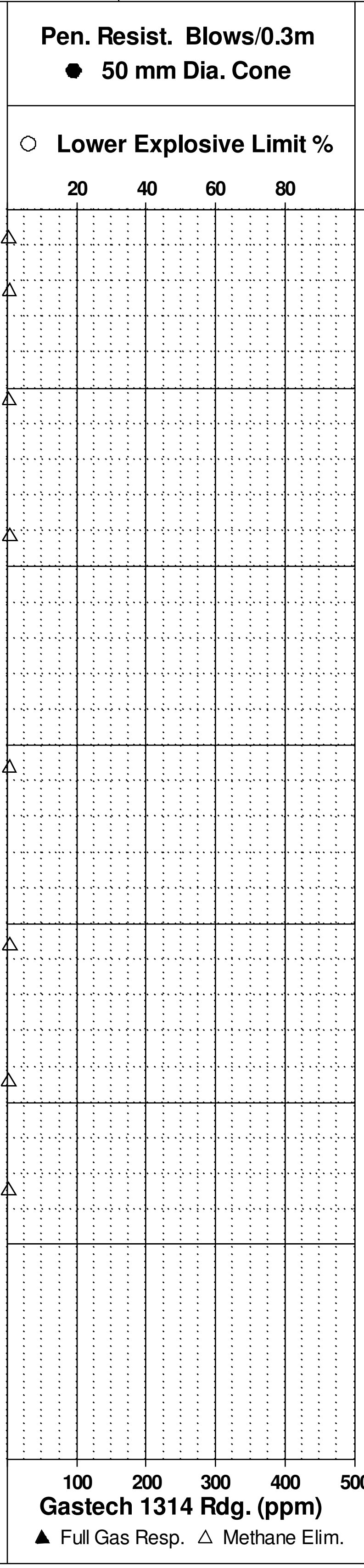
REMARKS

BORINGS BY CME 55 Power Auger **DATE** May 29, 07

FILE NO. PE1148

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
20mm Asphaltic concrete		AU	1				
FILL: Crushed stone 0.60		AU	2				
FILL: Brown silty sand, trace wood pieces		SS	3	25	15	1	
- trace coal by 1.7m depth		SS	4	50	3	2	
- with cobbles by 2.7m depth 3.81		SS	5	40	50+	3	
Loose, grey **CLAYEY SILT** 4.57		SS	6	12	9	4	
Grey **SILTY CLAY**		SS	7	33	4	5	
5.79		SS	8	100	3		
End of Borehole							



Monitoring Well Construction

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
1040 Somerset Street West
Ottawa, Ontario

DATUM

REMARKS

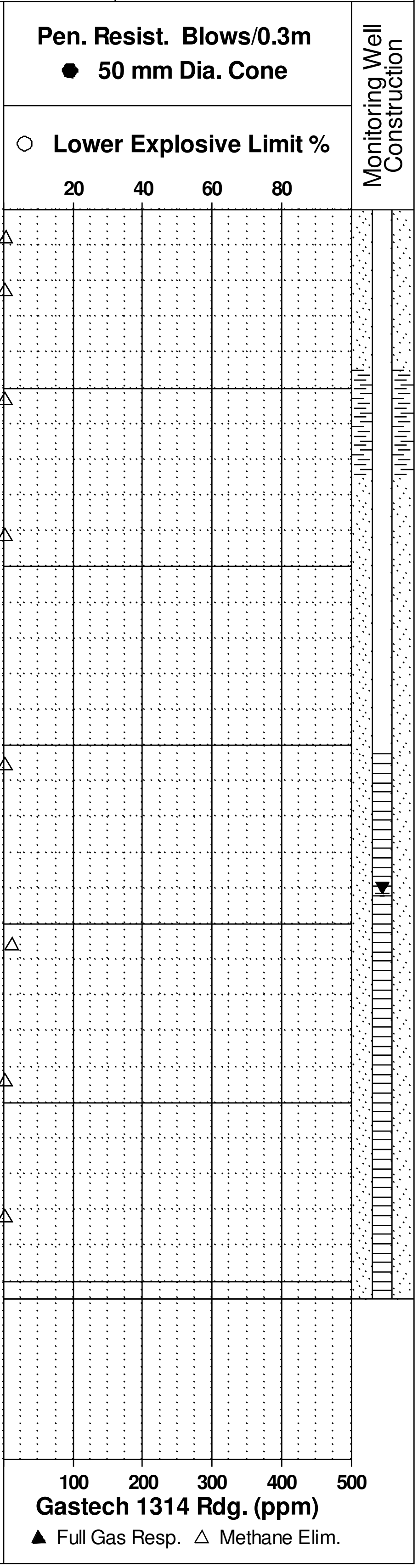
BORINGS BY **CME 55 Power Auger**

DATE **May 29, 07**

FILE NO. **PE1148**

HOLE NO. **BH 3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
20mm Asphaltic concrete over crushed stone **FILL** 0.30		AU	1				
		AU	2				
		SS	3	17	14	1	
FILL: Brown silty sand with gravel, asphalt and brick pieces, occasional coal pieces		SS	4	25	15	2	
3.66		SS	5	25	50+	3	
Grey **SILTY CLAY**, trace sand		SS	6	67	5	4	
		SS	7	100	2	5	
6.10		SS	8	100	1	6	
End of Borehole (GWL @ 3.83m-June 4/07)							



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SOIL PROFILE AND TEST DATA

Phase I-II Environmental Site Assessment
1040 Somerset Street West
Ottawa, Ontario

DATUM

REMARKS

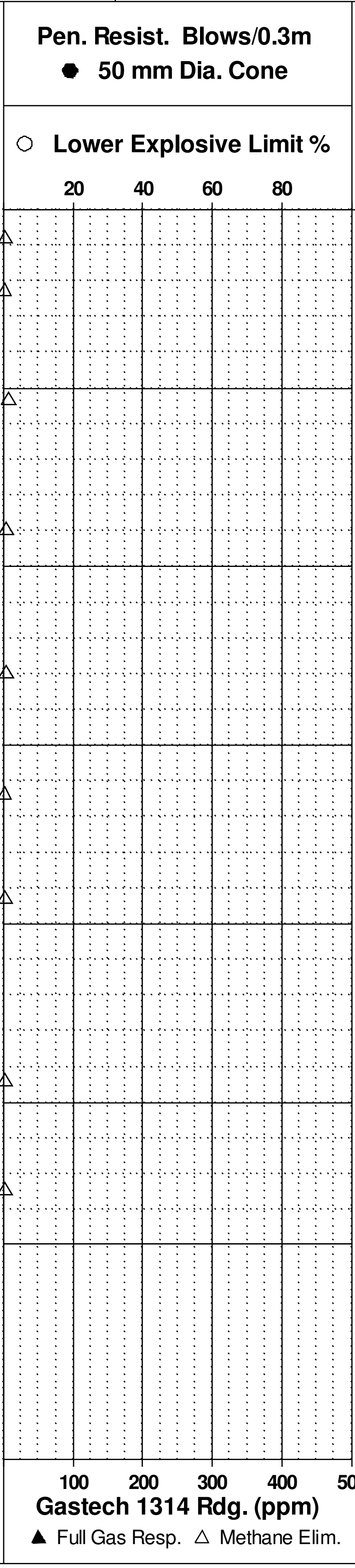
BORINGS BY **CME 55 Power Auger**

DATE **May 29, 07**

FILE NO. **PE1148**

HOLE NO. **BH 4**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
20mm Asphaltic concrete over crushed stone with asphalt 0.30	AU	1				
FILL	AU	2				
	SS	3	25	27	1	
FILL: Brown silty sand with gravel, brick pieces and cobbles with depth, trace coal	SS	4	57	7		
					2	
	SS	5	25	19		
					3	
	SS	6	35	73+		
3.81	SS	7	67	50+		
					4	
Grey **SILTY CLAY**, some sand	SS	8	100	1	5	
	SS	9	100	1		
5.79 End of Borehole						



Monitoring Well 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)

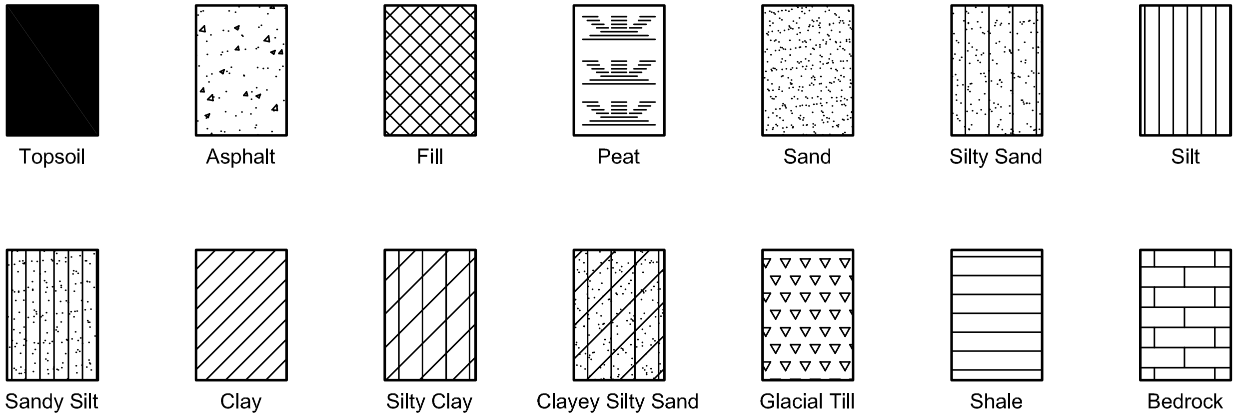
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

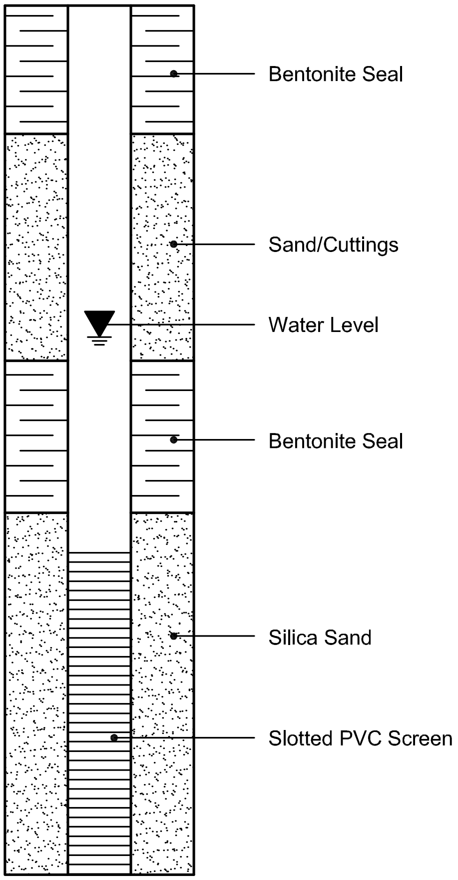
SYMBOLS AND TERMS (continued)

STRATA PLOT

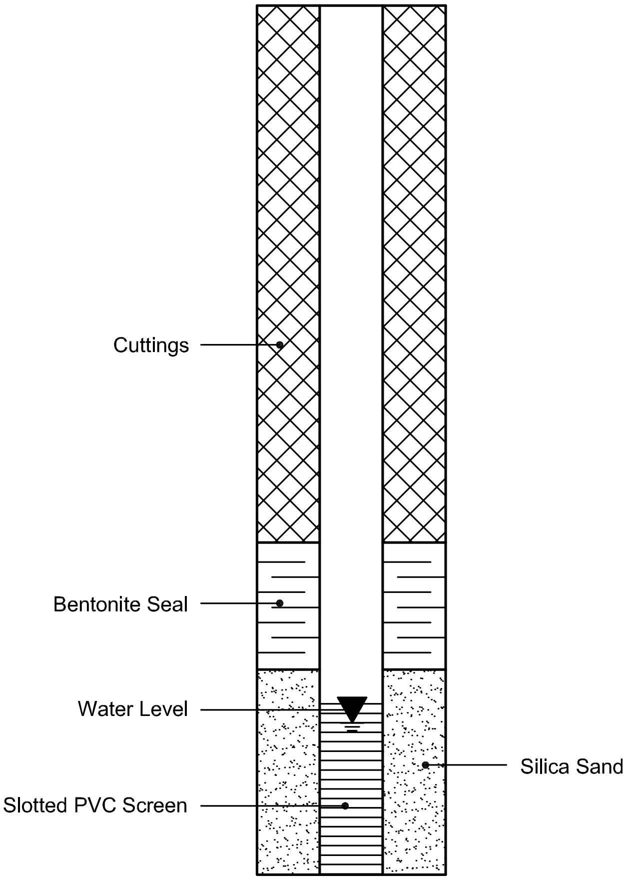


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29707

Report Date: 26-Nov-2020

Order Date: 20-Nov-2020

Project Description: PG2674

	Client ID:	BH 1-20 SS3 5'-7'	-	-	-
	Sample Date:	17-Nov-20 10:00	-	-	-
	Sample ID:	2047666-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	94.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.76	-	-	-
Resistivity	0.10 Ohm.m	21.2	-	-	-
Anions					
Chloride	5 ug/g dry	87	-	-	-
Sulphate	5 ug/g dry	282	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - SHEAR WAVE VELOCITY PROFILE AT SHOT LOCATION +22.6 m

FIGURE 3 - SHEAR WAVE VELOCITY PROFILE AT SHOT LOCATION -0.4 m

FIGURES 4 TO 7 - SLOPE STABILITY SECTIONS

DRAWING PG2674-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

FIGURE 2 – Shear Wave Velocity Profile at Shot Location +22.6 m

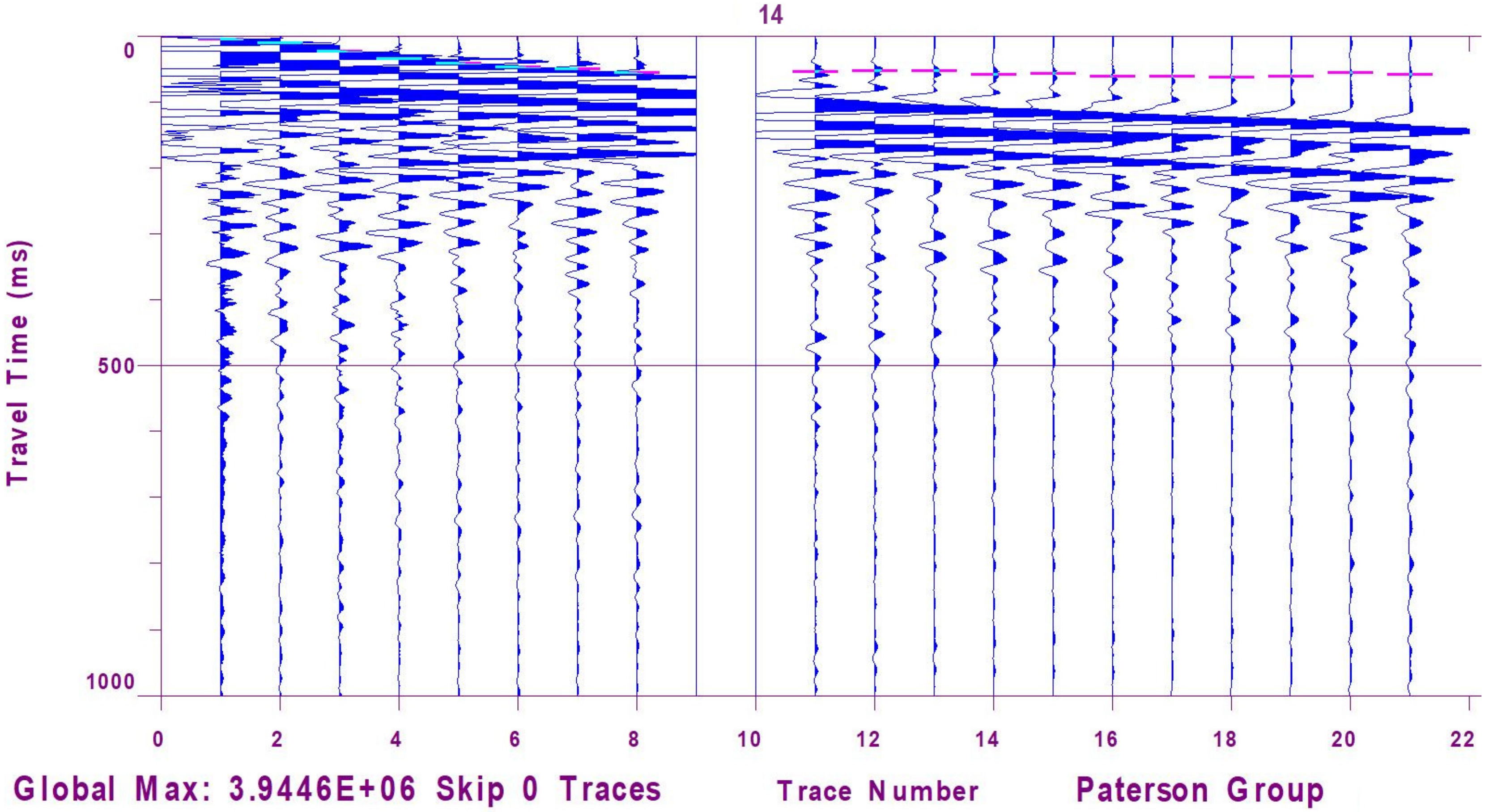


FIGURE 3 – Shear Wave Velocity Profile at Shot Location -0.4 m

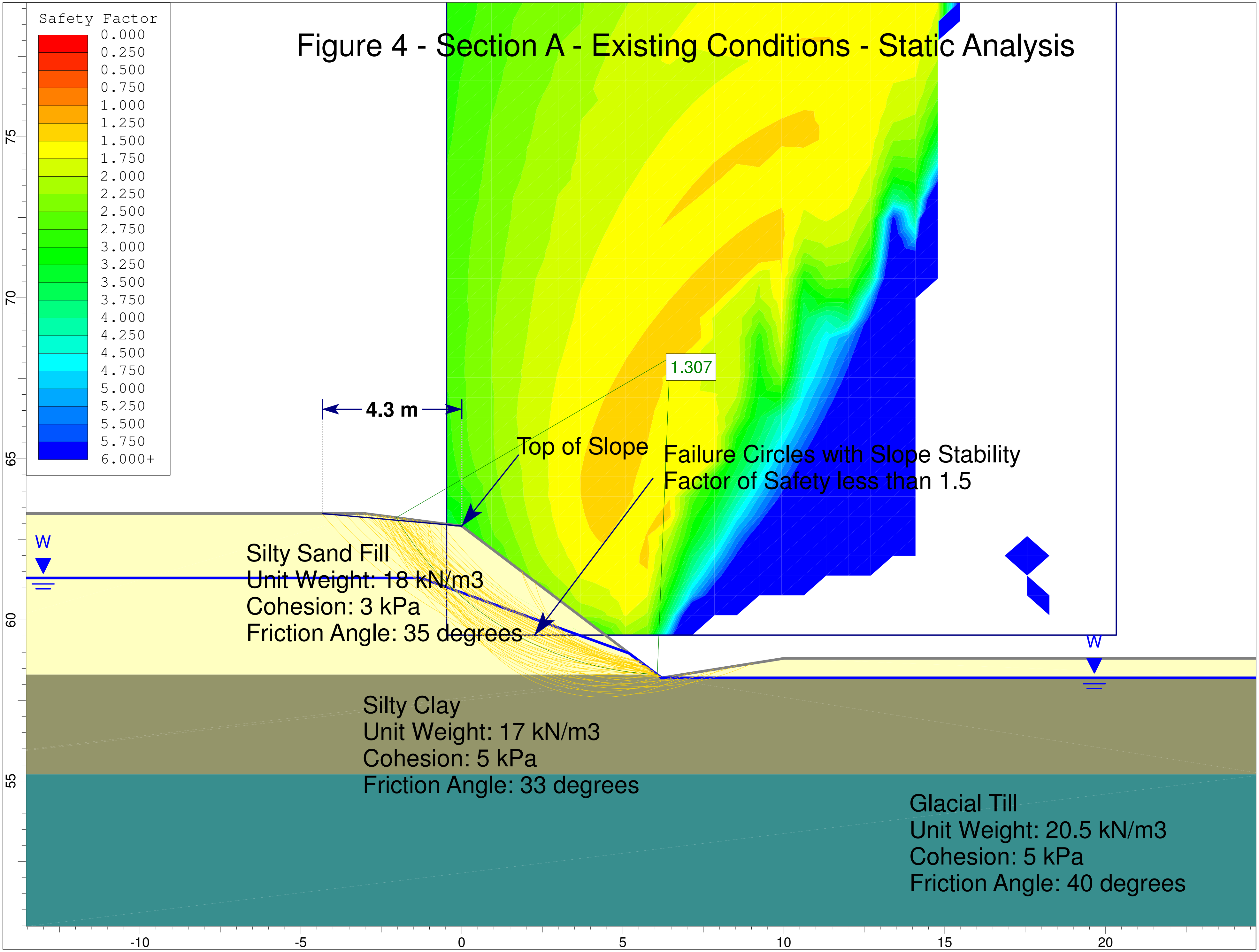


Figure 4 - Section A - Existing Conditions - Static Analysis

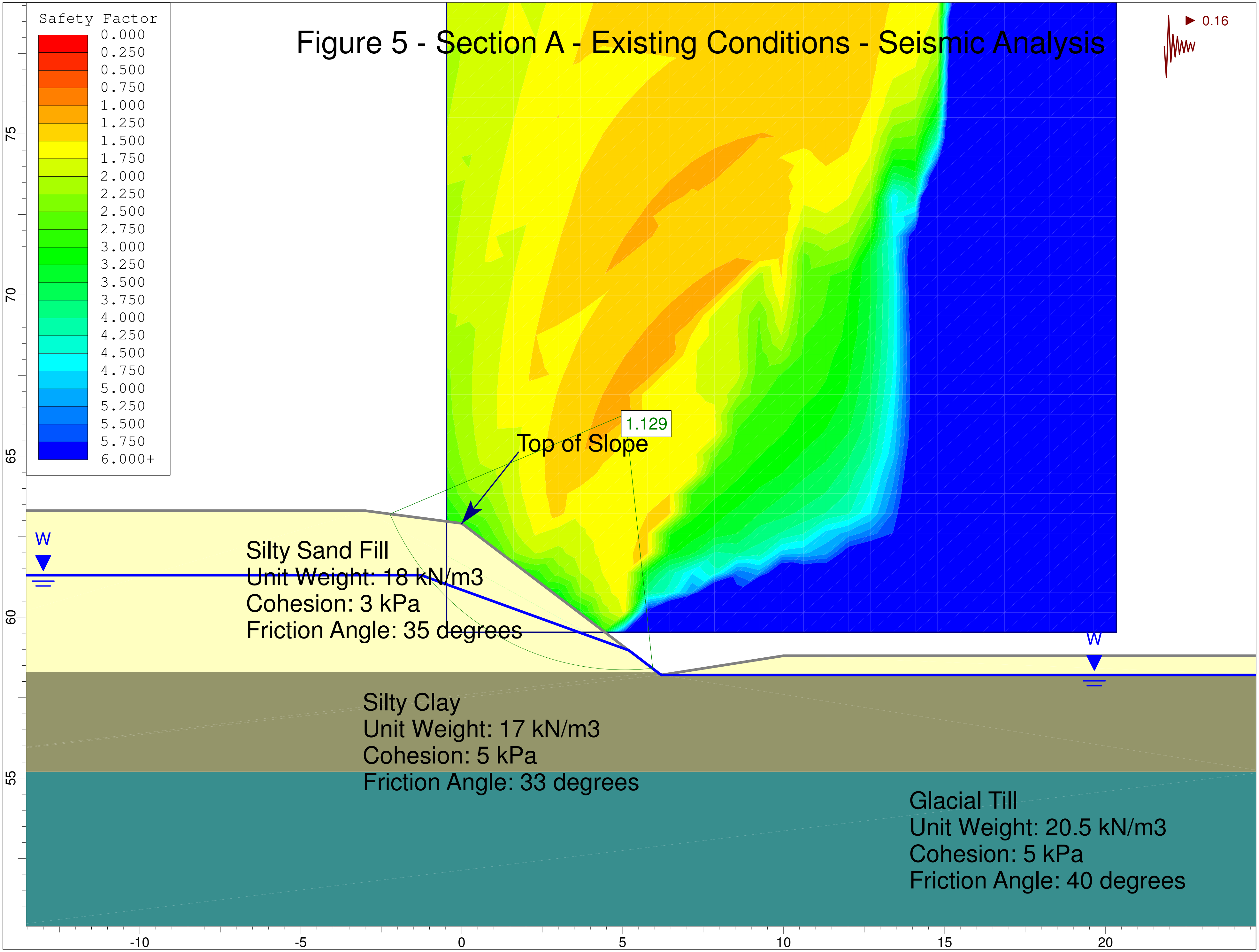


Figure 5 - Section A - Existing Conditions - Seismic Analysis

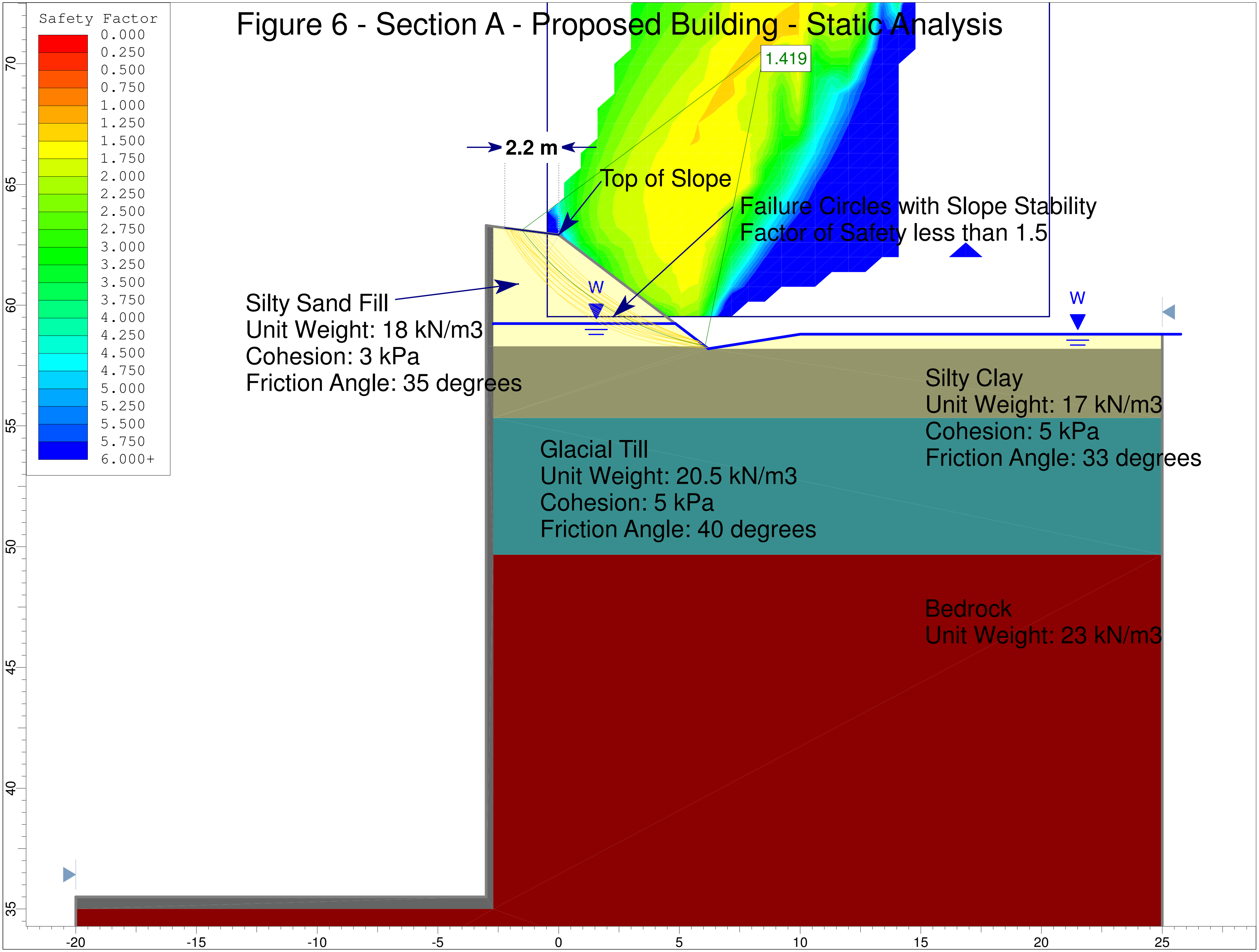


Figure 6 - Section A - Proposed Building - Static Analysis

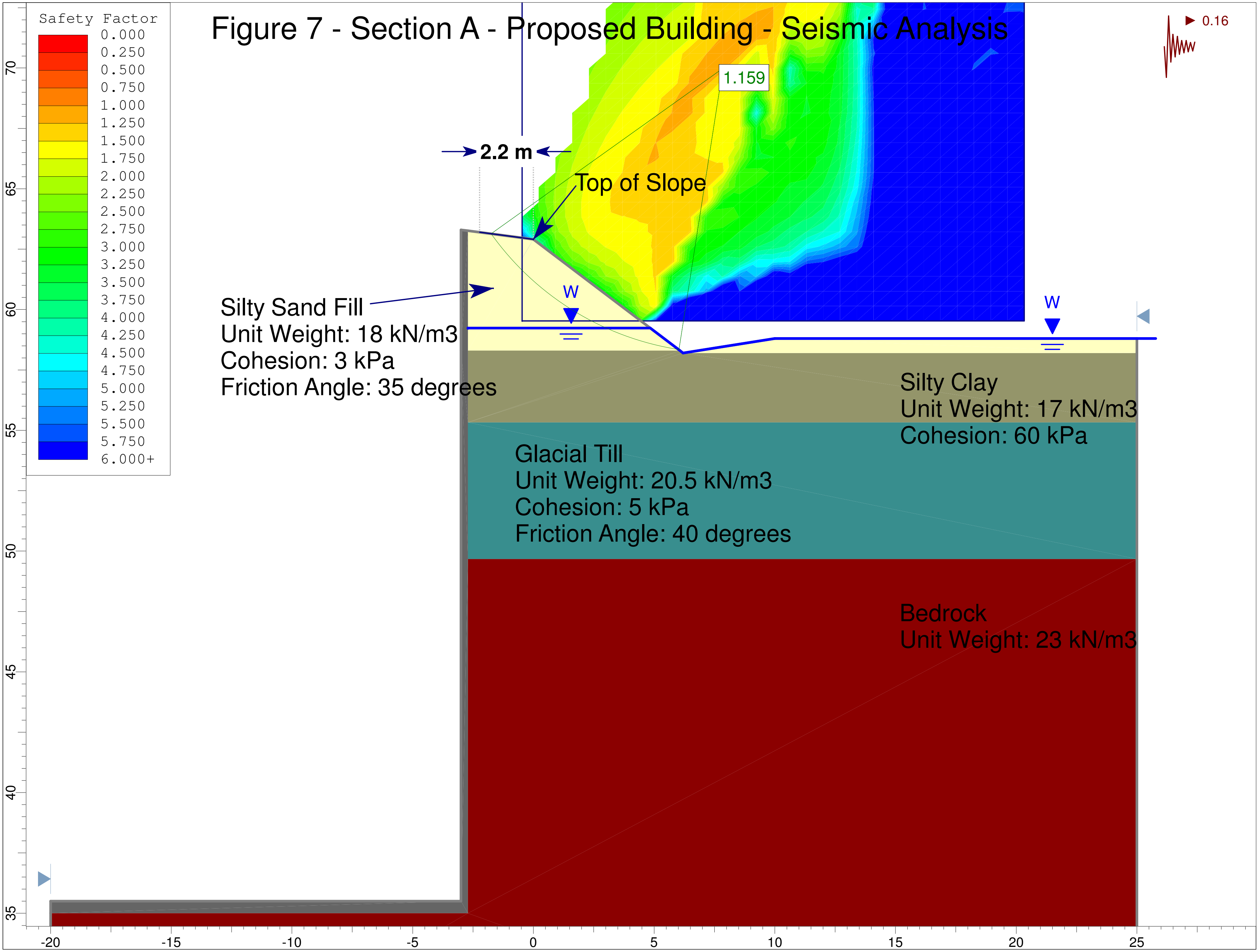
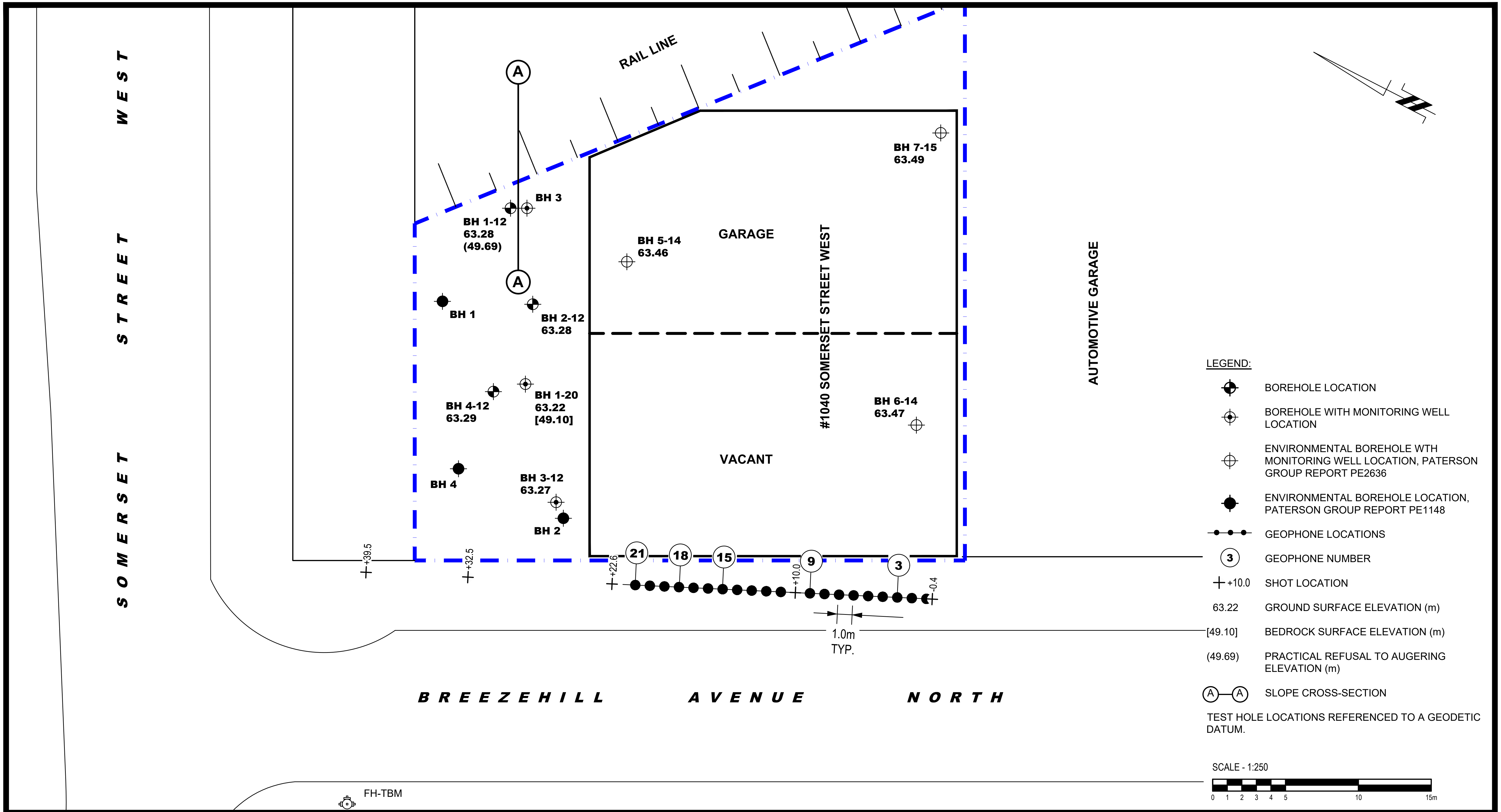


Figure 7 - Section A - Proposed Building - Seismic Analysis



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	ENVIRONMENTAL BOREHOLE WTH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE2636
●	ENVIRONMENTAL BOREHOLE LOCATION, PATERSON GROUP REPORT PE1148
●—●—●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
+10.0	SHOT LOCATION
63.22	GROUND SURFACE ELEVATION (m)
[49.10]	BEDROCK SURFACE ELEVATION (m)
(49.69)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
Ⓐ—Ⓐ	SLOPE CROSS-SECTION

TEST HOLE LOCATIONS REFERENCED TO A GEODETIC DATUM.

patersongroup
consulting engineers

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Ottawa, Ontario K2E 7J5
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NO.	REVISIONS	DATE	INITIAL
3	SHEAR WAVE VELOCITY TEST LOCATION ADDED	24/11/2020	DP
2	NEW BOREHOLE ADDED	19/11/2020	DG
1	NEW ENVIRONMENTAL BOREHOLES ADDED	03/06/2015	CDS

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
1040 SOMERSET STREET WEST

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 04/2014
Drawn by: MPG
Report No.: PG2674-2
Checked by: SD
Drawing No.: PG2674-1
Approved by: DJG
Revision No.: 3

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