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Geotechnical Investigation

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

Prepared For

Claridge Homes

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Report: PG2674-1R

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TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for a 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 current 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.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work for this geotechnical investigation.

2.0 PROPOSED PROJECT

It is understood that the proposed project will consist of a multi-storey building. It is further understood that seven (7) to nine (9) levels of underground parking are anticipated and will occupy the entire site.

The subject site is currently occupied by a commercial building which will be demolished prior to construction.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on April 20, 2012. At that time, four (4) boreholes were advanced to a maximum depth of 13.6 m. Four (4) boreholes were placed across the subject site as part of an previous environmental investigation. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG2674-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-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 recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to our laboratory for further review. 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.

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 3-12 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 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.

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject site is occupied by a commercial building with an associated access lane and parking areas. The site is generally at grade with Breezehill Avenue North and slightly below grade of Somerset Street West. The portion of Somerset Street West that borders the subject site to the north consists of the west landing for an elevated bridge over an OC Transpo railway line. Also, an OC Transpo railway corridor borders the east side of the subject site, which crosses below Somerset Street West in a north-south direction.

The top of an approximately 5 m high slope is located along the east property boundary of the subject site. The ground surface across the slope is noted to be brush covered with some construction debris noted at surface.

4.2 Subsurface Profile

The subsurface profile at the borehole locations consists of asphaltic concrete and/or a silty sand fill layer. A silty clay layer underlain by a glacial till was encountered at all of the borehole locations. A thin layer of silty sand was encountered at BH 1-12 underlying the silty sand fill layer. Practical refusal to excavation was encountered at BH 1-12 at a 13.6 m depth. Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded shale and limestone of the Verulam Formation.

4.3 Groundwater

A groundwater level reading was taken in the monitoring wells installed at BH 3-12 and BH 3 on May 8, 2012. A groundwater level of 3.49 m depth at BH 3-12 and a groundwater level of 3.60 m depth at BH 3 (previous Environmental Study) was recorded at that time. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 DISCUSSION

5.1 Geotechnical Assessment

The subject site is considered adequate from a geotechnical viewpoint, for the proposed building. It is anticipated that the proposed multi-storey building will be founded on shallow footings placed on clean, surface sounded bedrock.

Considering the close proximity of the west landing of the Somerset Street overpass to the subject site, the shoring design will have to include support for the foundation west landing. It is recommended that the foundation dimensions and location of the footing for the west landing be determined to ensure the shoring does not encounter the west landing foundation. Consideration should be given to using a diaphragm “slurry” wall to act as a temporary shoring system, which is capable of supporting the west landing foundation. The diaphragm walls will also act as the permanent foundation wall for the proposed building above the bedrock surface.

Due to the close proximity of the adjacent 1375 mm diameter watermain, which is located less than 5 m from the west property boundary along Breezehill Avenue, additional precautions should be taken during excavation activities to ensure the existing service is not disturbed. The temporary shoring design should utilize an interlocking sheet pile style shoring to ensure that supported soils do not move laterally, which could be detrimental to the existing watermain. A detailed monitoring program is presented in Subsection 5.2 to monitor soil movement and vibrations associated with the shoring program.

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 anticipated that all existing overburden material will be excavated from within the footprint of 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.

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 (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

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

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

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 as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipments could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of this equipment. Vibrations, whether it is 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 diaphragm wall and bedrock blasting program to ensure that the neighbouring OC Transpo railway line is 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 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 two (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. Any movement greater than 2 mm will be investigated immediately. A trigger movement of 6 mm will require immediate 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 1 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 re-commence.

The following vibration limits are recommended for the shoring/excavation operation to be completed adjacent to the 1,375 mm diameter watermain.

Table 1 - Vibration Limits for Work Completed Adjacent to 1,375 mm Watermain

Location of Vibration Monitor	**Peak Particle Velocity (mm/s)**	**Frequency (Hz)**
Inside the Valve Chamber	15	4 to 12
	50	>40
At Ground Surface (within 3 m of watermain)	10	4 to 12
	35	>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. Any mitigation measures contemplated for implementation will be discussed with City personnel.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean surface sounded bedrock surface at the proposed founding elevation 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 was applied to the bearing resistance value at ULS and a bearing resistance value at serviceability limit states (SLS) of **2,000 kPa**.

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

A factored bearing resistance value at ULS of **6,000 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance at SLS of **3,000 kPa** could be used if founded on limestone bedrock and 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.

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).

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.

Diaphragm Wall Foundation

The exterior foundation walls of the multi-storey structure will be founded on steel reinforced diaphragm walls that are socketed within the bedrock along the perimeter of the parking garage. It is anticipated that the interior portion of the building (shear walls, elevator shafts, stairwells and other portions of the structure selected by the structural engineer) will be founded by shallow footings placed directly on the bedrock surface.

Table 2 - Bearing Resistance Values for Diaphragm Walls Socketed Within Bedrock

Socketed Length (m)	Width of Diaphragm Wall (mm)	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
1	600	2000	3500
	750	2000	3500
2	600	3000	4500
	750	3000	4500

5.4 Design for Earthquakes

It is understood that several levels of underground parking are anticipated for the proposed building. It is expected that the proposed building foundation will be placed on a bedrock bearing surface. Therefore, a seismic site **Class A** can be used for design if a site specific shear wave velocity test is completed. A site specific shear wave velocity test is required to confirm the seismic site class according to the OBC 2006. If shear wave velocity testing is not completed, a seismic site Class C is applicable for the design purposes. It should be further noted that 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 stone under the lower basement floor.

5.6 Basement Wall

It is understood that the 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 0.25 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). 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 (2) distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two (2) 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 soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

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

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

Seismic 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 0.42g according to OBC 2006. 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 2006.

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 should be further noted that centre to centre spacing between bond lengths be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also 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	1.5	4.5	1000
125	1.1	0.5	1.6	250
	1.5	0.7	2.2	500
	2.6	1	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
150	**BASE** - 20 mm clear stone
	SUBGRADE - Bedrock

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 weathered 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

A properly executed diaphragm wall, which is sealed at vertical panel joints, horizontal cold joints, and tie-back insert locations can be considered a waterproofed system. It is understood that the building foundation walls below the bedrock surface will be placed in close proximity to all the boundaries. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be poured against a drainage system placed against the bedrock face.

It is recommended that the composite drainage system (such as Miradrain G100N or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration due to groundwater lowering within the bedrock. For design purposes, we recommend that 100 or 150 mm in perforated pipes be placed at 3 to 4.5 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.

6.2 Protection of Footings Against Frost Action

It is expected that the parking garage 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. It is expected that sufficient room will be available to permit the building excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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

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 may be required for the overburden soil to complete the required excavations if a diaphragm wall construction method will not be used for the proposed building. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. 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. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure 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.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

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.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. Because the depth at which the apex 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 capacity, than one where the bonded length was just the bottom part of the overall anchor.

The design of the rock anchors for temporary shoring can be based on the values provided in Subsection 5.7 of the present report.

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

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

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

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

Diaphragm Wall System

For design purposes, the earth pressure acting on a slurry wall shoring system can be estimated using a trapezoidal earth pressure envelope with a maximum pressure of $0.3 \cdot \gamma \cdot H$ for strutted or anchored shoring. The earth pressure will be zero for the top and bottom of the excavation, and will increase to the maximum which occurs at $0.25 \cdot H$ from both the bottom and the top of the excavation. The earth pressure distribution can also be estimated using an earth pressure coefficient and a quasi-hydrostatic distribution.

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

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations (i.e. below the groundwater level).

The excavation of a diaphragm wall should carried out in sections or panels with the excavation filled with a bentonite-rich slurry to provide adequate support for the trench walls. Once the excavation is complete, reinforcing may be installed, if required, and concrete can be poured from the bottom of the excavation using tremie methods.

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.

A temporary MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. At least 3 to 4 months should be allowed for completion of the application and issuance of the permit by the MOE.

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 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 of a silty sand fill mixed with clay. 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 0.21G 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 overturning of the existing slope.

The results of the stability analysis for the existing static conditions at Section A are presented in Figure 2 in Appendix 2. The factor of safety for the slope was less than 1.5 for Section A. Figure 4 presents our analysis, which includes the proposed building footprint. The factor of safety for the slope 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 in Figures 3 and 5 for the slope sections. The results indicate that the factor of safety for the sections are greater than 1.1 for all the sections.

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.

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

☐ Observation of all subgrades prior to backfilling.

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

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

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

8.0 STATEMENT OF LIMITATIONS

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

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

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

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
D. J. GILBERT
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PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng.

Report Distribution:

- ☐ Claridge Homes (3 copies)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

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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

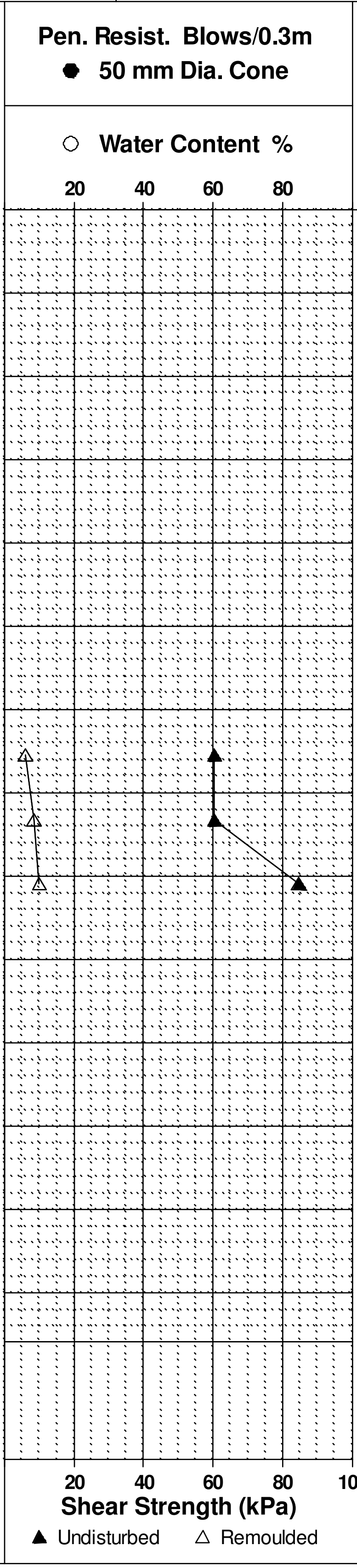
DATE April 20, 2012

FILE NO. PG2674

HOLE NO. BH 1-12

SOIL DESCRIPTION	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
	SS	4	100	50+
3.70				
Very loose, grey **SILTY SAND** with clay	SS	5	83	3
	SS	6	92	1
5.25				
Stiff, grey **SILTY CLAY**	SS	7	100	1
	SS	8	100	P
Stiff, grey **SILTY CLAY**	SS	9	100	P
	SS	10	100	P
8.30				
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
	SS	17	80	50+
13.59				
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

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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

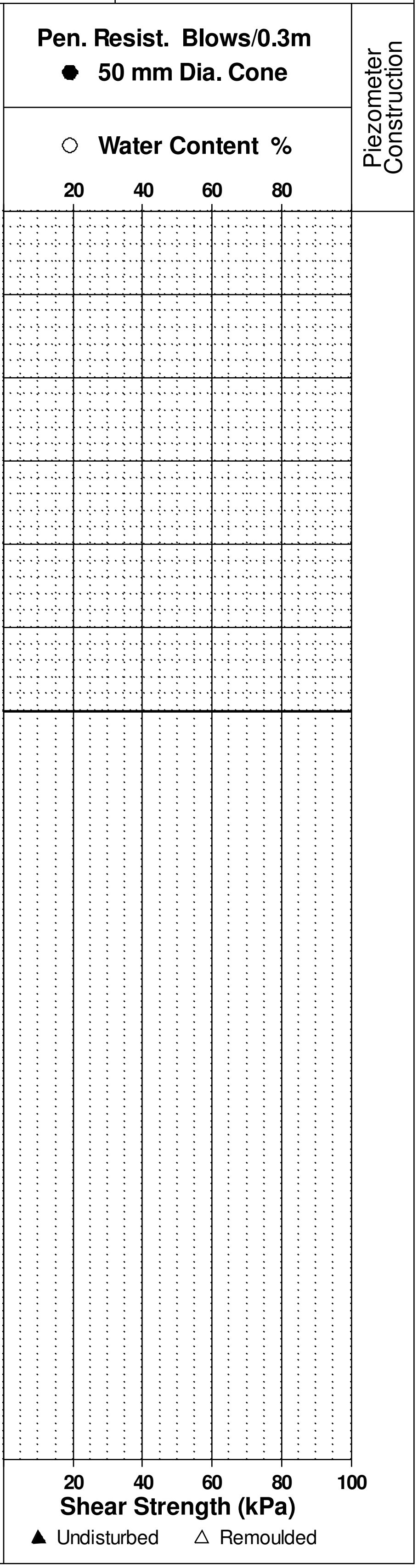
BORINGS BY CME 55 Power Auger

DATE May 3, 2012

FILE NO. PG2674

HOLE NO. BH 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 3.73		SS	2	25	9	2	61.28
		SS	3	33	9		
						3	60.28
		SS	5	42	0		
Grey **SILTY CLAY** with sand 6.02		SS	5	50	3	4	59.28
		SS	6	92	3	5	58.28
		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

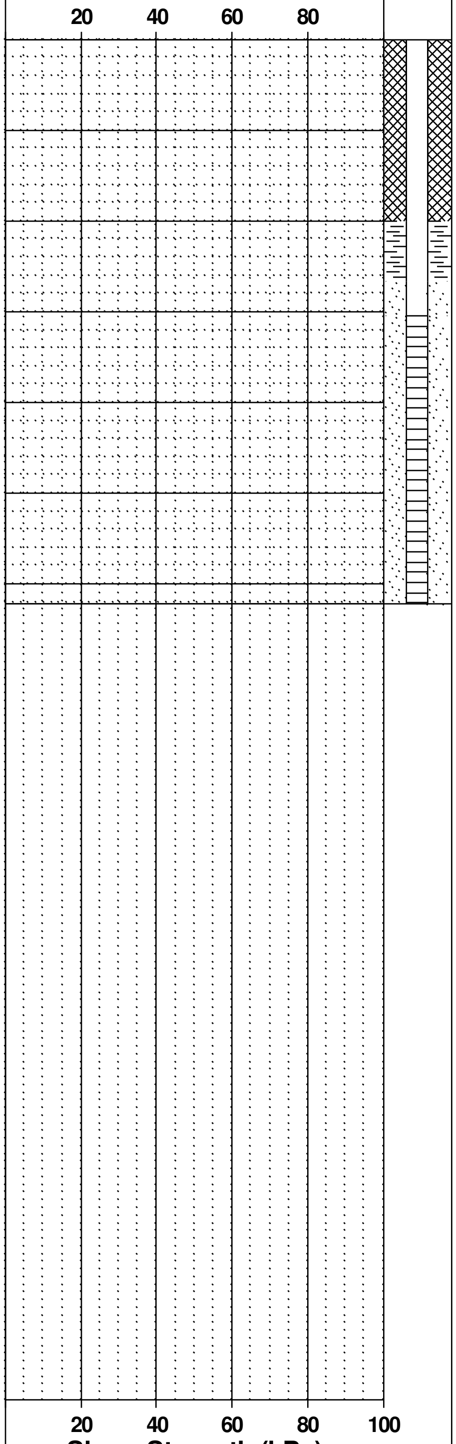
HOLE NO. BH 3-12

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone	0.25				
FILL: Brown silty sand with gravel, coal and concrete		SS	1	33	29
	2.21	SS	2	8	3
FILL: Brown silty sand with gravel and cobbles		SS	3	33	14
	3.73	SS	4	42	81
Grey **SILTY CLAY,** trace sand		SS	5	58	6
		SS	6	67	2
	6.22	SS	7	100	3
End of Borehole					

DEPTH (m)	ELEV. (m)
0	63.27
1	62.27
2	61.27
3	60.27
4	59.27
5	58.27
6	57.27

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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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		6	57.29
End of Borehole							

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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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

FILE NO. PE1148

HOLE NO. BH 1

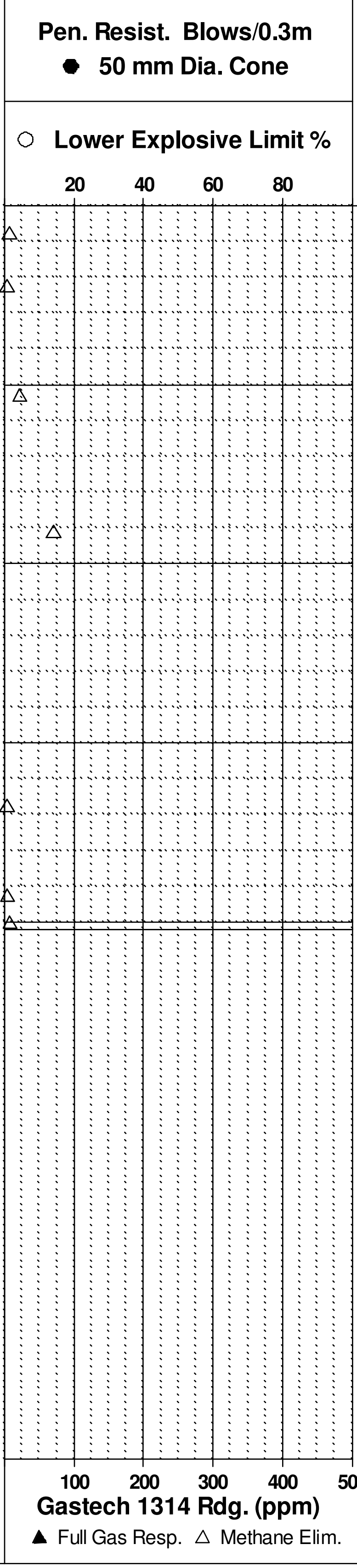
DATUM

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 29, 07

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				
0.60	▨	AU	2				
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							



Monitoring Well Construction

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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

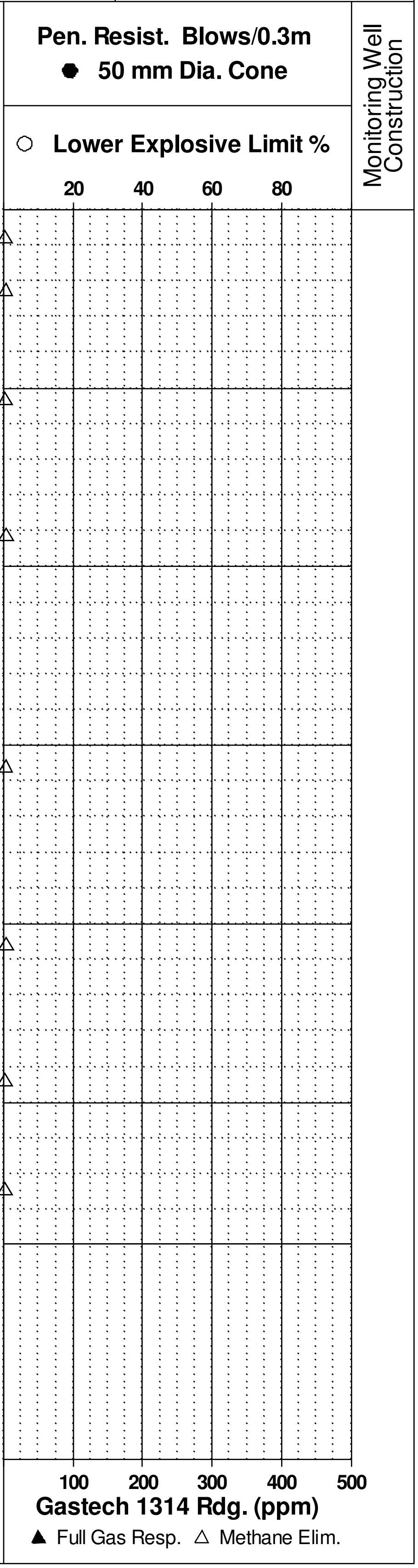
BORINGS BY CME 55 Power Auger

DATE May 29, 07

FILE NO. PE1148

HOLE NO. BH 2

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
20mm Asphaltic concrete	AU	1				
FILL: Crushed stone	AU	2				
0.60						
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	SS	5	40	50+	3	
3.81						
Loose, grey **CLAYEY SILT**	SS	6	12	9	4	
4.57						
Grey **SILTY CLAY**	SS	7	33	4	5	
	SS	8	100	3		
5.79						
End of Borehole						



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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

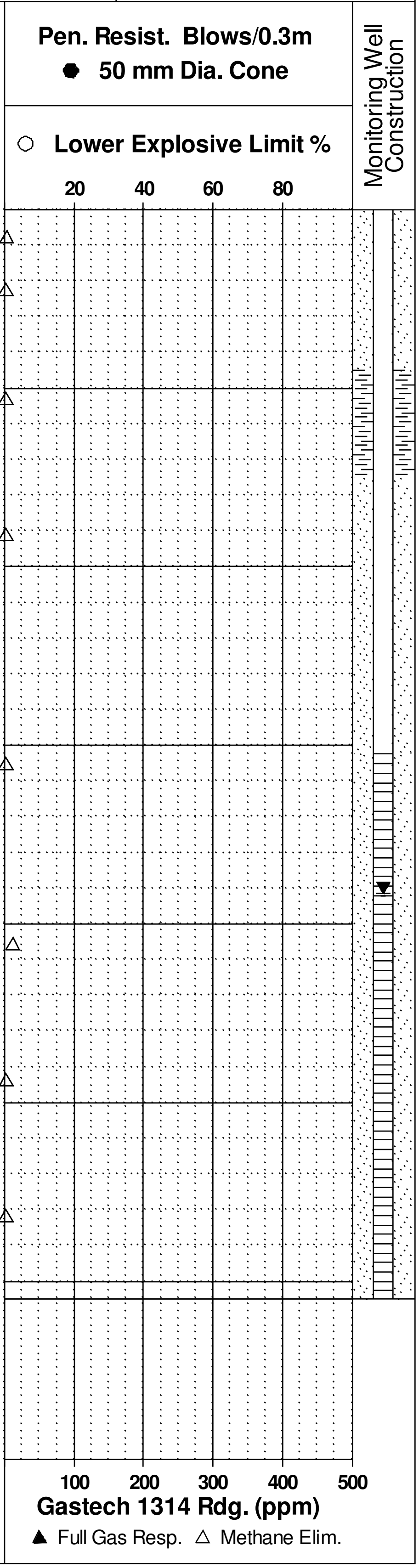
BORINGS BY **CME 55 Power Auger**

DATE **May 29, 07**

FILE NO. **PE1148**

HOLE NO. **BH 3**

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
20mm Asphaltic concrete over crushed stone **FILL**	0.30	AU	1				
FILL: Brown silty sand with gravel, asphalt and brick pieces, occasional coal pieces		AU	2				
		SS	3	17	14	1	
		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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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**

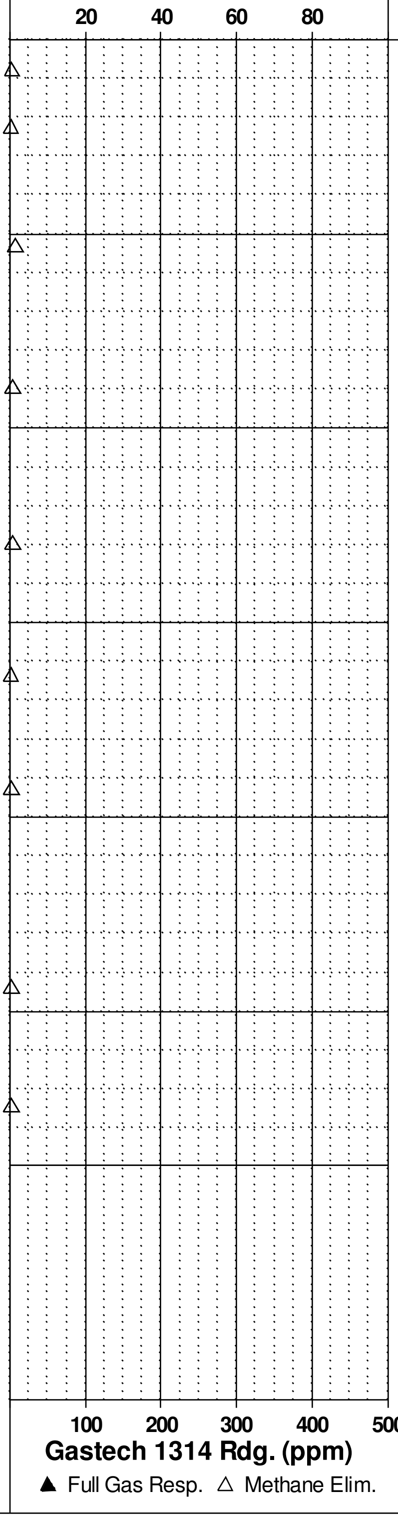
FILE NO. **PE1148**

HOLE NO. **BH 4**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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						

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

○ Lower Explosive Limit %



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 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

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

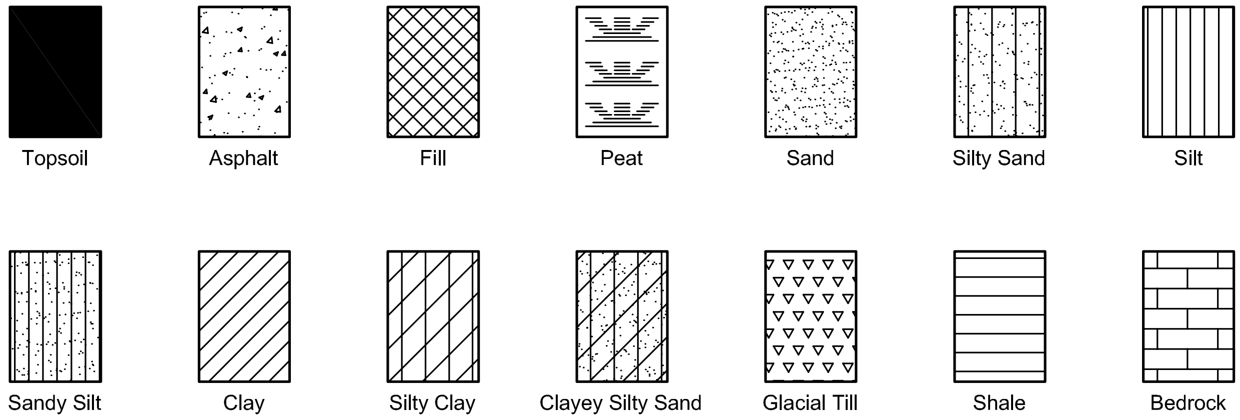
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

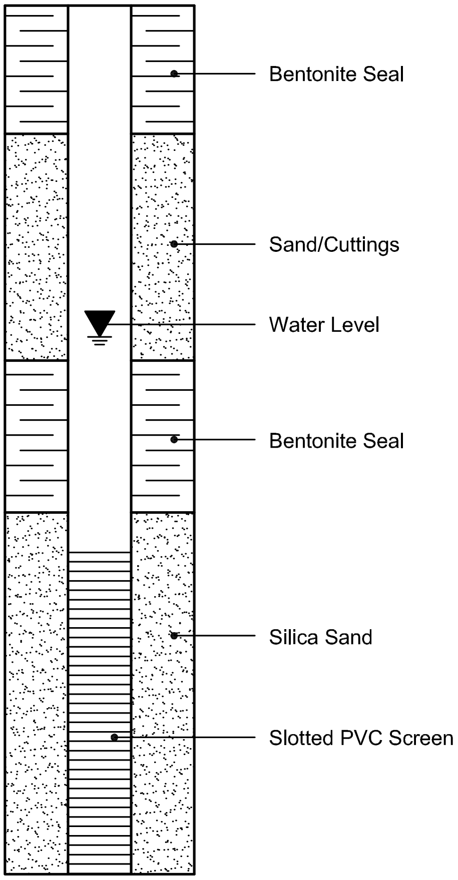
SYMBOLS AND TERMS (continued)

STRATA PLOT

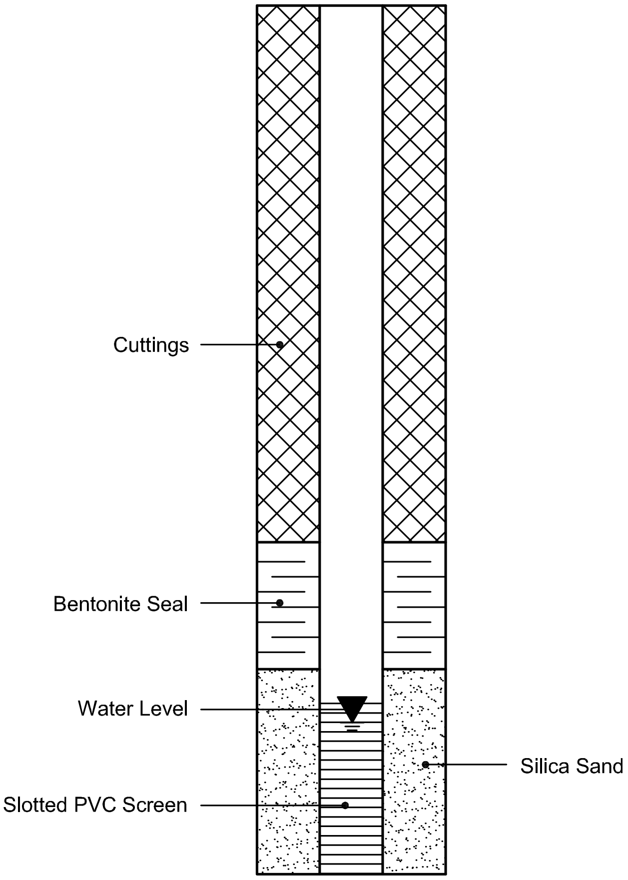


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 5 - SLOPE STABILITY SECTIONS

DRAWING PG2674-1 - TEST HOLE LOCATION PLAN

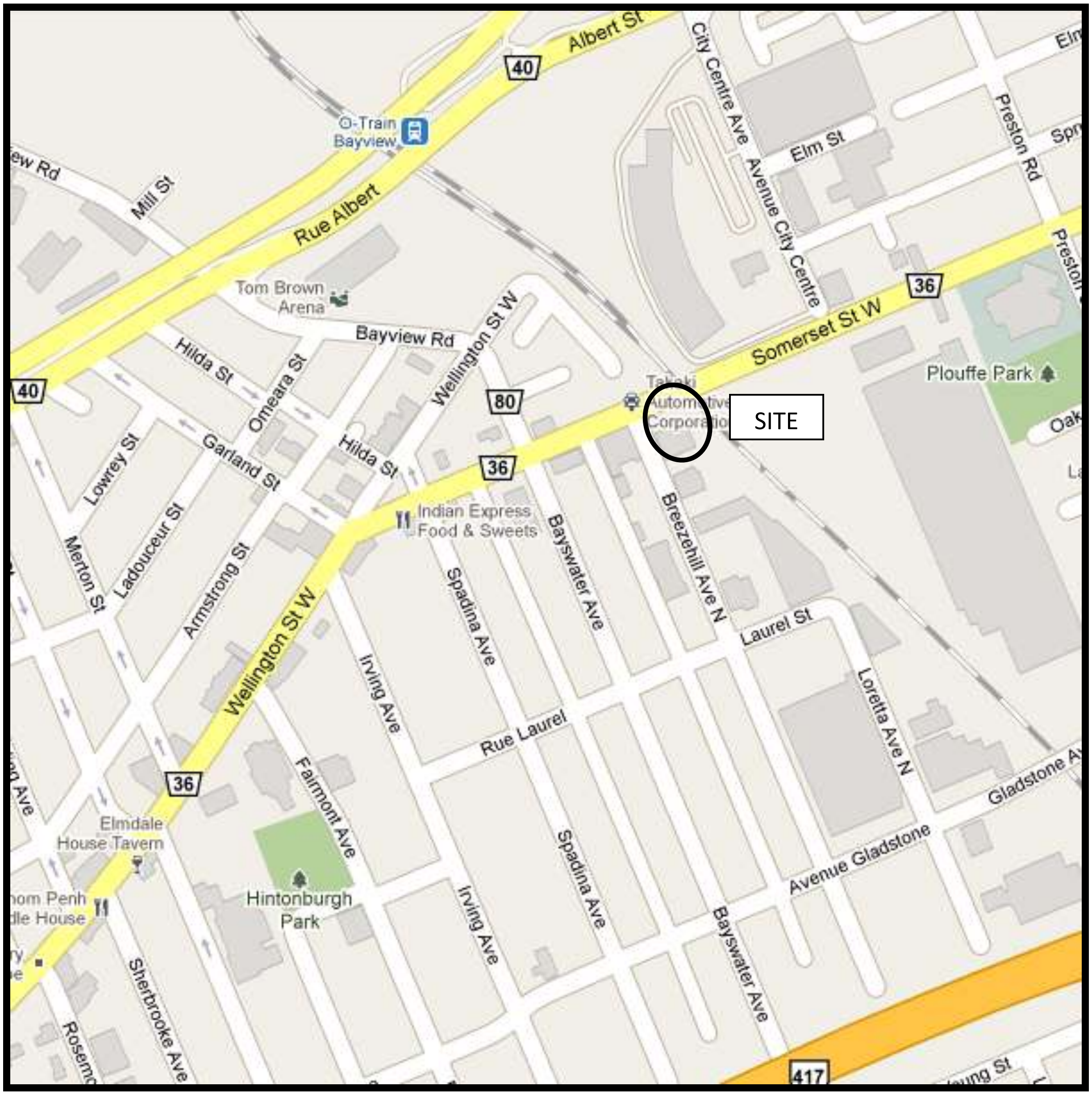


FIGURE 1

KEY PLAN

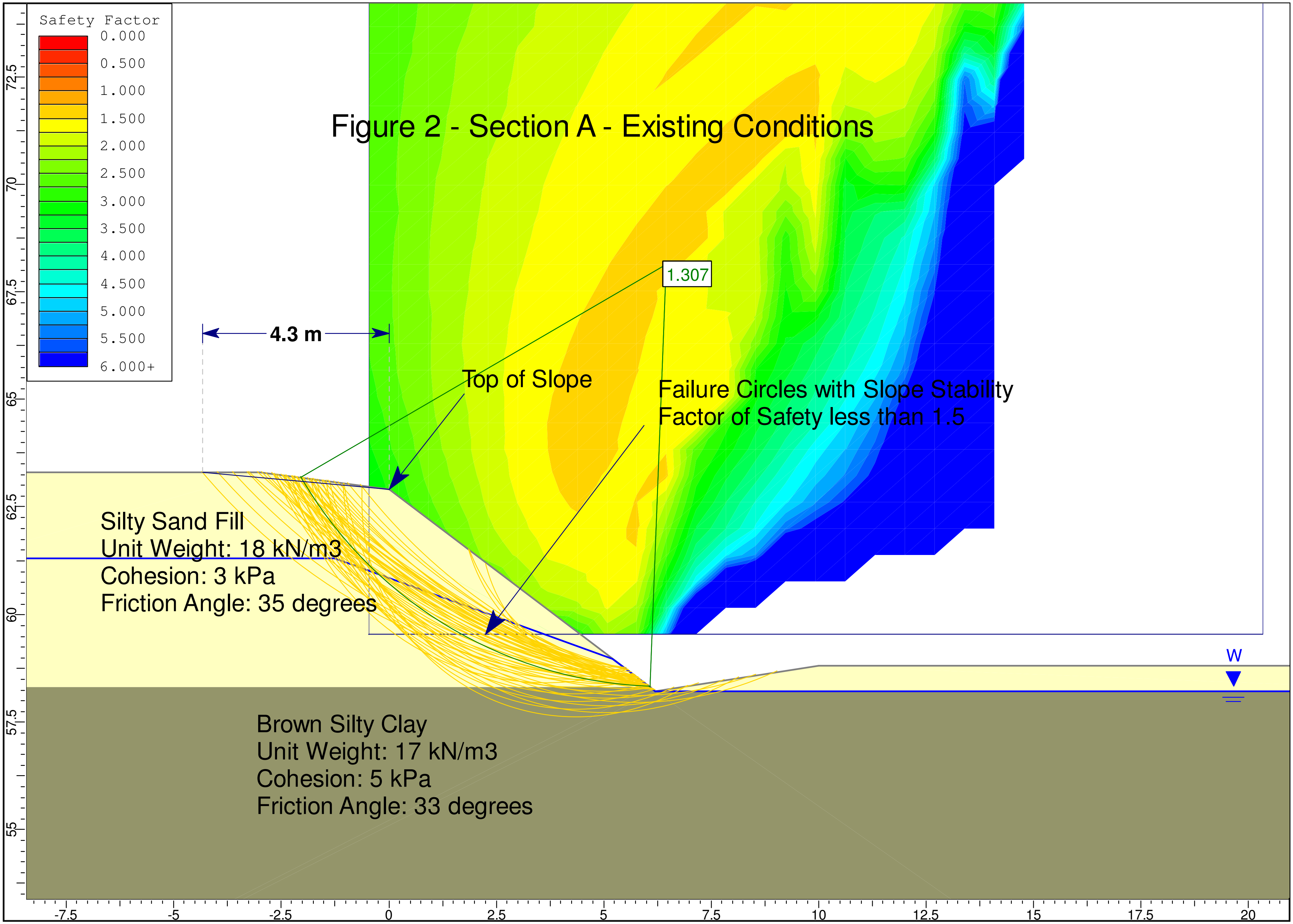


Figure 2 - Section A - Existing Conditions

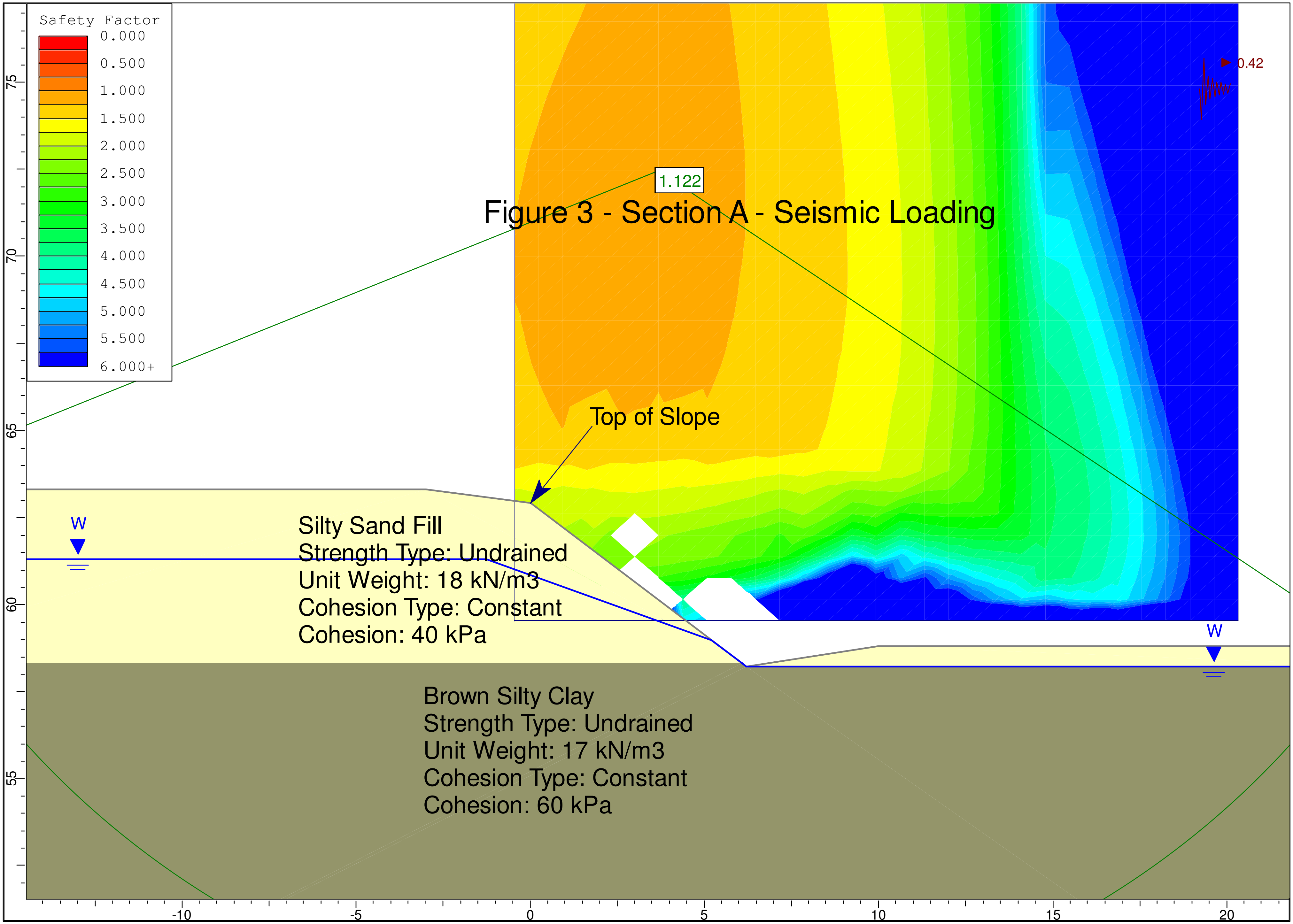


Figure 3 - Section A - Seismic Loading

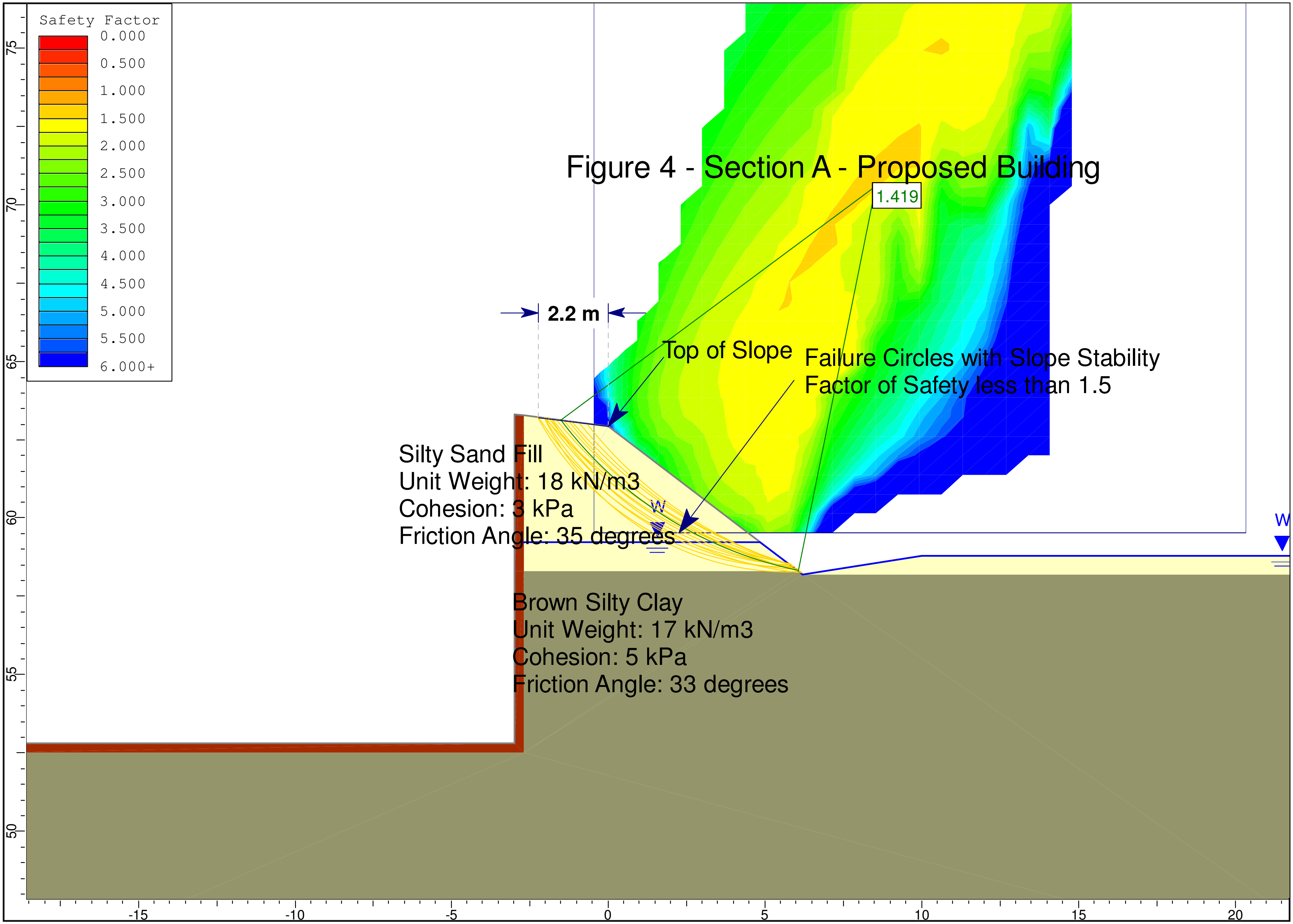


Figure 4 - Section A - Proposed Building

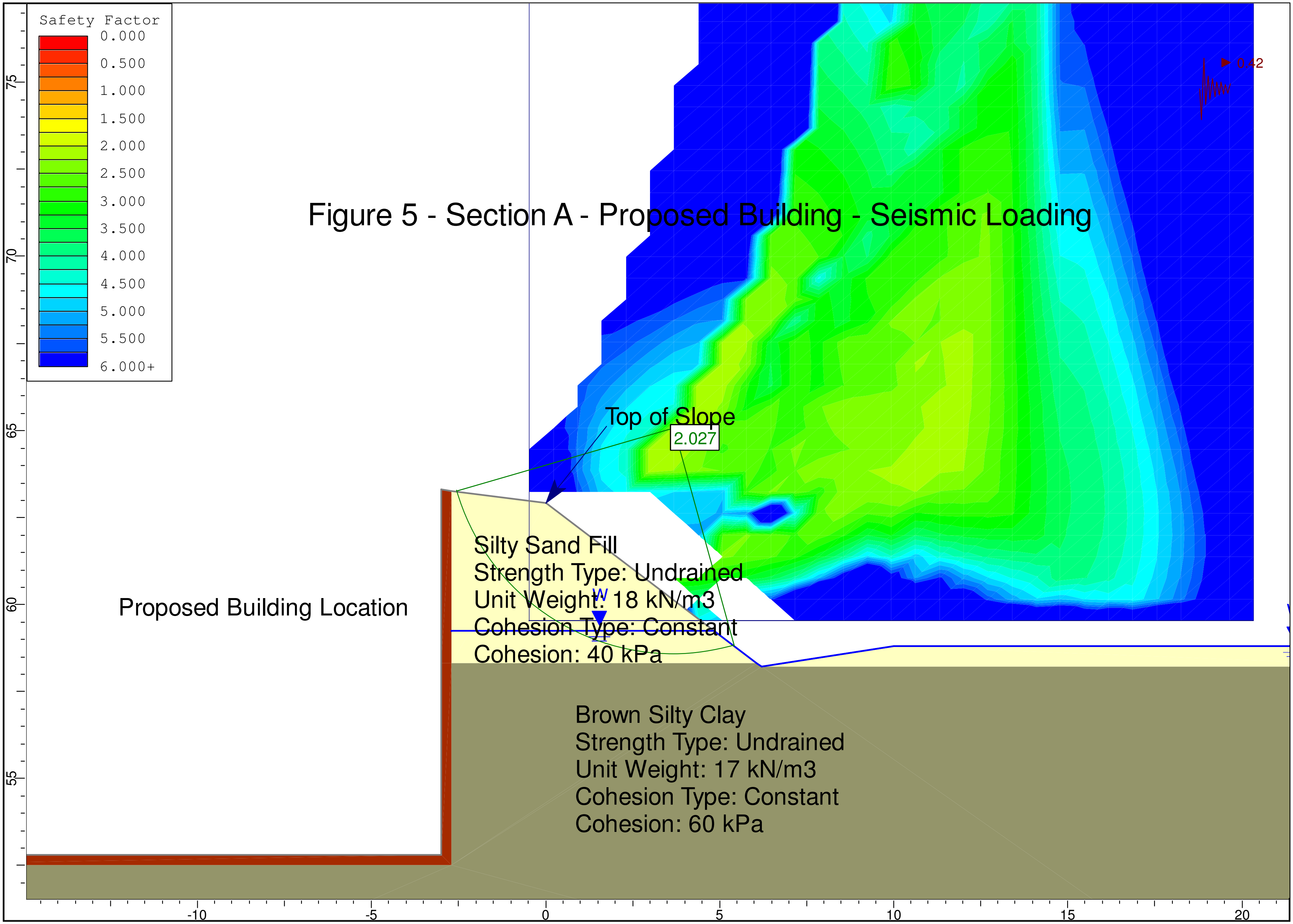
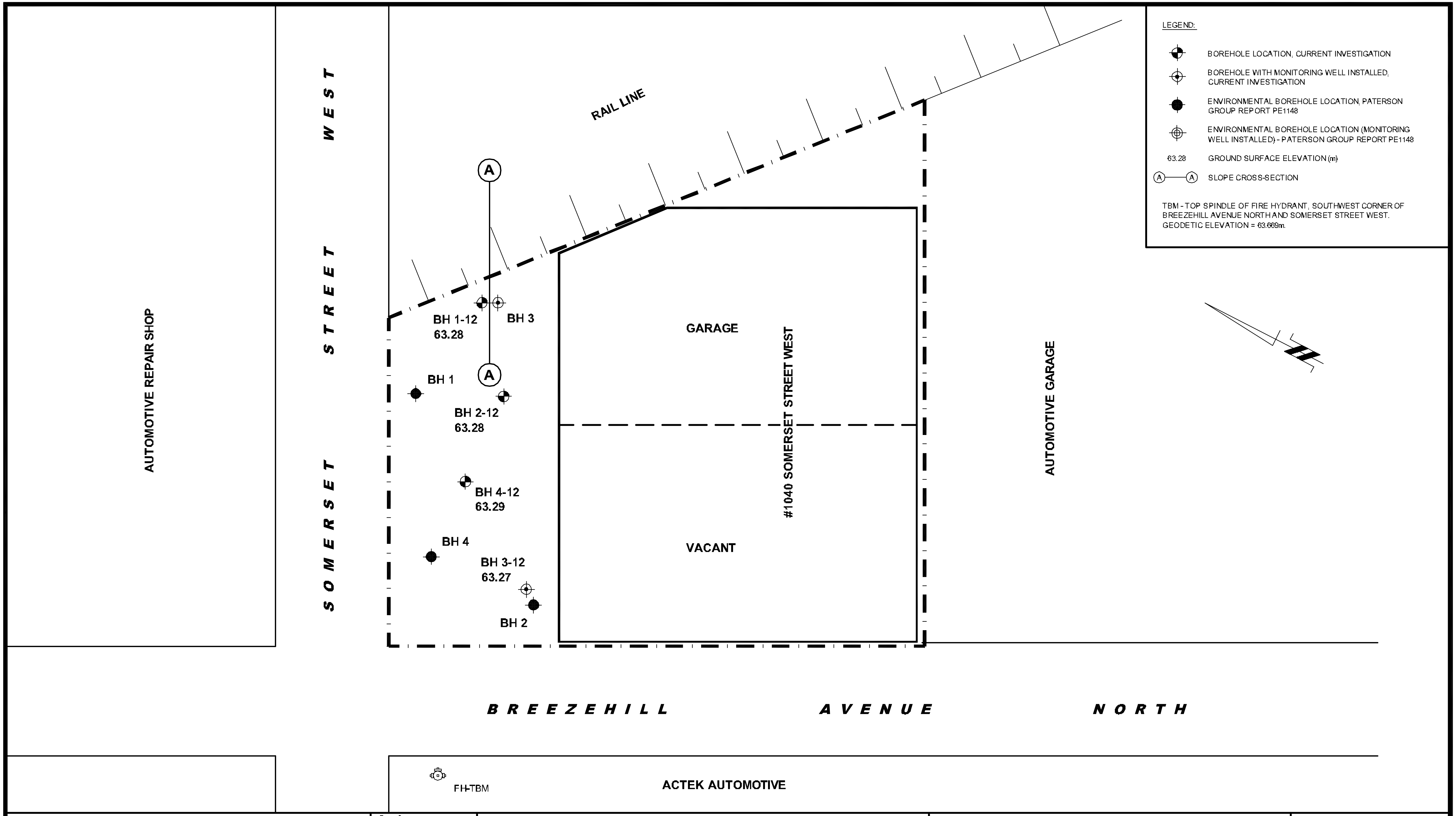


Figure 5 - Section A - Proposed Building - Seismic Loading



LEGEND:

- ⊕ BOREHOLE LOCATION, CURRENT INVESTIGATION
- ⊕ BOREHOLE WITH MONITORING WELL INSTALLED, CURRENT INVESTIGATION
- ● ENVIRONMENTAL BOREHOLE LOCATION, PATERSON GROUP REPORT PE1148
- ⊕ ENVIRONMENTAL BOREHOLE LOCATION (MONITORING WELL INSTALLED) - PATERSON GROUP REPORT PE1148
- 63.28 GROUND SURFACE ELEVATION (m)
- A——A SLOPE CROSS-SECTION

TBM - TOP SPINDLE OF FIRE HYDRANT, SOUTHWEST CORNER OF BREEZEHILL AVENUE NORTH AND SOMERSET STREET WEST. GEODETIC ELEVATION = 63.669m.

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

Scale: 1:250
Des.:
Dwn: MPG
Chkd: CDS

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
1040 SOMERSET STREET WEST
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No. PG2674-1
Report No.: PG2674-1
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