Landslide Hazard Assessment

Proposed Multi-Storey Building

3030 Saint Joseph Boulevard
Ottawa, Ontario

Prepared for Vuze Construction

Report PG6609 - 2 dated February 12, 2024

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1.0 Introduction

1.1 Purpose of Study and Scope of Work

Paterson Group (Paterson) was commissioned by Vuze Construction to conduct a landslide hazard assessment study for the proposed multi-storey building to be located at 3030 Saint Joseph Boulevard, Ottawa, Ontario (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report). The study has been prepared in response to the requirement by the Rideau Valley Conservation Authority (RVCA) as part of the Site Plan Approval process for the City of Ottawa for the subject site.

The objectives of the hazard assessment were to:

❏ Demonstrate that any landslide on the sloped areas, including a large "catastrophic landslide", has an annual probability less than 1:10,000.

❏ If the landslide hazard cannot be demonstrated to have an annual probability of less than 1:10,000, it must be demonstrated that the individual risk is <$1x10^{-5}$ per year and group risk falls within the "Acceptable" zone on a suitable group risk chart.

❏ If none of these criteria can be satisfied without mitigation measures, then the mitigation actions required must be demonstrated to reduce the risk below 10^{-5} per year and to "as low as reasonably practicable" (ALARP). If mitigation is required, further discussion with the RVCA will be required to determine what will be acceptable.

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.

1.2 Hazard Assessment Methodology

The methodology of this study was undertaken using a combination of the criteria and requirements set out by the following guidelines:

❏ Fraser Valley Regional District's Hazard Acceptability Thresholds for Development Applications dated October 2020

❏ The Association of Professional Engineers and Geoscientists of British Columbia's (APEGBC) Guidelines for Legislates Landslide Assessments for Proposed Residential Developments in BC, dated May 2010

- ❑ Geological Survey of Canada’s Open File 7312 - Landslide Risk Evaluation Technical Guidelines and Best Practices, dated 2013

The scope of work used in this study included a review of published literature describing local landslides and their associated triggers, geotechnical hazards, inventoried regional landslides and the geological setting of the study area. Desktop review of published topographic mapping, LiDAR imaging, and other geological mapping was also used as part of this assessment.

Field reconnaissance was carried out over several geotechnical field programs that have taken place throughout the subject site, including field review and subsurface investigations. Review of publicly available well records located in close proximity to the subject site was also considered as part of our assessment.

1.3 Proposed Development

Based on available information, the proposed development will consist of a multi-storey building with 3 to 4 underground parking levels. Associated asphaltic parking areas, access lanes and landscaped areas are also anticipated as part of the development. It is expected that the proposed buildings will be fully municipally serviced.

1.4 Review of Previous Geotechnical Investigation

For this assessment, subsurface information was collected from a set of site-specific investigations carried out by Paterson throughout the subject site. The results of the previous investigations are presented in Paterson Group Investigation Report PG6609-1 dated May 10, 2023.

2.0 Background of Study Area

2.1 Field Investigation

Geotechnical Investigations

Paterson has undertaken a series of geotechnical investigations at the subject site. The initial portion of the geotechnical investigation was carried out on April 26, 2017. At that time a total of three (3) boreholes were advanced to a maximum depth of 15.4 m below existing ground surface. Two supplemental investigations were carried out on April 19, 2018 and April 10, 2023 and consisted of advancing four (4) boreholes to a maximum depth of 15.0 m below the existing ground surface throughout the subject site.

The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site access, features and underground utilities. The borehole locations were determined and surveyed in the field by Paterson personnel. The locations of the boreholes are illustrated on Drawing PG6609-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The testing procedure consisted of augering and coring to the required depths and at the selected locations and sampling and testing the overburden soils and bedrock.

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. Bedrock samples were recovered from core recovery barrels. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags and the 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 flights, split spoon samples, 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.

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

Overburden thickness was evaluated by dynamic cone penetration testing (DCPT) at BH 1. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Rock samples were recovered from boreholes BH 1-23 and BH 2-23 using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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

Groundwater

Flexible polyethylene standpipes were installed in the boreholes advanced during the geotechnical investigations carried out on April 26, 2017 and April 10, 2023, to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Two groundwater monitoring wells were installed in the supplemental investigation carried out on April 19, 2018, to further monitor the groundwater levels below the subject site. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

Geotechnical Laboratory Testing

Moisture content testing was performed on 9 samples recovered from each borehole during the supplemental geotechnical investigation on April 10, 2023. The results of the moisture content testing are shown on the Soil Profile and Test Data Sheets in Appendix 1 of this report.

2.2 Existing Conditions

Surface Conditions

The subject site is located at the southwest corner of the intersection between St. Joseph Boulevard and Duford Drive and designated as 3030 St. Joseph Boulevard. The subject site is bordered to the west by a commercial property. The subject site is currently observed to be undeveloped and generally vacant.

The ground surface across the site slopes downward towards the north. It should be noted that the area following Duford Drive is understood to present features corresponding to a slope failure. Such failure is expected to have occurred due to a drastic change in slope resulting from excavations during the construction of Duford Drive. However, the current slope face was noted to include a terraced area along the base of the slope and to be grass covered with mature trees. Signs of slope instability were not noted during our field investigations.

Due to the presence of the slopes within and bordering the subject site, a slope stability assessment was carried out considering the slope conditions present in the subject site and described above. The results of the slope stability assessment indicated that the existing slopes are stable. The assessment is discussed further in Section 3.0 of this report.

Subsurface Conditions

Generally, the overburden profile consisted of a thin layer of topsoil and/or fill material underlain by a silty clay deposit, followed by a glacial till deposit. Where encountered, the fill material was generally observed to consist of brown silty clay with varying amounts of sand, and trace amounts of gravel and organics. The fill layer was observed to extend to depths ranging between 0.2 to 3.2 m below the existing grade.

The silty clay deposit generally consisted of a hard to very stiff weather silty clay crust followed by a grey silty clay deposit. The brown silty clay deposit was observed to extend to depths ranging between 5.5 to 7.6 m below the existing grade.

The glacial till deposit generally consisted of very dense grey silty clay with varying amounts of sand, gravel and cobbles.

Practical refusal to DCPT was encountered at BH 1 at an approximate depth of 15.3 m below existing grade.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each test hole location and Drawing PG6609-1 - Test Hole Location Plan in Appendix 2.

Bedrock

Limestone bedrock was cored in BH 1-23 and BH 2-23 to a depth of 12.1 and 15.0 m below the existing ground surface, respectively. The recorded RQD value ranged from 92 to 100, while the recovery values were equal to 100% for all samples. Based on the recovered rock core samples, the bedrock was observed to be of excellent quality.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of limestone of the Bobcaygeon Formation with an overburden drift thickness of 5 to 10 m depth.

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

Groundwater

Groundwater levels were measured in the monitoring wells installed at BH 1-18 and BH 2-18 on June 13, 2018. The measured groundwater level (GWL) readings are presented in Table 1 below and in the Soil Profile and Test Data sheets in Appendix 1.

It should be noted that level readings from piezometers are understood to have been influenced by surface water trapped within the backfilled borehole column, leading to higher than typical groundwater level observations. Additionally, groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

Long-term groundwater level can be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Moisture content testing completed on the recovered soil samples from BH 1-18 and BH 2-18 were observed to be consistent with the recorded groundwater level readings from June 13, 2018, and as indicated in Table 1 on the following page.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Method	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-23*	Piezometer	72.85	11.00	61.85	April 17, 2023
BH 2-23*	Piezometer	74.65	Dry	-	April 17, 2023
BH 1-18	Monitoring Well	76.38	6.72	69.66	June 13, 2018
BH 2-18	Monitoring Well	71.66	3.99	67.67	June 13, 2018
BH 1	Piezometer	77.16	3.29	73.87	May 4, 2017
BH 2	Piezometer	72.75	1.68	71.07	May 4, 2017
BH 3	Piezometer	70.24	Blocked	-	May 4, 2017

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

3.0 Slope Stability Analysis

Slope Conditions

Paterson completed a field review of three slope sections along the subject site within areas that were identified as worst-case scenarios, based on available topographic information.

The cross-section locations and topographic mapping information are presented on Drawing PG6609-1 - Test Hole Location Plan in Appendix 2.

Section A was profiled across the site from Duford Drive to St. Joseph Boulevard. A difference in elevation of approximately 7 m is present across the slope section. The slope across the subject site is shaped to an approximately 5H:1V slope. The top of slope at Section B and Section C is located behind the rear yards of the Kennedy Lane West dwellings with a difference in elevation of approximately 18 m between the top and toe of slope. The slope surface across the subject slopes was noted to be grass covered with no signs of slope instability noted.

Section C was located within a former slope failure area. It is understood that a slope failure occurred along the east side of Duford Drive in the 1960s. Photographs of the slope failure were provided to Paterson for this response. Photographs 1 and 2 presented in Appendix 2 show a shallow slope failure across a limited section of overall slope face. Based on slope features noted in the photographs, such as lack of vegetation across the slope face and the soil surface in the area of Duford Drive, it appears that the slope failure occurred across a section of the slope, which had been recently re-shaped as part of the construction of Duford Drive.

The natural grade of the slope face was drastically changed during the construction of Duford Drive. It is expected that the slope failure can be directly contributed to the steepness of excavated slope face along with exposure to precipitation events before a vegetative layer could establish. It should be further noted that a vegetative layer across a slope face promotes surficial run-off during precipitation events and limits infiltration of rainwater into the slope soil. Infiltration of water from precipitation events into a slope reduces overall slope stability.

The current slope face was noted to include a terraced area along the base of the slope face in the area of the former slope failure (see Photos 3, 4 and 5 in Appendix 2). It is suspected that the terraced area was introduced after the initial slope failure to stabilize the reinstated slope. Currently, the slope face was noted to be grass covered with mature trees. No signs of slope instability were noted.

Slope Stability Analysis

The slope stability analysis was modeled in SLIDE, a computer program which permits a two-dimensional slope stability analysis calculating several methods including the Bishop's method, which is a widely accepted slope analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to forces favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsurface soil and groundwater conditions, a factor of safety greater than 1.0 is generally required for the failure risk to be considered acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise permanent structures. An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

Three (3) slope cross-sections were analyzed based on the existing conditions observed during our site visit, and review of the available topographic mapping. The slope stability analysis was completed at each slope cross-section under worst-case-scenario by assigning cohesive soils under fully saturated conditions. Subsoil conditions at the cross-sections were inferred based on the findings at borehole locations along the top of slope, field observations during site visits and general knowledge of the area's geology.

The cross-section locations are presented on Drawing PG6609-1 - Test Hole Location Plan in Appendix 2.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 2.

Table 2 – Effective Stress Soil Parameters (Static Analysis)			
Soil Layer	**Unit Weight (kN/m^3)**	**Friction Angle (degrees)**	**Cohesion (kPa)**
Fill material	21	38	0
Brown Silty Clay (Crust)	17	33	8
Grey Silty Clay	16	33	10
Bedrock	22	Infinite Strength	

The total strength parameters for seismic analysis were chosen based on the subsurface conditions observed in the test holes, and our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 3.

Table 3 – Total Stress Soil Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill material	22	40	0
Brown Silty Clay (Crust)	17	-	120
Grey Silty Clay	16	-	80
Bedrock	22	Infinite Strength	

Static Analysis

The results for the existing slope conditions at Section A and Section B are shown in Figure 2 and Figure 4 in Appendix 2. The factor of safety was found to be greater than 1.5 for Section A and B when analyzed under static conditions. It should be noted that a slope stability analysis was completed for Section A due to the steepness of the slope observed, which was considered to be a worst-case scenario for the subject site. Section B was analyzed to include the adjacent slope opposite of Duford Drive. Section C was analyzed considering the upper 2 m of the slope face to be fully saturated and the remainder of the slope is saturated below the long-term groundwater table at the former slope failure location. A global slope stability factor of safety of greater than 1.5 was determined for Section C based on our analysis.

As a result, the three slope cross-sections analyzed were above the recommended Factor of Safety of 1.5, and are considered stable under static conditions.

It should be noted that the above noted saturated condition for the subject slope is considered to be a worst-case scenario due to the low permeability of the stiff silty clay deposit based on our knowledge of the subsoil conditions and the spring groundwater level readings at the monitoring well locations within the subject site. Based on the monitoring program measurements, the groundwater level was found to be at an elevation of 69.7 m at the top of slope (6.7 m depth) within the subject site and an elevation of 67.7 m at the bottom of slope (4 m depth).

Seismic Loading Analysis

An analysis considering seismic loading was also completed. A horizontal seismic acceleration, Kh, of 0.16G was considered for the analyzed sections.

A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the analyses including seismic loading are shown in Figure 3, Figure 5, and Figure 7B for the slope sections and attached in Appendix 2 of this report.

The overall slope stability factor of safety at the three slope cross-sections when considering seismic loading was found to be greater than 1.1, which is considered stable under seismic loading. Therefore, setbacks due to slope stability are not required from a geotechnical perspective at the subject site.

Seismic Design Considerations

Based on the results of the geotechnical investigation, a seismic **Site Class C** is considered applicable for foundation design within the area of the subject site as per Table 4.1.8.4.A of the OBC 2012.

The soils underlying the proposed foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

4.0 Landslide Hazard and Risk Assessment

4.1 General Methodology of Assessment

The methodology for the landside hazard assessment undertaken for this report may be considered as the following:

- ❑ Identify factors that are documented to contribute to the susceptibility for a landslide to occur throughout sloped terrain.
- ❑ Relate the aforementioned factors to the susceptibility for a landslide to occur throughout the subject site.
- ❑ Estimate the probability of a landslide to occur throughout the subject site based on historical regional landslide inventories. A baseline regional probability will be adjusted to a site-specific probability considering the site-specific factors that may promote landslide susceptibility using a Frequency Estimation Method.

If the hazard under consideration cannot be demonstrated to have an annual probability of less than 1:10,000, a group risk assessment estimating the annual probability of loss of lives would be carried out in accordance with the following equation:

$$Risk = P(H) \times P(S{:}H) \times P(T{:}S) \times V \times E$$

Where R represents the risk or annual probability of loss of life of an individual, P(H) stands for the annual probability that a landslide occurs, P(S:H) indicates the probability of impacting the elements taking into consideration the scale and location of the landslide events, P(T:S) is the temporal spatial probability of the elements being present at the time of a landslide (i.e.- the probability that a person is present at the location at risk), V represents the vulnerability, or likelihood of death or permanent injury of the individual given they are impacted and E represents the number of elements that would be impacted. The variable E can also be considered equal to the number of occupants for grouped areas.

4.2 Factors Affecting Landslide Susceptibility

The following sections discuss factors understood to affect the potential for a landslide to occur. The factors are described briefly and subsequently discussed on their impact to the susceptibility of a landslide throughout the subject site. The study area for the purpose of this discussion is considered as the area bound by the area considered by the Geological Survey of Canada under Open File 5311. The property discussed throughout this report is considered the subject site.

4.2.1 Clay Overburden

Based on the findings of the geotechnical investigation, the slope profiles throughout the subject site consist primarily of a silty clay deposit underlain by a relatively thin layer of glacial till and further by bedrock. Based on geological mapping undertaken by the Geological Survey of Canada under Open File 5311, the local deposit is considered as offshore marine sediments.

The clay deposit encountered throughout the subject site was observed to consist of a hard to very stiff, weathered, brown silty clay crust extending to depths between 5.5 to 7.6 m below the ground surface. The brown silty clay was underlain by a grey silty clay deposit underlain by a glacial till. Sand was not encountered above the clay deposit to form a “sand cap” layer at any borehole as has been documented throughout the Ottawa valley.

Review of landslides inventoried under Geological Survey of Canada (GSC) Open Files 5311, 7432 and 8600 document approximately 132 large landslide footprints throughout the Ottawa region. There is some overlap between these three inventories given the background for each document. Open File (OF) 5311 identifies these footprints as “Landslide Area – Reworked Marine Sediments”. OF8600 identifies these landslide footprints with greater precision, as it is understood to have been carried using digital elevation models (DEM) and LiDAR imaging for the boundary occupied by the City of Ottawa. OF7432 is a compilation of radiocarbon dates for approximately 45 landslides throughout the Ottawa Valley.

Review of the surficial geology for land adjacent to the landslides inventoried by the above-noted sources indicated approximately 83% (i.e., 109 out of 114 landslides captured by the study area published in OF5311) of these landslides may have originated from marine deposits consisting of clay. The remaining five landslides were considered to have consisted of alluvial sediments and/or organic deposits.

It has been documented that approximately 10 very large (i.e., surface area greater than 1 km^2) landslides in the Ottawa Valley have occurred throughout subsurface profiles containing a surficial sand cap layer (Fransham and Gadd, 1977). This study provided a surficial geology map for the Ottawa Valley identifying areas of sand and gravel overlying clay and areas consisting solely of silt and clay.

The study concluded there is a higher incidence for very large landslides to occur throughout clay deposits with an overlying sand cap. Nearly a hundred additional landslides have been identified by GSC throughout the area of the mapping prepared by Fransham and Gadd. The majority of the more recently documented smaller-sized landslides have occurred throughout the “clay” unit.

The presence of a weathered clay crust had been considered favorable in resisting the potential for a landslide to occur. However, review of 37 landslides throughout the Ottawa Valley and downstream of the Ottawa River and throughout Champlain Sea marine clay deposits indicate that clay crust and sand-capped clay deposits behave similarly during large retrogressive landslides (Perret, 2019). Based on this, it is inconclusive if the presence of a clay crust may or may not improve the resistance for clay soils to be susceptible to a landslide.

Further, studies have related the retrogression of landslides to the undrained shear strength using Taylor's stability number (Ns) as indicated below:

$$N_s = yH/S_u$$

Where y = unit weight of clay (kN/m^3), H = height of slope (m), and S_u = peak undrained shear strength (kPa). Analysis of forty landslides determined that N_s should be greater than or equal to 6 for the potential of retrogression to occur (Mitchell & Markell, 1974). Shear strength at the subject site ranges between 89 to more than 249 kPa for areas where the bank height is observed to be at most 10 m. Based on this, the worse-case scenario N_s values are less than 6, which would not suggest the potential for retrogression.

Mitchell and Markell have also explored the sensitivity of clays as a factor in retrogression, where sensitivity is defined as the ratio of undisturbed to remolded shear strength. In their studies, Mitchell and Markell concluded that the sensitivity of retrogressive clays ranges between 10 to 1,000. Even though there might be sampling intervals where the sensitivity of the clay is slightly higher than 10, the sensitivity of the clay deposit throughout the subject site has been observed to be generally below 10 and considered on the lower end for Champlain Sea clay deposits. A summary of clay sensitivity calculations is included in Table 4 on the following page.

Therefore, the potential for a very large retrogressive landslide is not considered to be very likely throughout the subject site given the presence of the subsurface profile encountered during the geotechnical investigation.

Table 4 – Clay Sensitivity

Borehole	Depth (m)	Undisturbed Shear Strength (kPa)	Remoulded Shear Strength (kPa)	Clay Sensitivity
BH 1-23	5.9 m	>249	NA	NA
BH 2-23	8.7 m	159	60	2.6
BH 2-23	9.5 m	104	20	5.2
BH 2-23	11.0 m	114	40	2.8
BH 1	5.7 m	154	20	7.7
BH 1	6.4 m	159	17	9.3
BH 1	7.9 m	90	8	11.2
BH 1	8.7 m	96	10	9.6
BH 2	6.4 m	110	10	11
BH 2	7.9 m	116	12	9.6
BH 3	4.9 m	144	17	8.5
BH 3	5.7 m	85	10	8.5
BH 3	6.4 m	104	13	8

4.2.2 Bedrock Depth and Surface Relief

Overburden thickness and surface relief are understood to be significant factors contributing to the potential for a landslide. Landslide susceptibility mapping carried out throughout National Topographic System (NTS) area 31H generally correlated higher values of drift thickness and surface relief to a higher rate of landslide incidence in Champlain Sea clays (Quinn, 2014). The study considered a weight of evidence approach which assigns a positive or negative weight for the ranges in these parameters with respect to the frequency of landslide occurrence.

A similar review was carried out to understand the relationship between overburden thickness and topographic relief for landslides that have occurred throughout the study area (area comprised by OF5311). The results of our interpretation of the available information are summarized in Table 5 and Table 6 on the following pages.

<table>
<tr><th colspan="3">Table 5 – Summary of Drift Thickness Throughout Historic Landslide Footprints</th></tr>
<tr><th>Drift Thickness</th><th>Number of Incidences</th><th>%</th></tr>
<tr><td>0 to 1</td><td>0</td><td>0.0</td></tr>
<tr><td>1 to 2</td><td>0</td><td>0.0</td></tr>
<tr><td>2 to 3</td><td>0</td><td>0.0</td></tr>
<tr><td>3 to 5</td><td>0</td><td>0.0</td></tr>
<tr><td>5 to 10</td><td>8</td><td>7.0</td></tr>
<tr><td>10 to 15</td><td>7</td><td>6.1</td></tr>
<tr><td>15 to 25</td><td>34</td><td>29.8</td></tr>
<tr><td>25 to 50</td><td>49</td><td>43.0</td></tr>
<tr><td>50 to 100</td><td>16</td><td>14.0</td></tr>
<tr><td>Total Landslides Within Study Area</td><td>114</td><td rowspan="2">94.2</td></tr>
<tr><td>Total Landslides Documented by Open Files</td><td>121</td></tr>
<tr><td colspan="3">Note: Drift thickness interpreted using Google Earth and is considered subjective, however, appropriate based on the available information for each of the landslides identified by OF5311, OF7432 and OF8600 and the purpose of this assessment.</td></tr>
</table>

Topographic relief was interpreted using DEM provided by Google Earth. Relief was considered as the difference between the lowest and highest elevations, distances extending beyond a landslide footprint. Greater distances were considered where a landslide formed into a slope profile. Significantly large landslides could not be evaluated reasonably due to the highly variable topography beyond their footprint. The measure is considered subjective, however, appropriate based on the available topographic information for each of the landslides identified by OF5311, OF7432 and OF8600 and the purpose of this assessment.

In summary, incidences of landslides occur more frequently in areas with intermediate overburden thickness ranging between 15 to 40 m, and greater than 10 m of topographic relief throughout the study area. Based on the current test hole coverage and slope stability sections, it is anticipated that less than 10 m of overburden may be present throughout the subject site. Further, up to 18 m of relief may be observed at the southwest portion of the subject site along slope stability cross sections B and C. The remainder of the subject site presents a topographic relief below 10 m.

Table 6 – Summary of Topographic Relief Throughout Historic Landslide Footprints		
Topographic Relief	**Number of Incidences**	**%**
<1	0	0.0
1-2	0	0.0
2-3	1	0.9
3-4	2	1.8
4-5	0	0.0
5-6	2	1.8
6-7	0	0.0
7-8	2	1.8
8-9	3	2.7
9-10	3	2.7
10-12	8	7.1
12-14	11	9.7
14-16	16	14.2
16-18	8	7.1
18-20	5	4.4
20-25	21	18.6
25-30	12	10.6
30-40	13	11.5
>40	6	5.3
Total Landslides Within Study Area Capable of Being Measured	113	93.4
Total Landslides Documented by Open Files	121	

Based on the above, the potential for a landslide as based on the above-noted factors may increase gradually towards the southwest portion of the subject site. This is discussed in further detail in *Section 4.3 – Hazard Assessment* of this report.

4.2.3 Groundwater

Groundwater is understood to be a factor contributing to landslide susceptibility. Landslides throughout Ottawa Valley have been understood to generally occur most frequently during the spring thaw, which results in seasonal increases in the depth of the groundwater table and porewater pressure. It has been documented that larger slopes typically fail by a combination of a downward gradient throughout the table lands and an upward gradient (artesian) throughout the bottom of the slope profile and along the channel (Hugenholtz and Lacelle, 2004).

Groundwater regimes with primarily downward gradients from the table lands to the watercourse typically have stronger stability attributes in resisting the potential for a slope failure. Groundwater regimes may be influenced by other factors, such as rising bedrock surfaces (Quinn et al., 2010). The combination of a temporary (seasonal) artesian groundwater table gradient throughout the lower portion of the slope and rising bedrock surface may significantly impact the stability of a slope.

Our slope stability assessment in Section 3.1 of this report considered fully saturated slope conditions, which represent the worst-case scenario. Fully saturated slope conditions are anticipated to govern over the downward gradient conditions as a loading case from a slope stability perspective. The slope stability factors of safety were found to be greater than 1.5 for all slope sections analyzed. Further, the groundwater regime throughout the subject site is expected to follow general surficial topography such that sheet drainage of surface water across and away from the subject site is expected to occur. Further, the proposed post-development surface conditions will further improve this condition through the grading anticipated at the site, the use of catch basins to redirect water for drainage and the installation of asphalt surface roads. Based on the above, surface and meltwater is not expected to pond throughout the subject site.

Therefore, based on the above, the likelihood for a landslide or slope failure to occur due to snowmelt or rain events is considered unlikely throughout the subject site.

4.2.4 Toe Erosion

The existing ground surface located at the bottom of the slope noted along the northern property boundary currently consists of dry land with some vegetation and is not subjected to erosion or rapid drawdown loading from water. Throughout these areas, the adjacent lots are provided drainage systems to divert surface run-off to municipal drains further down-stream.

As there are no watercourses that may erode or affect the stability of the existing slope throughout the subject site, the toe erosion trigger mechanism is considered to be of insignificant and negligible risk and likelihood under current and post-development conditions. Based on this, this hazard trigger will not be explored further as a risk factor that may negatively impact the proposed development.

4.2.5 Proximity to Landslides

The proximity of land to previous landslides has been documented as a significant factor in assessing the susceptibility of potential for future landslides. It had been assessed that there is between 49.2 and 96.7% likelihood of a landslide in areas located less than 50 to 2,000 m from a previous landslide event (Quinn et al., 2011).

It is further documented that areas that have previously been affected by landslide events are more vulnerable to experiencing new landslides. This was observed by Hugenholtz (2004) in their review of Green's Creek and the concentration of landslides to re-occur in concentrated areas along the creek alignment.

Landslide inventory mapping published by GSC indicates the presence of potentially up to 4 landslides within a proximity of 2 km to the subject site. However, none of these landslides intersect the subject site.

Landslides Oln10, Oln11, Oln12 and Oln13 are located approximately between 0.7 and 2.2 km to the southwest of the subject site. Landslide Oln13, characterized as probable, is the one located closer to the subject site and has been depicted in Figure 1, included below for reference. The totality of the group has been reported by GSC to have retrogressed into the scarp slope above a terrace surface of the proto-Ottawa River (GSC OF8600, 2019).

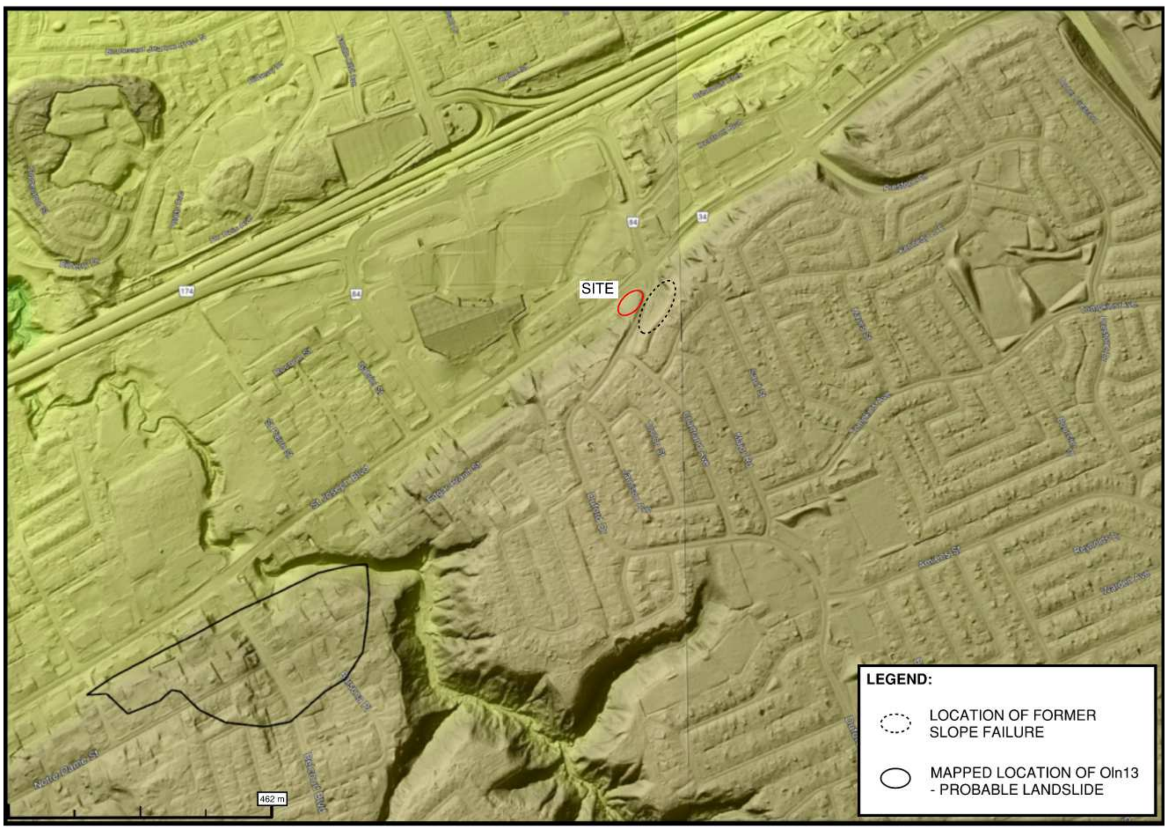


Figure 1 – LiDAR Image of Subject Site and closest landslide and slope failure.

The totality of the group is heavily altered by urban development which results in challenges to complete a thorough characterization of the landslide scar. However, it can be estimated that the aforementioned landslides experienced a topographic relief of approximately 20 m.

Furthermore, drift thickness throughout the area of the aforementioned landslides has been documented by GSC OF5311 as ranging between 25 to 100 m.

Given that the above-mentioned landslides retrogressed into the scarp slope along the margin of the proto-Ottawa River, it is speculated that these landslides may have been triggered by extensive toe erosion by the proto-Ottawa River. Given the current established Ottawa River watercourse alignment and the shoreline protection against toe erosion implemented along the Ottawa River, this trigger factor is not considered to currently affect landslide susceptibility within the subject site. Therefore, it is not considered that the presence of Oln10, Oln11, Oln12 and Oln13 indicate a higher likelihood of a landslide to occur throughout the subject site.

Furthermore, and as discussed in the previous sections, it is understood that a slope failure occurred along the east side of Duford Drive in the 1960s. It is expected that the abovementioned failure occurred as a consequence to a drastic change in slope resulting from excavations during the construction of Duford Drive. It should further be noted that the current slope face is terraced and grass covered, which increases sheet drainage and improves overall slope stability.

It should be further noted that, based on the review of the subject area, no signs of slope instability were noted, including ground movement along the face of slope, cracks in landscaped and hardscaped areas, or damage of the existing roadways and properties located throughout and in the neighbouring area of the landslide scar. Furthermore, based on analysis of aerial photos, Paterson considers it unlikely that the slope has undergone intolerable ground movements since the occurrence of the failure. Based on the above analysis and the results of the slope stability study completed on the area, the current slope is considered stable from a geotechnical perspective.

Therefore, it is not considered that the presence of the slope failure indicates a higher likelihood of a landslide to occur throughout the subject site. However, it is considered appropriate and conservative to increase the baseline probability for landslides to occur throughout the subject site by one order of magnitude to account for the presence of the aforementioned local feature.

4.2.6 Earthquakes

Earthquakes are understood to be a major contributing factor in triggering some of the largest landslides inventoried throughout Champlain Sea clay deposits. Many large landslides have been estimated to have occurred approximately 4,550 years before present (BP) and another significant cluster approximately 7,060 years BP (GSC OF7432, 2021; Aylsworth and Lawrence, 2003).

The lower bound of these paleo-earthquakes have been estimated to have consisted of M5.9 to M6.0 earthquakes. Several landslides were triggered by the 1663 M7 Charlevoix and 2010 Val-des-Bois M6.2 earthquakes.

The behavior of clay slopes during earthquakes is uncertain and is a topic of current research. Current research suggests that large earthquakes can propagate failures along pre-existing or partially developed planes of weakness along the slope footprint. The critical length of the propagation is understood to be influenced by the sensitivity and fracture toughness, or brittleness, of the clay deposit (Quinn et al. 2012).

The slopes and clay deposit throughout the subject site have been subject to large historic earthquakes that may have triggered significantly large historic landslides throughout the Ottawa Valley. Earthquake-induced landslides generally occur where the potential for slope failures already exists and has generally been assessed as part of our slope stability analysis.

Pseudo-static (seismic) loading of the slope profiles considered a PGA of 0.16g and resulted in factors of safety exceeding 1.1 as discussed in Section 3.0 of this report. This PGA is considered equivalent to a 1:1,000-year earthquake event. This value is considered suitable for assessing the stability of the subject slopes when subject to loading that may be associated with earthquakes experienced locally.

Further, larger landslides are understood to be associated with clay deposits with remolded shear strength measurements equal to or less than 1 kPa (Quinn et al., 2011). It would be expected that clay deposits with such low values of remolded strength to be conducive to propagating planes of weakness and unable to resist high earthquake loads. Review of our test hole coverage indicated that remolded shear strength values typically range between 8 and 20 kPa and exceed the 1 kPa threshold associated with landslides. Based on this, it is not expected a significant shear band would propagate throughout the slopes located throughout the subject site that would increase landslide susceptibility due to earthquake loading.

This conclusion may be extrapolated further to the potential for sources of subsurface vibrations such as those associated with building construction, compaction equipment, general earthworks equipment, and installation of temporary shoring. These sources of vibrations are not anticipated to exceed or be close to the magnitude of vibrations associated with the assessed earthquake load of 0.16g.

Given the above, earthquake loading is not anticipated to impact landslide susceptibility.

4.3 Hazard Assessment

Frequency Estimation Method

Approximately 132 individual landslides have been identified between GSC files OF8600, OF7432 and OF5311. The study area between these files considers an approximate surface area of approximately 11,800 km^2. This surface area may be decreased to approximately 6,845 km^2 when neglecting the area comprised of bedrock. The study area was reduced accordingly to consider the absence of Champlain Sea marine deposits throughout areas of bedrock outcrops and where overburden is not present. An average landslide density of 1.9×10^{-2} per km^2 may be extrapolated from this information.

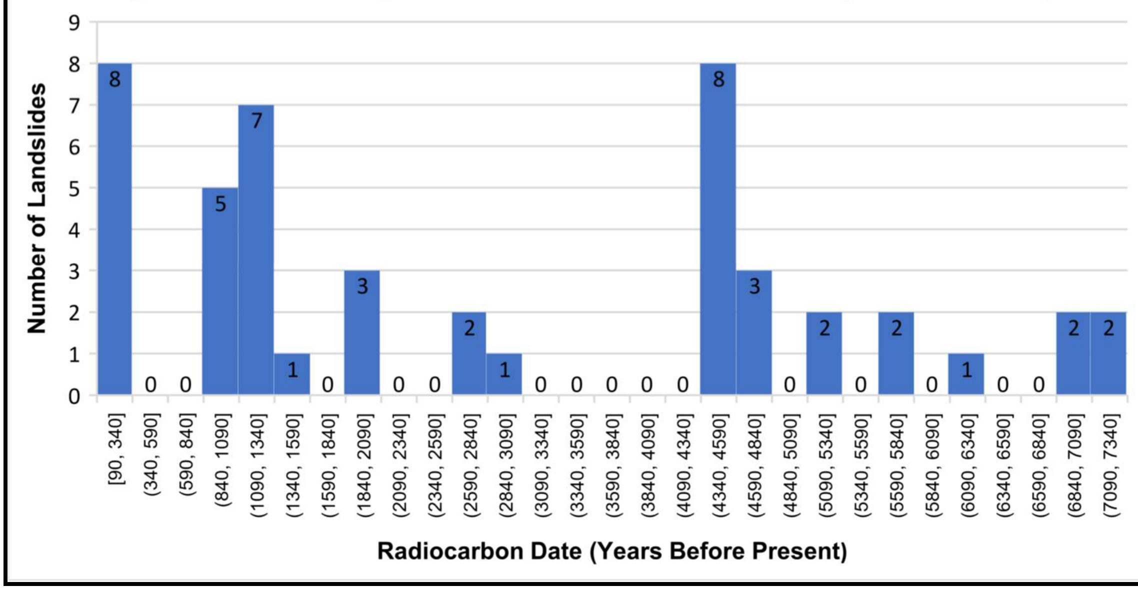
Based on the information provided in OF5311, landslides have not been recorded to have originated from areas comprised of till or glaciofluvial deposits. The study area may be therefore reduced further to approximately 5,354 km^2 and consisting of nearshore and offshore marine deposits, alluvial sediments, organic deposits, and sand dunes. The surficial deposits are considered susceptible to a landslide given their vulnerability to failure by the factors discussed in the preceding sections of this report. Based on this, the baseline landslide frequency, and probability, may be considered as 2.5×10^{-2} per km^2 throughout the study area.

The estimated density may vary notably across the study area given that many landslides generally occurred in localized clusters. The distinct clusters of landslides are likely indicative of conditions that are more conducive to landslide hazards in localized zones rather than the entire study area. However, this is considered appropriate as an average density for the purpose of this assessment.

The temporal frequency of landslide occurrence may vary substantially across the study area. OF7432 sought to carbon date 45 separate landslide features throughout the study area. The landslides interpreted by that study documented landslides having occurred potentially between approximately 90 to 7,140 years before present.

The results from the study and approximations provided by OF8600, neglecting the potential deviation and range of uncertainty, are summarized in Figure 2 below.

Figure 2 - Summary of Carbon-dated Landslides (GSC OF7432)



Temporal factors such as periods of increased earthquakes and climatic factors affecting these frequencies have been explored by others. Based on the above, more than half of the carbon dated landslides have occurred within the past 3,090 years, and over a quarter within the past 1,090 years.

Quinn et al. (2011) proposed a conservative lower bound of 500 years as a return period for the study area of NTS 31H. This value could be considered appropriate throughout the subject site based on the information presented above. However, the study area of NTS 31H considers a much higher density of landslides (i.e., 1,248 landslides over 75-80,000km^2) than the study area considered for the subject site.

Based on this, a return period equivalent to the average frequency of landslides (i.e., 132 landslides over 7,140 years) provides a smaller lower bound return period of approximately one large landslide every 54.1 years. An upper bound return period of 1,000 years was indicated in Subsection 4.2 of this report. Then, a 54.1-year return period is within the previously defined range. With a return period of 54.1 years, a baseline landslide probability of 4.6x10^{-4} landslides per km^2 per annum is calculated over the study area defined by the GSC files.

The current baseline probability (4.6x10^{-4} per km^2 per annum) assumes uniform susceptibility across the study area. The baseline estimate may be adjusted based on our judgement of a combination of regional landslide inventories, local site attributes and our experience assessing the performance of slopes comprised of Champlain Sea marine deposits throughout the study area.

Based on our review, it had been assessed that the proximity of historic landslides to the subject site was of sufficient significance to increase the estimate by one order of magnitude. Therefore, the baseline probability may be considered as $4.6x10^{-3}$ per km^2 per annum.

The probabilities for landslides to occur throughout the subject site considering drift thickness (Table 7) and surface relief (Table 8) are estimated accordingly in Table 9.

Table 7 – Summary of Drift Thickness Throughout Historic Landslide Footprints

Drift Thickness (m)	Number of Incidences	%	Probability (54.1-year return period, cumulative)
0 to 1	0	0.0	0.0
1 to 2	0	0.0	0.0
2 to 3	0	0.0	0.0
3 to 5	0	0.0	0.0
5 to 10	8	7.0	0.0013
10 to 15	7	6.1	0.0024
15 to 25	34	29.8	0.0079
25 to 50	49	43.0	0.0159
50 to 100	16	14.0	0.0185

Table 8 – Summary of Surface Relief Throughout Historic Landslide Footprints

Surface Relief (m)	Number of Incidences	%	Probability (54.1-year return period, cumulative)
0 to 4	0	0.0	0.0005
4 to 6	0	0.0	0.0008
6 to 8	1	0.9	0.0011
8 to 10	2	1.8	0.0021
10 to 12	0	0.0	0.0034
12 to 14	2	1.8	0.0052
14 to 16	0	0.0	0.0079
16 to 18	2	1.8	0.0092
18 to 20	3	2.7	0.0100
20 to 25	3	2.7	0.0135
25 to 30	8	7.1	0.0154
30 to 40	11	9.7	0.0175
>40	16	14.2	0.0185

Table 9 – Estimate of Landslide Probability				
Surface Relief (m)	**Drift Thickness (m)**			
	0 to 10	**0 to 15**	**0 to 25**	**0 to 50**
0 to 4	2.9E-09	5.4E-09	1.8E-08	3.6E-08
0 to 6	4.8E-09	9.1E-09	3.0E-08	5.9E-08
0 to 8	**6.8E-09**	1.3E-08	4.1E-08	8.3E-08
0 to 10	**1.3E-08**	2.4E-08	7.7E-08	1.5E-07
0 to 12	**2.0E-08**	3.8E-08	1.2E-07	2.5E-07
0 to 14	**3.1E-08**	5.8E-08	1.9E-07	3.8E-07
0 to 16	**4.6E-08**	8.7E-08	2.8E-07	5.7E-07
0 to 18	**5.4E-08**	1.0E-07	3.3E-07	6.6E-07
0 to 20	5.9E-08	1.1E-07	3.6E-07	7.2E-07
0 to 25	7.9E-08	1.5E-07	4.9E-07	9.7E-07
Note: Bolded text is considered reflective of site-specific conditions. The above-noted values are considered in units of landslide per annum.				

Based on our assessment, the probability for a landslide to occur throughout the subject site has been estimated to range between **1:18,461,014 and 1:344,605,588 per annum** for a 1:54.1-year return period (product of baseline probability, probability of landslide occurrence based on cumulative drift thickness between 0 and 50 m and probability based on cumulative surface relief between 0 and 12 m).

Based on the above, the annual probability of a large landslide occurring at or directly impacting the subject site is estimated to be less than 1:10,000 per annum.

5.0 Conclusion

In summary, a multi-storey building is currently being proposed to occupy the subject site. It is understood that a slope failure occurred in close proximity to the subject site. Based on our review, this failure may have occurred as a consequence of a drastic change of slope resulting from excavation works completed during the construction of Duford Drive. However, the current conditions differ from those encountered at the time of the slope failure. At the moment, the slope is terraced, and grass covered with mature trees which stabilizes the reinstated slope. Further, the vegetative cover increases the slope stability by promoting surficial run-off of rainwater during precipitation events. Furthermore, field investigations and reconnaissance completed by Paterson at the subject site did not indicate signs of movement, activity, or cause of concern with respect to landslide susceptibility. Therefore, the existing slope failure is not considered an indicative feature for a higher likelihood of a landslide to occur throughout the subject site.

The area was also reviewed by means of available published satellite images, LiDAR information, landslide inventory in the neighbouring area, research and studies carried out by others specializing in the field of earthquakes, landslides, and geology. Using a combination of the above and our experience with sites of similar geological settings throughout the Ottawa region, the annual probability of a large catastrophic landslide occurring at or directly impacting the subject site is estimated to be less than 1:10,000. Based on our interpretation of the information available to carry out this assessment, the subject site is considered safe and suitable for consideration of the proposed development.

6.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project and the applicable guidelines.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock, and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The assessments provided in this report are intended for the use of design professionals associated with this project. 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 Vuze Construction or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Feb. 12, 2024
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, P. Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❑ Vuze Construction
- ❑ Paterson Group Inc

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

EARTHQUAKES CANADA SEISMIC HAZARD (NBCC 2015)

TABLE 1 - SUMMARY OF REVIEWED LANDSLIDE INVENTORY DATA

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
3020 St-Joseph Boulevard
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

FILE NO. **PG6609**

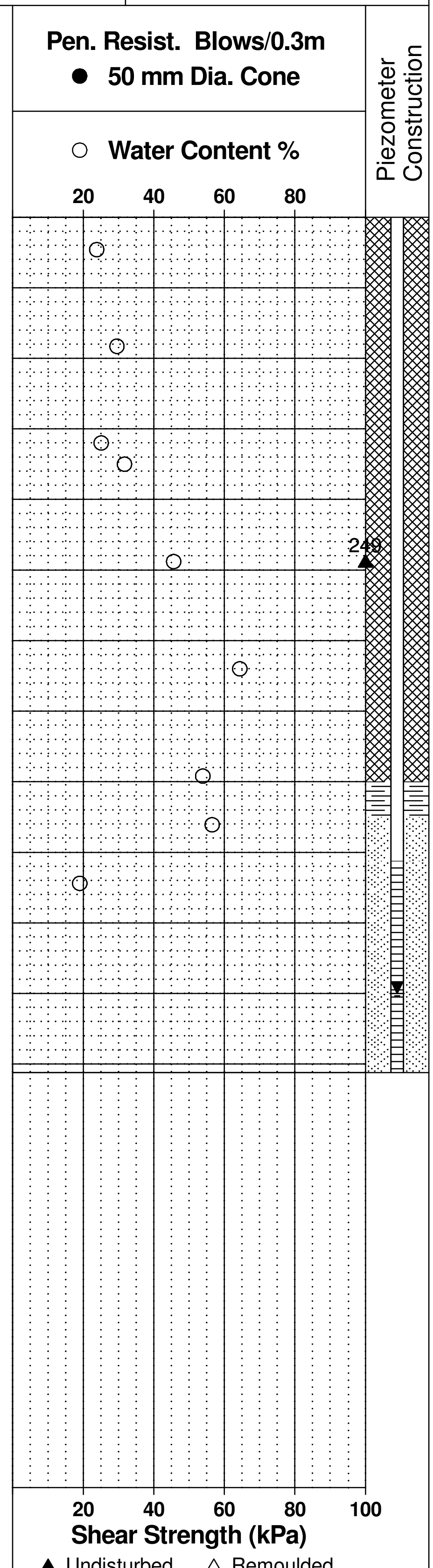
REMARKS

HOLE NO. **BH 1-23**

BORINGS BY CME-55 Low Clearance Drill **DATE** April 10, 2023

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
TOPSOIL	0.08	AU	1		
FILL: Reddish brown silty clay, trace gravel and organics	3.25	SS	2	63	15
Hard to very stiff, brown **SILTY CLAY** - grey by 5.6m depth	8.08	SS	3	88	16
		SS	4	100	P
		SS	5	100	6
		SS	6	100	6
GLACIAL TILL: Very dense, grey silty clay with sand, gravel and cobbles	9.86	G	7		
		SS	8	21	50+
BEDROCK: Excellent quality, grey limestone	12.12	RC	1	100	93
		RC	2	100	100
End of Borehole					
(GWL @ 11.00m - April 17, 2023)					

DEPTH (m)	ELEV. (m)
0	72.85
1	71.85
2	70.85
3	69.85
4	68.85
5	67.85
6	66.85
7	65.85
8	64.85
9	63.85
10	62.85
11	61.85
12	60.85



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
3020 St-Joseph Boulevard
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

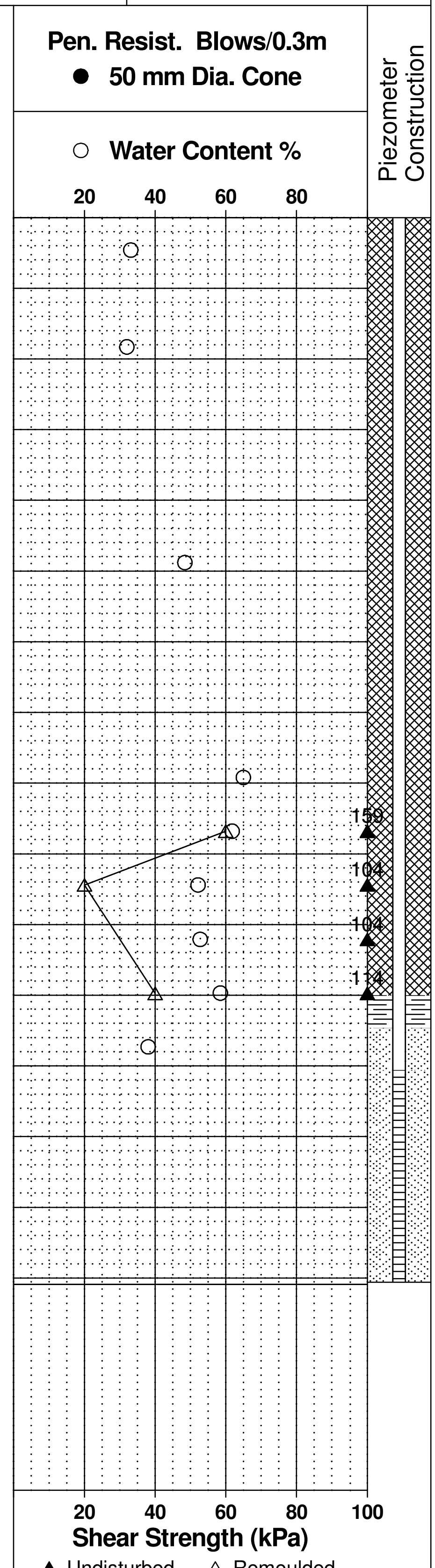
BORINGS BY CME-55 Low Clearance Drill **DATE** April 10, 2023

FILE NO. PG6609

HOLE NO. BH 2-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
TOPSOIL	0.13	AU	1		
FILL: Reddish brown silty clay, trace gravel	3.20	SS	2	63	7
Hard to very stiff, brown **SILTY CLAY** - grey by 7.3m depth	11.35	SS	3	100	8
		SS	4	100	2
		SS	5	100	P
		SS	6	100	P
		SS	7	100	P
		SS	8	100	P
GLACIAL TILL: Very dense, grey silty clay with sand, gravel and cobbles	12.06	SS	9	100	P
BEDROCK: Excellent quality, grey limestone	15.08	RC	1	100	92
		RC		100	100
End of Borehole					
(BH dry - April 17, 2023)					

DEPTH (m)	ELEV. (m)
0	74.65
1	73.65
2	72.65
3	71.65
4	70.65
5	69.65
6	68.65
7	67.65
8	66.65
9	65.65
10	64.65
11	63.65
12	62.65
13	61.65
14	60.65
15	59.65



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 3018 St. Joseph Boulevard. Geodetic elevation = 69.77m.

REMARKS

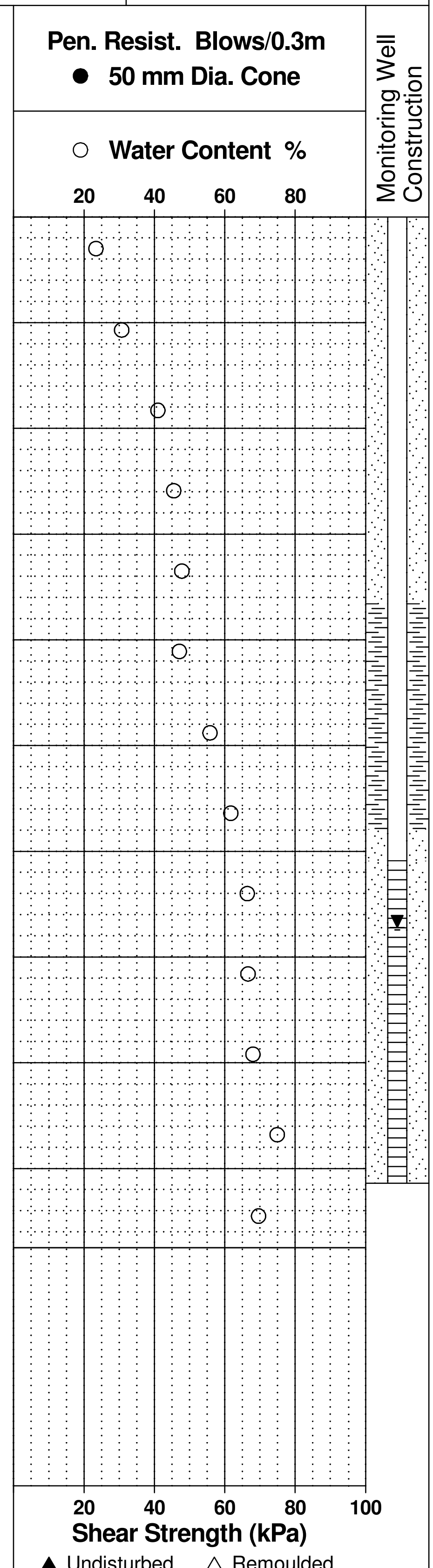
BORINGS BY CME 55 Power Auger

DATE April 19, 2018

FILE NO. PG4083

HOLE NO. BH 1-18

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	76.38
TOPSOIL 0.15	AU	1				
Brown **SILTY CLAY,** trace sand 0.76						
	SS	2	71	8	1	75.38
	SS	3	100	9	2	74.38
	SS	4	100	7	3	73.38
	SS	5	100	7		
Stiff to firm, brown **SILTY CLAY**	SS	6	100	6	4	72.38
	SS	7	100	5	5	71.38
- firm to soft and grey by 5.5m depth	SS	8	100	1	6	70.38
	SS	9	100	2		
	SS	10	100	1	7	69.38
	SS	11	100	W	8	68.38
	SS	12	100	W	9	67.38
9.75	SS	13	100	W		
End of Borehole						
(GWL @ 6.72m - June 13, 2018)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 3018 St. Joseph Boulevard. Geodetic elevation = 69.77m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 19, 2018

FILE NO. PG4083

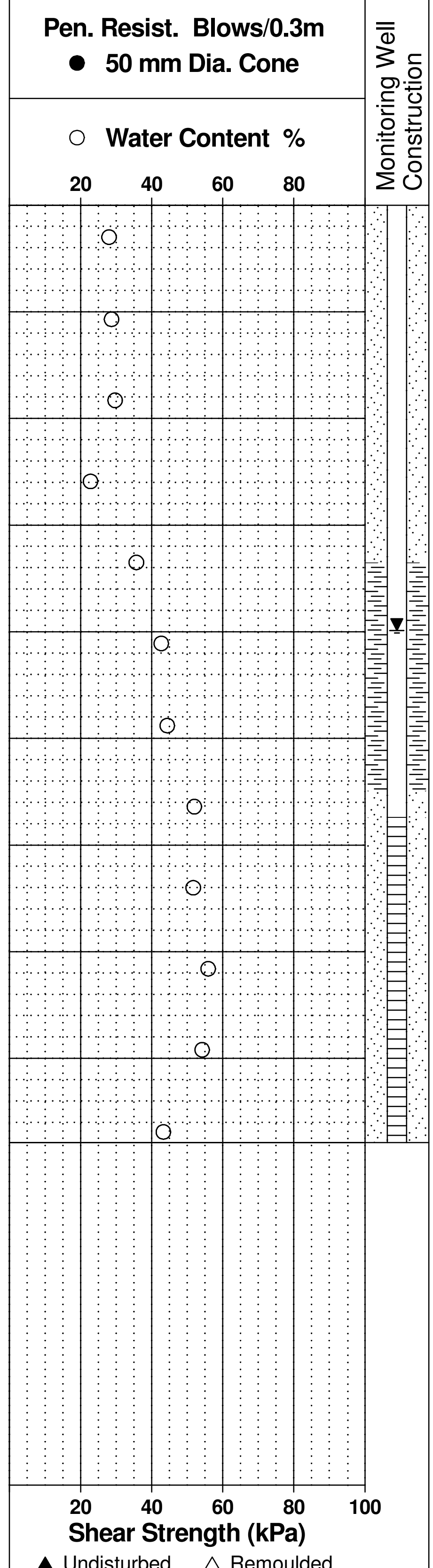
HOLE NO. BH 2-18

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.66
TOPSOIL 0.13	AU	1				
Very stiff, brown **SILTY CLAY** 1.52	SS	2	58	12	1	70.66
Very stiff, brown **SILTY CLAY,** some gravel, trace sand 2.30	SS	3	21	9	2	69.66
Very stiff to stiff, brown **SILTY CLAY**	SS	4	62	12		
	SS	5	100	17	3	68.66
	SS	6	100	13	4	67.66
	SS	7	100	9	5	66.66
	SS	8	100	5	6	65.66
- firm to soft and grey by 6.4m depth	SS	9	100	W		
	SS	10	100	W	7	64.66
	SS	11	100	2	8	63.66
8.79	SS	12	92	50+		

End of Borehole

Practical refusal to augering at 8.79m depth

(GWL @ 3.99m - June 13, 2018)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (as shown on Dwg. PG4083-1). Geodetic elevation = 76.32m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 26, 2017

FILE NO. PG4083

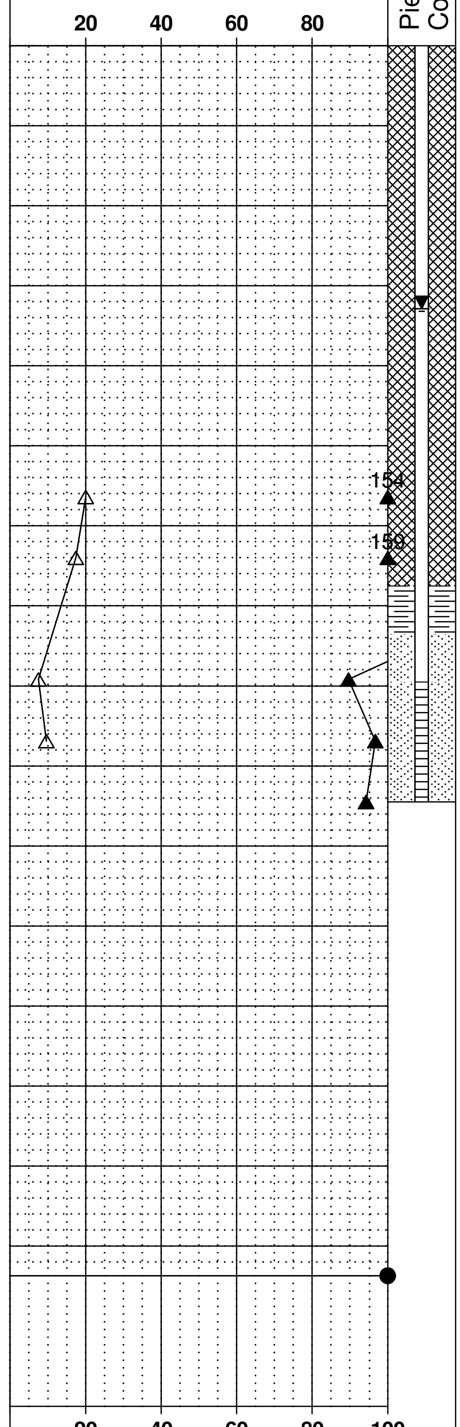
HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.18
FILL: Brown silty clay, some sand, trace gravel, cobbles, boulders and wood	2.13
Very stiff to stiff, brown **SILTY CLAY** - grey by 7.6m depth	9.45
Dynamic Cone Penetration Test (DCPT) commenced at 9.45m depth. Cone pushed to 15.37m depth. (GWL @ 3.29m - May 4, 2017)	15.37
End of Borehole Practical DCPT refusal at 15.37m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	8
SS	3	38	10
SS	4	54	16
SS	5	100	17
SS	6	100	11
SS	7	100	9

DEPTH (m)	ELEV. (m)
0	77.16
1	76.16
2	75.16
3	74.16
4	73.16
5	72.16
6	71.16
7	70.16
8	69.16
9	68.16
10	67.16
11	66.16
12	65.16
13	64.16
14	63.16
15	62.16

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



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (as shown on Dwg. PG4083-1). Geodetic elevation = 76.32m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 26 April 2017

FILE NO. PG4083

HOLE NO. BH 2

SOIL DESCRIPTION	Layer depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	72.75
TOPSOIL	0.15	AU	1				
FILL: Loose, brown silty clay, some sand, trace gravel and organics	1.52	SS	2	67	8	1	71.75
Very stiff to firm, brown **SILTY CLAY** - sand seams to 2.5 m depth - grey by 7.6m depth	9.14	SS	3	75	13	2	70.75
		SS	4	83	16	3	69.75
		SS	5	100	12	4	68.75
		SS	6	100	10	5	67.75
		SS	7	100	10	6	66.75
		SS	8	100	6	7	65.75
						8	64.75
						9	63.75
GLACIAL TILL: Very dense, grey silt with clay, sand, gravel, cobbles and shale fragments	9.73	SS	9	100	50+		

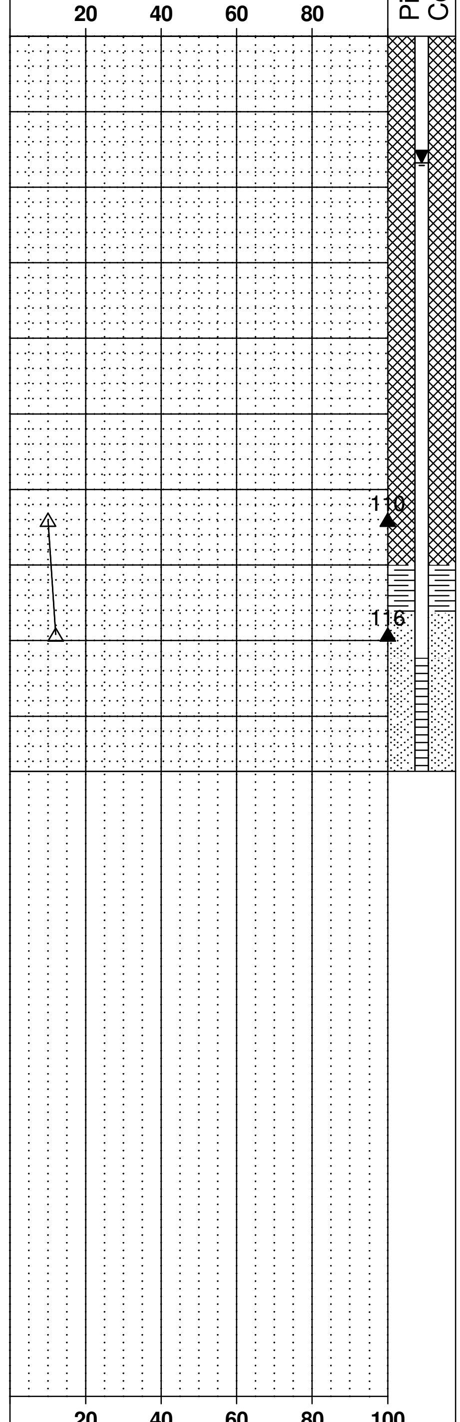
End of Borehole

Practical refusal to augering at 9.73m depth

(GWL @ 1.68m - May 4, 2017)

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 3018 St. Joseph Boulevard. Geodetic elevation = 69.77m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 26, 2017

FILE NO. PG4083

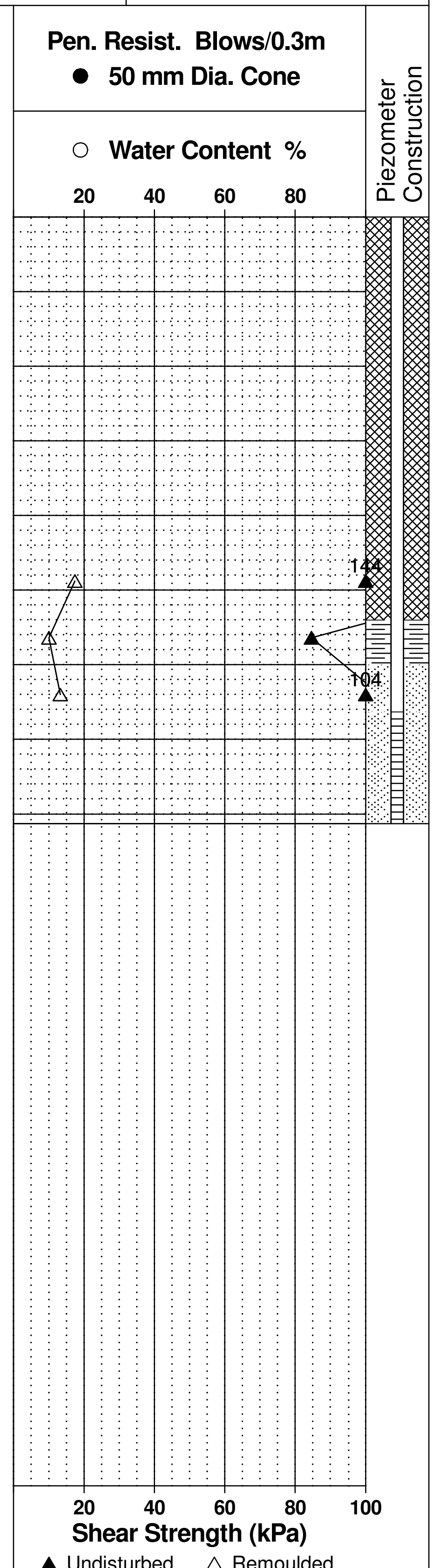
HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	70.24
FILL: Brown silty clay with topsoil, trace construction debris (0.20)		AU	1				
		SS	2	62	15	1	69.24
		SS	3	58	21	2	68.24
		SS	4	100	17		
Very stiff to firm, brown **SILTY CLAY**		SS	5	100	14	3	67.24
		SS	6	100	7	4	66.24
						5	65.24
- grey by 5.3m depth						6	64.24
						7	63.24
(8.13)		SS	7		4	8	62.24

End of Borehole

Practical refusal to augering at 8.13m depth

(BH dry and blocked at 3.63m depth - May 4, 2017)



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

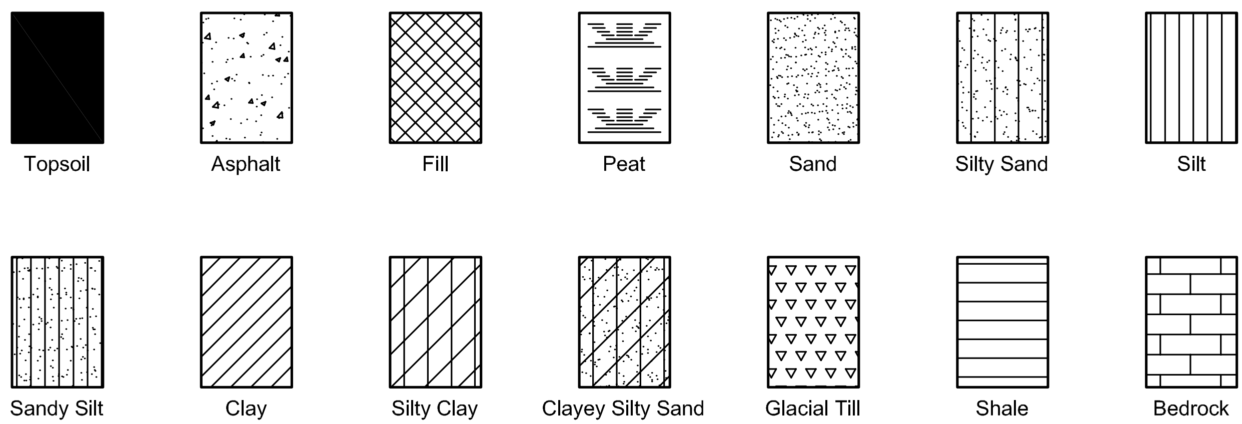
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

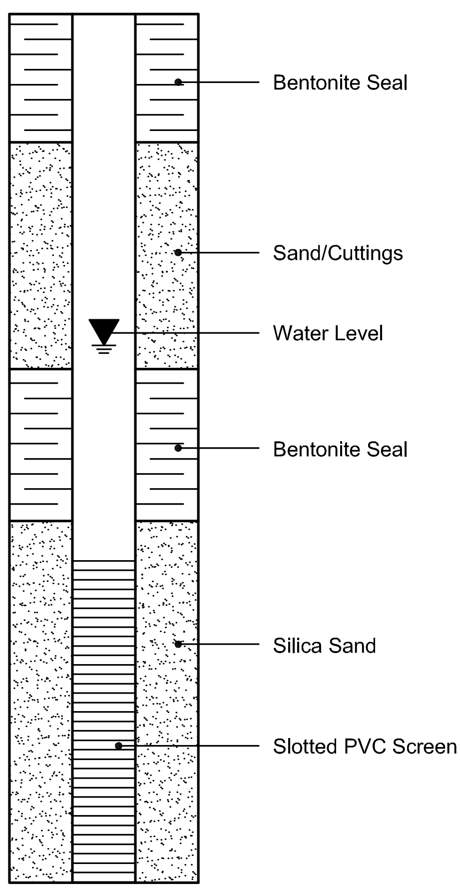
SYMBOLS AND TERMS (continued)

STRATA PLOT

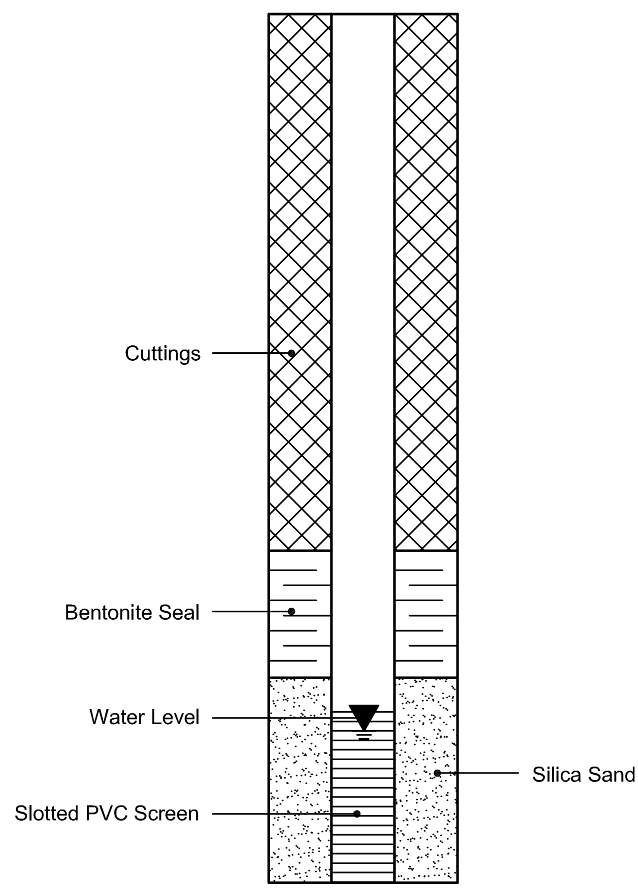


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 45.477N 75.513W

2024-01-09 19:22 UT

Requested by: 3030 St Joseph Boulevard

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.492	0.273	0.163	0.048
Sa (0.1)	0.571	0.328	0.203	0.065
Sa (0.2)	0.474	0.276	0.173	0.058
Sa (0.3)	0.358	0.209	0.133	0.046
Sa (0.5)	0.252	0.147	0.093	0.032
Sa (1.0)	0.123	0.072	0.046	0.016
Sa (2.0)	0.058	0.034	0.021	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.006	0.003	0.002	0.001
PGA (g)	0.304	0.177	0.110	0.035
PGV (m/s)	0.209	0.117	0.072	0.022

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Natural Resources Canada Ressources naturelles Canada

Canada

Table 1 - Proposed Multi-Storey Building - Summary of Reviewed Landslide Inventory Data

Location	Source	Site Code	Geographical Coordinate		Feature	Morphology	Scar Area	Age	Relief	Distance from PG6609	Surface Geology	Drift Thicknes	Bedrock
			Latitude	Longitude			(km2)		(m)	(m)		(m)	
Mississippi River	OF8600	Mss1	45.41279	-76.24891	Landslide	Source area with truncated debris field	0.08	Unknown	15.00	61.13	Marine Deposits	15 to 25	Granite
Mississippi River	OF8600	Mss2	45.41224	-76.25685	Landslide	Source area with debris field	0.03	Unknown	11.00	61.78	Marine Deposits	15 to 25	Granite
Mississippi River	OF8600	Mss3	45.40384	-76.24579	Landslide	Source area with debris field	0.11	Unknown	23.00	61.02	Marine Deposits	15 to 25	Marble
Mississippi River	OF8600	Mss4	45.40073	-76.25147	Landslide	Truncated source area	0.01	Unknown	14.00	61.53	Marine Deposits	15 to 25	Marble
Mississippi River	OF8600	Mss5	45.39957	-76.24593	Landslide	Source area with truncated debris field	0.02	Unknown	20.00	61.10	Erosional Terraces	15 to 25	Marble
Mississippi River	OF8600	Mss6	45.39906	-76.25294	Landslide	Source area with truncated debris field	0.04	Unknown	15.00	61.68	Marine Deposits	15 to 25	Marble
Mississippi River	OF8600	Mss7	45.39121	-76.25506	Landslide	Truncated source area	0.01	Unknown	15.00	62.00	Erosional Terraces	10 to 15	Interbedded Limestone and Shale
Mississippi River	OF8600	Mss8	45.38970	-76.25421	Landslide	Source area with truncated debris field	0.03	Unknown	12.00	61.96	Erosional Terraces	5 to 10	Interbedded Limestone and Shale
Mississippi River	OF8600	Mss9	45.38953	-76.25872	Landslide	Source area with truncated debris field	0.02	Unknown	15.00	62.33	Organic Deposits	5 to 10	Interbedded Limestone and Shale
Mississippi River	OF8600	Mss10	45.38715	-76.25825	Landslide	Source area with truncated debris field	0.01	Unknown	15.00	62.34	Erosional Terraces	5 to 10	Interbedded Limestone and Shale
Mississippi River	OF8600	Mss11	45.37849	-76.26739	Landslide	Truncated source area	0.03	Unknown	13.00	63.27	Erosional Terraces	50 to 100	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss12	45.37130	-76.26864	Landslide	Truncated source area	0.02	Unknown	11.00	63.53	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Mississippi Valley	OF8600	Mss13	45.36543	-76.27229	Landslide	Truncated source area	0.02	Unknown	12.00	63.97	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss14	45.36519	-76.27788	Landslide	Source area with truncated debris field	0.02	Unknown	12.00	64.43	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss15	45.36304	-76.27430	Landslide	Source area with truncated debris field	0.03	Unknown	15.00	64.19	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss16	45.36359	-76.26926	Landslide	Truncated source area	0.01	Unknown	17.00	63.77	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss17	45.36075	-76.26736	Landslide	Source area with debris field	0.06	Unknown	20.00	63.69	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss18	45.36124	-76.27193	Landslide	Truncated source area	0.01	Unknown	12.00	64.04	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss19	45.36122	-76.27561	Landslide	Source area with debris field	0.04	Unknown	10.00	64.34	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite

PATERSON GROUP

Table 1 - Proposed Multi-Storey Building - Summary of Reviewed Landslide Inventory Data													
Location	**Source**	**Site Code**	**Geographical Coordinate**		**Feature**	**Morphology**	**Scar Area**	**Age**	**Relief**	**Distance from PG6609**	**Surface Geology**	**Drift Thicknes**	**Bedrock**
			Latitude	**Longitude**			**(km2)**		**(m)**	**(m)**		**(m)**	
Mississippi River	OF8600	Mss20	45.36267	-76.28135	Landslide	Source area with debris field	0.02	Unknown	10.00	64.77	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss21	45.35604	-76.26640	Landslide	Source area with debris field	0.11	Unknown	15.00	63.74	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss22	45.35416	-76.27441	Landslide	Truncated source area	0.01	Unknown	17.00	64.43	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss23	45.35244	-76.27982	Landslide	Truncated source area	0.01	Unknown	25.00	64.91	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss24	45.35168	-76.28166	Landslide	Truncated source area	0.01	Unknown	25.00	65.08	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Mississippi River	OF8600	Mss25	45.35103	-76.28318	Landslide	Truncated source area	0.02	Unknown	20.00	65.22	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy1	45.35143	-76.26277	Landslide	Source area with truncated debris field	0.01	Unknown	16.00	63.57	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy2	45.35023	-76.26130	Landslide, possibly	Source area with debris field	0.01	Unknown	***	63.49	***	***	***
Cody Creek	OF8600	Cdy3	45.34522	-76.26452	Landslide	Truncated source area	0.01	Unknown	21.00	63.89	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy4	45.34223	-76.26721	Landslide	Truncated source area	0.02	Unknown	17.00	64.19	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy5	45.33939	-76.25827	Landslide	Source area with truncated debris field	0.07	Unknown	17.00	63.56	Erosional Terraces	15 to 25	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy6	45.33979	-76.24991	Landslide	Source area with truncated debris field	0.06	Unknown	13.00	62.88	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy7	45.34175	-76.24524	Landslide, possibly	Source area with truncated debris field	0.10	Unknown	***	62.45	***	***	***
Cody Creek	OF8600	Cdy8	45.33762	-76.24262	Landslide	Source area with truncated debris field	0.03	Unknown	23.00	62.37	Marine Deposits	10 to 15	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy9	45.33822	-76.23647	Landslide	Source area with truncated debris field	0.01	Unknown	17.00	61.86	Marine Deposits	5 to 10	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy10	45.33477	-76.23220	Landslide	Source area with truncated debris field	0.06	Unknown	12.00	61.63	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy11	45.33386	-76.23415	Landslide, probably	Source area with truncated debris field	0.04	Unknown	**	61.81	**	**	**
Cody Creek	OF8600	Cdy12	45.32989	-76.22645	Landslide, probably	Source area with truncated debris field	0.04	Unknown	**	61.34	**	**	**
Cody Creek	OF8600	Cdy13	45.32654	-76.22004	Landslide, probably	Source area with truncated debris field	0.03	Unknown	**	60.94	**	**	**

Table 1 - Proposed Multi-Storey Building - Summary of Reviewed Landslide Inventory Data

Location	Source	Site Code	Geographical Coordinate		Feature	Morphology	Scar Area	Age	Relief	Distance from PG6609	Surface Geology	Drift Thicknes	Bedrock
			Latitude	Longitude			(km2)		(m)	(m)		(m)	
Cody Creek	OF8600	Cdy14	45.32031	-76.21358	Landslide	Source area with truncated debris field; isolated areas of debris field	0.05	Unknown	24.00	60.65	Marine Deposits	10 to 15	Interbedded Limestone and Dolomite
Cody Creek	OF8600	Cdy15	45.34728	-76.23587	Landslide, probably	Debris field within a narrow stream valley	0.01	Unknown	5.00	61.53	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite
Madawaska Lake reservoir	OF8600	Mdw1	45.40855	-76.35190	Landslide, former site of	Inundated beneath lake waters	**	Unknown	7.00	69.62	Marine Deposits	10 to 15	Marble
Fitzroy	OF8600	Ftz1	45.50319	-76.22097	Landslide	Truncated source area	0.03	Unknown	10.00	58.39	Alluvial Sediments	5 to 10	Interbedded Limestone and Dolomite
Fitzroy	OF8600	Ftz2	45.50437	-76.21394	Landslide	Truncated source area	0.02	Unknown	16.00	57.82	Alluvial Sediments	5 to 10	Interbedded Limestone and Dolomite
Fitzroy	OF8600	Ftz3	45.49835	-76.15980	Landslide	Source area with truncated debris field	0.16	Unknown	27.00	53.33	Erosional Terraces	10 to 15	Interbedded Limestone and Dolomite
Fitzroy	OF8600	Ftz4	45.50664	-76.13974	Landslide	Truncated source area	0.23	Unknown	11.00	51.74	Erosional Terraces	5 to 10	Interbedded Limestone and Dolomite
Buckhams Bay	OF8600	BkB1	45.48572	-76.10521	Landslide	Source area with truncated debris field	0.49	Unknown	34.00	48.80	Erosional Terraces	15 to 25	Interbedded Limestone and Dolomite
Buckhams Bay	OF8600	BkB2	45.48122	-76.10138	Landslide	Source area with truncated debris field	0.13	Unknown	30.00	48.47	Erosional Terraces	15 to 25	Interbedded Limestone and Dolomite
Buckhams Bay	OF8600	BkB3	45.47977	-76.09564	Landslide	Source area with truncated debris field	0.10	Unknown	30.00	48.00	Erosional Terraces	15 to 25	Interbedded Limestone and Dolomite
Carp Creek	OF8600	Crp1	45.34812	-76.04299	Landslide	Source area with debris field	0.10	Unknown	20.00	46.26	Marine Deposits	25 to 50	Interbedded Limestone and Shale
Rideau River	OF8600	Rid1	45.38818	-75.70428	Landslide	Truncated source area	0.05	Unknown	15.00	18.91	Erosional Terraces	5 to 10	Limestone
Rideau River	OF8600	Rid2	45.32436	-75.69166	Landslide	Source area with debris field	0.06	Unknown	30.00	23.21	Alluvial Sediments	15 to 25	Interbedded Dolomite and Sandstone
Rideau River	OF8600	Rid3	45.28377	-75.69606	Landslide, possibly	Truncated source area?	0.01	Unknown	12.00	27.26	Erosional Terraces	25 to 50	Interbedded Dolomite and Sandstone
Rockcliffe	OF8600	Rkf1	45.45147	-75.67312	Landslide, probably	Truncated source area	0.02	Unknown	25.00	13.53	Erosional Terraces	1 to 3	Shale
Gloucester	OF8600	Glt1	45.44963	-75.59729	Landslide	Source area with debris field	0.12	About 1000 cal yr BP	30.00	7.66	Erosional Terraces	25 to 50	Interbedded Limestone and Dolomite
Orleans	OF8600	Oln1	45.45962	-75.55209	Landslide, possibly	Truncated source area?	0.02	Unknown	30.00	3.81	Marine Deposits Erosional Terraces	10 to 50	Dolomite
Orleans	OF8600	Oln2	45.45719	-75.54766	Landslide	Debris field within a narrow stream valley	0.11	Unknown	17.00	3.68	Marine Deposits	25 to 50	Shale

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Table 1 - Proposed Multi-Storey Building - Summary of Reviewed Landslide Inventory Data

Location	Source	Site Code	Geographical Coordinate		Feature	Morphology	Scar Area	Age	Relief	Distance from PG6609	Surface Geology	Drift Thicknes	Bedrock
			Latitude	Longitude			(km2)		(m)	(m)		(m)	
Orleans	OF8600	Oln3	45.46016	-75.54108	Landslide	Debris field within a narrow stream valley	0.05	Unknown	5.00	3.04	Marine Deposits	50 to 100	Shale
Orleans	OF8600	Oln4	45.45726	-75.54049	Landslide	Debris field within a narrow stream valley	0.02	Unknown	5.00	3.24	Nearshore Marine	50 to 100	Shale
Orleans	OF8600	Oln5	45.45487	-75.53897	Landslide	Debris field within a narrow stream valley	0.02	Unknown	9.00	3.37	Marine Deposits	50 to 100	Interbedded Limestone and Dolomite
Orleans	OF8600	Oln6	45.45863	-75.53838	Landslide	Debris field within a narrow stream valley	0.01	Unknown	3.00	3.00	Nearshore Marine	50 to 100	Shale
Orleans	OF8600	Oln7	45.45977	-75.53858	Landslide	Debris field within a narrow stream valley	0.00	Unknown	2.00	2.92	Nearshore Marine	50 to 100	Shale
Orleans	OF8600	Oln8	45.46051	-75.53690	Landslide	Debris field within a narrow stream valley	0.01	Unknown	7.00	2.76	Nearshore Marine	50 to 100	Shale
Orleans	OF8600	Oln9	45.46092	-75.53424	Landslide	Debris field within a narrow stream valley	0.02	Unknown	3.00	2.57	Nearshore Marine	50 to 100	Shale
Orleans	OF8600	Oln10	45.46378	-75.53684	Landslide	Truncated source area	0.07	Unknown	20.00	2.50	Nearshore Marine	50 to 100	Shale
Orleans	OF8600	Oln11	45.46497	-75.53146	Landslide	Truncated source area	0.06	Unknown	18.00	2.08	Nearshore Marine	50 to 100	Shale
Orleans	OF8600	Oln12	45.46706	-75.52809	Landslide	Truncated source area	0.05	Unknown	18.00	1.71	Nearshore Marine	25 to 50	Shale
Orleans	OF8600	Oln13	45.47042	-75.52019	Landslide, probably	Truncated source area	0.07	Unknown	20.00	0.97	Marine Deposits	25 to 50	Shale
Orleans	OF8600	Oln14	45.48981	-75.50782	Landslide	Source area with debris field	0.08	Late Holocene?	18.00	1.57	Alluvial Sediments	15 to 25	Interbedded Limestone and Dolomite
Orleans	OF8600	Oln15	45.48871	-75.47487	Landslide, probably	Truncated source area	0.08	Unknown	24.00	3.43	Offshore Marine/Erosional Terrace	25 to 50	Interbedded Limestone and Dolomite
Orleans	OF8600	Oln16	45.48586	-75.47170	Landslide	Debris field within a narrow stream valley	0.07	Unknown	29.00	3.55	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
Orleans	OF8600	Oln17	45.48877	-75.46692	Landslide	Debris field within a narrow stream valley	0.04	Unknown	8.00	4.04	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite
Orleans	OF8600	Oln18	45.49016	-75.46497	Landslide	Debris field within a narrow stream valley	0.03	Unknown	10.00	4.25	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite
Cumberland	OF8600	Cmb1	45.51313	-75.43362	Landslide	Truncated source area	0.14	Unknown	35.00	7.79	Erosional Terraces	15 to 25	Shale
Cumberland	OF8600	Cmb2	45.51302	-75.40335	Landslide	Source area with debris field	0.04	Unknown	24.00	9.97	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite

Table 1 - Proposed Multi-Storey Building - Summary of Reviewed Landslide Inventory Data

Location	Source	Site Code	Geographical Coordinate		Feature	Morphology	Scar Area	Age	Relief	Distance from PG6609	Surface Geology	Drift Thicknes	Bedrock
			Latitude	Longitude			(km2)		(m)	(m)		(m)	
Cumberland	OF8600	Cmb3	45.51737	-75.38140	Landslide	Source area with debris field	0.53	relatively young, less than 2000(?) yrs	30.00	11.83	Erosional Terraces	50 to 100	Dolomite
Cumberland	OF8600	Cmb4	45.51651	-75.33631	Landslide	Source area with debris field	0.02	Unknown	49.00	15.27	Nearshore Marine	25 to 50	Interbedded Limestone and Dolomite
Mer Bleue paleochannel	OF8600	MBu1	45.43409	-75.53765	Landslide	Truncated source area	0.02	Unknown	13.00	5.43	Erosional Terraces	25 to 50	Interbedded Limestone and Shale
Mer Bleue paleochannel	OF8600	MBu2	45.43092	-75.51828	Landslide	Source area with debris field	0.12	Unknown	15.00	5.43	Nearshore Marine	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu3	45.42843	-75.51485	Landslide	Source area with debris field	0.01	Unknown	12.00	5.70	Nearshore Marine	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu4	45.42782	-75.51290	Landslide	Truncated source area	0.01	Unknown	**	5.77	Nearshore Marine	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu5	45.42639	-75.50741	Landslide, former site of	Completely altered	N.A.	Unknown	**	5.96	Nearshore Marine	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu6	45.42500	-75.50289	Landslide, former site of	Completely altered	N.A.	Unknown	**	6.16	Nearshore Marine	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu7	45.42364	-75.49702	Landslide, former site of	Completely altered	N.A.	Unknown	**	6.40	Nearshore Marine	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu8	45.42335	-75.49197	Landslide, former site of	Completely altered	N.A.	Unknown	**	6.53	Nearshore Marine	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu9	45.42158	-75.48102	Landslide	Truncated source area	0.03	Unknown	15.00	7.02	Erosional Terraces	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu10	45.42089	-75.47444	Landslide	Truncated source area	0.03	Unknown	15.00	7.31	Nearshore Marine	25 to 50	Shale
Mer Bleue paleochannel	OF8600	MBu11	45.41891	-75.46077	Landslide	Truncated source area	0.03	Unknown	14.00	8.06	Nearshore Marine	15 to 25	Shale
Mer Bleue paleochannel	OF8600	MBu12	45.41829	-75.45649	Landslide	Truncated source area	0.01	Unknown	15.00	8.31	Nearshore Marine	15 to 25	Shale
Mer Bleue paleochannel	OF8600	MBu13	45.41206	-75.27053	Landslide	Source area with debris field	1.42	about 5200 cal yrBP	19.00	21.39	Nearshore Marine	25 to 50	Interbedded Limestone and Shale
Beta-90881	OF7432	1	45.46110	-75.26110	Landslide	*	*	3050±70	20.00	20.83	Nearshore Marine	15 to 25	Interbedded Limestone and Shale
Beta-122473	OF7432	1	45.44170	-75.22220	Landslide	*	*	4590±40	8.00	24.31	Nearshore Marine	25 to 50	Interbedded Limestone and Shale
Beta-122475	OF7432	1	45.44240	-75.19240	Landslide	*	*	2760±50	20.00	26.72	Erosional Terraces	25 to 50	Interbedded Limestone and Shale

Table 1 - Proposed Multi-Storey Building - Summary of Reviewed Landslide Inventory Data

Location	Source	Site Code	Geographical Coordinate		Feature	Morphology	Scar Area	Age	Relief	Distance from PG6609	Surface Geology	Drift Thicknes	Bedrock
			Latitude	Longitude			(km2)		(m)	(m)		(m)	
Beta-127281	OF7432	1	45.54160	-75.24160	Landslide	*	*	5130±60	53.00	23.59	Nearshore Marine	10 to 15	Limestone
Beta-127284	OF7432	1	45.52080	-75.26670	Landslide	*	*	4440±80	21.00	20.92	Erosional Terraces	25 to 50	Interbedded Limestone and Shale
Beta-127244	OF7432	1	45.50000	-75.20280	Landslide	*	*	4570±70	30.00	25.69	Erosional Terraces	25 to 50	Interbedded Limestone and Shale
Beta-122472	OF7432	1	45.48330	-75.19170	Landslide	*	*	4520±50	30.00	26.47	Nearshore Marine	15 to 25	Interbedded Limestone and Shale
Beta-127282	OF7432	1	45.47500	-75.12920	Landslide	*	*	4540±90	24.00	31.61	Nearshore Marine	15 to 25	Interbedded Limestone and Shale
Beta-127283	OF7432	1	45.52500	-75.01110	Landslide	*	*	4530±60	12.00	41.71	Erosional Terraces	10 to 15	Interbedded Limestone and Shale
Beta-122478	OF7432	1	45.51390	-75.00280	Landslide	*	*	4700±50	15.00	42.23	Erosional Terraces	15 to 25	Interbedded Limestone and Shale
Beta-122471	OF7432	1	45.51850	-74.95570	Landslide	*	*	1870±40	26.00	46.14	**	**	**
Beta-127242	OF7432	1	45.51380	-74.93750	Landslide	*	*	4820±70	26.00	47.58	**	**	**
Beta-122474	OF7432	1	45.53610	-75.15830	Landslide	*	*	4470±50	**	30.01	Nearshore Marine	25 to 50	Limestone
GSC-1922	OF7432	2	45.54370	-75.40110	Landslide	*	*	4620±80	81.00	12.09	Marine Deposits	15 to 25	Felsic Intrusive Rocks
GSC-2068	OF7432	4	45.52080	-75.49170	Landslide	*	*	6240±70	59.00	5.44	Marine Deposits	25 to 50	Dolomite
UCIAMS-71217	OF7432	6	45.57980	-75.04260	Landslide	*	*	7105±20	35.00	40.55	Erosional Terraces	50 to 100	Shale
UCIAMS-71211	OF7432	7	45.57020	-75.11560	Landslide	*	*	7140±20	31.00	34.49	Marine Deposits	50 to 100	Interbedded Limestone and Dolomite
GSC-1741	OF7432	10	45.46500	-75.75130	Landslide	*	*	120±150	**	19.69	Marine Deposits	25 to 50	Dolomite
UCIAMS-88796	OF7432	11	45.48290	-75.93490	Landslide	*	*	1125±15	29.00	34.76	Marine Deposits	25 to 50	Felsic Intrusive Rocks
UCIAMS-88704	OF7432	11	45.48530	-75.93630	Landslide	*	*	2805±20	29.00	34.89	Marine Deposits	25 to 50	Felsic Intrusive Rocks
GSC-6233	OF7432	11	45.48310	-75.93320	Landslide	*	*	7050±80	25.00	34.62	Marine Deposits	25 to 50	Felsic Intrusive Rocks
UCIAMS-88816	OF7432	11	45.48020	-75.93090	Landslide	*	*	200±15	24.00	34.43	Marine Deposits	15 to 25	Felsic Intrusive Rocks

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			Latitude	Longitude			(km2)		(m)	(m)		(m)	
GSC-6449	OF7432	11	45.47180	-75.91290	Landslide	*	*	1080±70	15.00	32.95	Marine Deposits	15 to 25	Interbedded Limestone and Dolomite
UCIAMS-88703	OF7432	11	45.47990	-75.91740	Landslide	*	*	180±20	26.00	33.32	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
GSC-6318	OF7432	11	45.47860	-75.91180	Landslide	*	*	1030±70	24.00	32.86	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
UCIAMS-88806	OF7432	11	45.47730	-75.90280	Landslide	*	*	1895±25	12.00	32.11	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
GSC-6482	OF7432	11	45.48120	-75.90670	Landslide	*	*	1210±50	8.00	32.44	Marine Deposits	25 to 50	Interbedded Limestone and Dolomite
GSC-6433	OF7432	11	45.48540	-75.89640	Landslide	*	*	1440±50	18.00	31.60	Marine Deposits	15 to 25	Felsic Intrusive Rocks
UCIAMS-88818	OF7432	11	45.48520	-75.90600	Landslide	*	*	2755±20	22.00	32.39	Marine Deposits	25 to 50	Felsic Intrusive Rocks
GSC-6355	OF7432	11	45.48250	-75.91180	Landslide	*	*	1170±50	27.00	32.86	Marine Deposits	25 to 50	Felsic Intrusive Rocks
Beta-139135	OF7432	11	45.49650	-75.92780	Landslide	*	*	310±40	10.00	34.25	Marine Deposits	50 to 100	Felsic Intrusive Rocks
UCIAMS-122468	OF7432	12	45.53530	-76.03060	Landslide	*	*	1095±20	21.00	43.17	Marine Deposits	15 to 25	Felsic Intrusive Rocks
UCIAMS-106656	OF7432	13	45.54090	-76.04890	Landslide	*	*	1150±15	22.00	44.76	Marine Deposits	25 to 50	Felsic Intrusive Rocks
UCIAMS-171460	OF7432	14	45.55390	-76.13020	Landslide	*	*	1305±20	9.00	51.61	Nearshore Marine	50 to 100	Felsic Intrusive Rocks
UCIAMS-171459	OF7432	15	45.55130	-76.14060	Landslide	*	*	185±20	9.00	52.40	Nearshore Marine	50 to 100	Felsic Intrusive Rocks
UCIAMS-106587	OF7432	16	45.55190	-76.28630	Landslide	*	*	1180±20	24.00	64.27	Erosional Terraces	15 to 25	Felsic Intrusive Rocks
UCIAMS-106575	OF7432	17	45.61920	-76.37190	Landslide	*	*	955±15	32.00	72.62	**	**	**
UCIAMS-106650	OF7432	18	45.50140	-76.28260	Landslide	*	*	1145±20	52.00	63.45	Nearshore Marine	15 to 25	Felsic Intrusive Rocks
UCIAMS-106581	OF7432	19	45.51700	-76.27470	Landslide	*	*	5830±20	34.00	62.90	Nearshore Marine	15 to 25	Felsic Intrusive Rocks
UCIAMS-122453	OF7432	20	45.54620	-76.52600	Landslide	*	*	5745±20	**	83.79	**	**	**
UCIAMS-137113	OF7432	21	45.72570	-75.89150	Landslide	*	*	4525±20	52.00	42.69	**	**	**

Table 1 - Proposed Multi-Storey Building - Summary of Reviewed Landslide Inventory Data

Location	Source	Site Code	Geographical Coordinate		Feature	Morphology	Scar Area	Age	Relief	Distance from PG6609	Surface Geology	Drift Thicknes	Bedrock
			Latitude	Longitude			(km2)		(m)	(m)		(m)	
UCIAMS-137101	OF7432	22	45.69440	-75.89960	Landslide	*	*	90±20	23.00	40.78	**	**	**
UCIAMS-122455	OF7432	23	45.80960	-75.95980	Landslide	*	*	940±15	25.00	53.61	**	**	**

'*' - Indicates information not provided by source (Geological Survey of Canada Open File 7432)

'' Indicates information could not be interpreted from available mapping.**

APPENDIX 2

FIGURE 1 - KEY PLAN

HISTORICAL PHOTOGRAPHS

FIGURE 2 to 7 - SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG6609-1 - TEST HOLE LOCATION PLAN

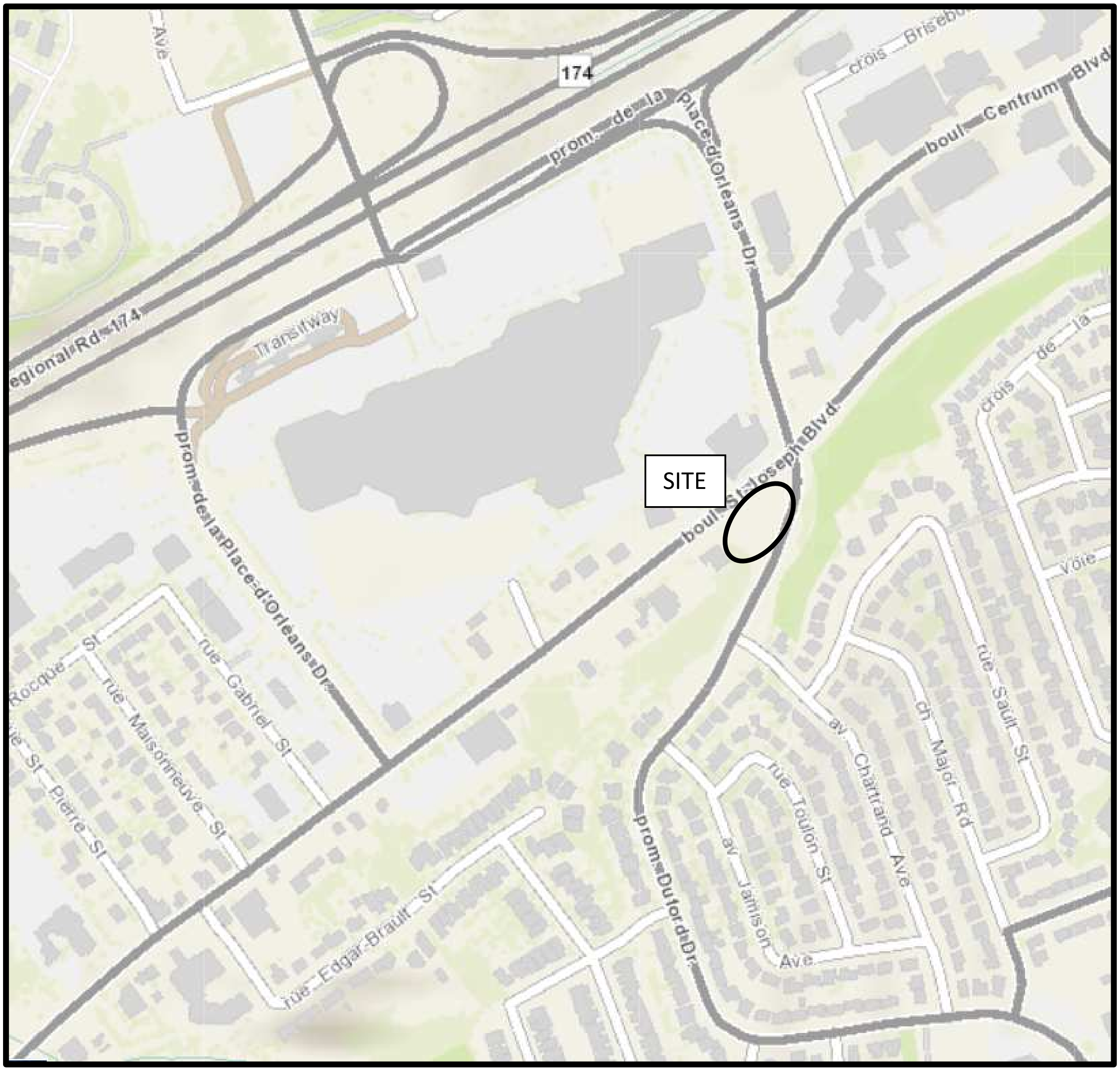


FIGURE 1

KEY PLAN

PATERSON GROUP

Photo 1: Localized slope failure occurring in the mid 1960s adjacent to Duford Drive. Subject site is located within the foreground of the photograph. St. Joseph Boulevard is in the background. Ground surface adjacent to Duford Drive is noted to be free of vegetation, which is indicative that construction of the subject roadway section and cutting of the subject slope was recently completed. It is suspected that the exposed slope was re-shaped to an unstable slope angle as part of the construction work at that time.

Photo 2: Same localized slope failure, which occurred in mid 1960s.

Photo 3: Street view image from Google Earth of former slope failure area (presented in Photos 1 and 2) adjacent to Duford Drive. Ground noted to be re-shaped with a terraced slope in front of reinstated slope.

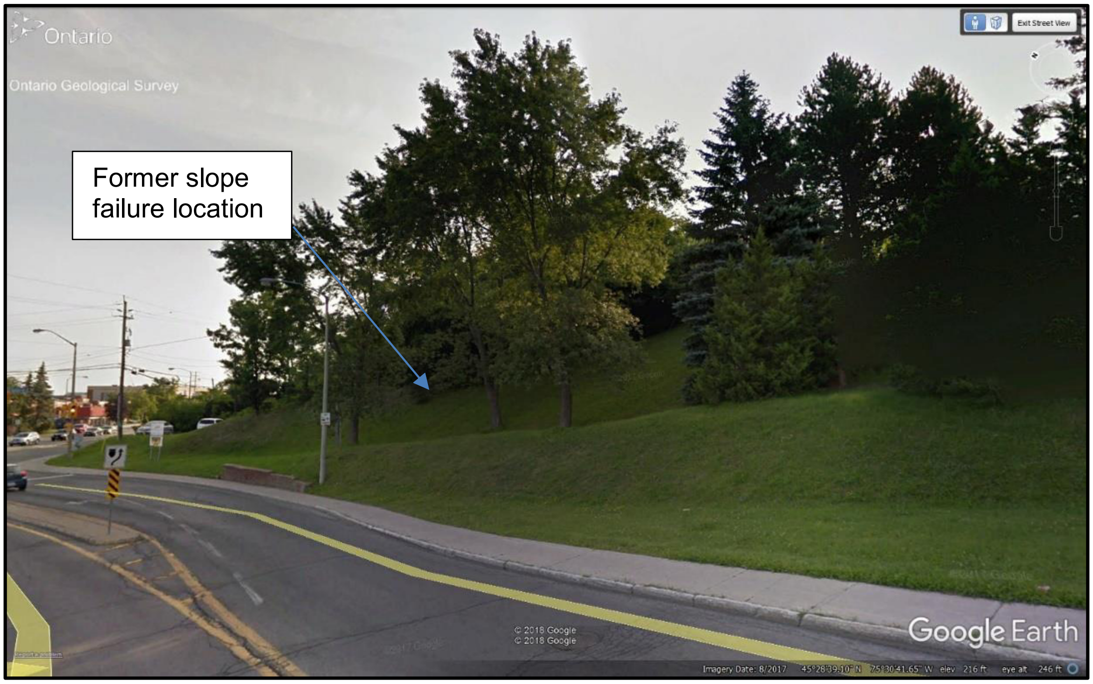
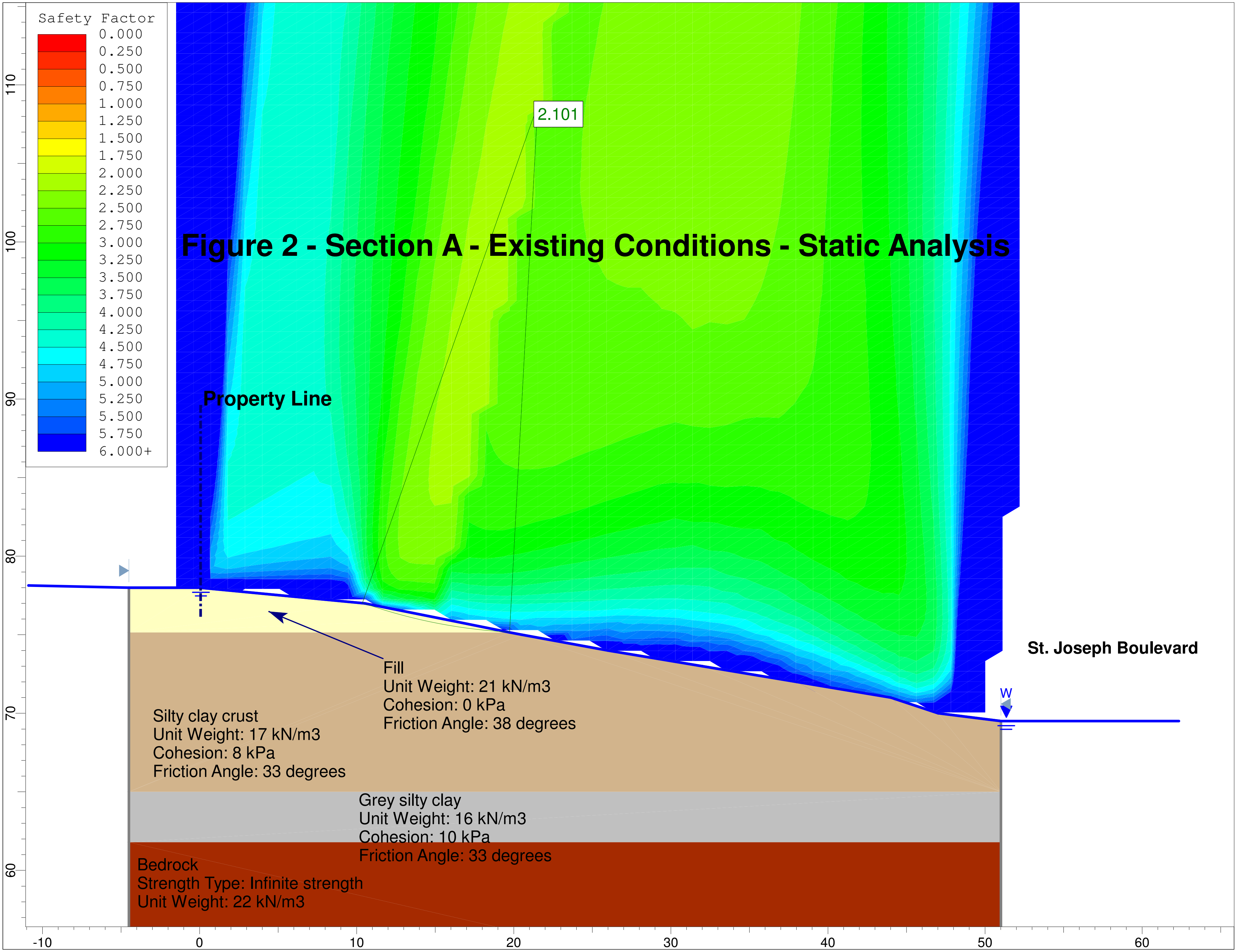
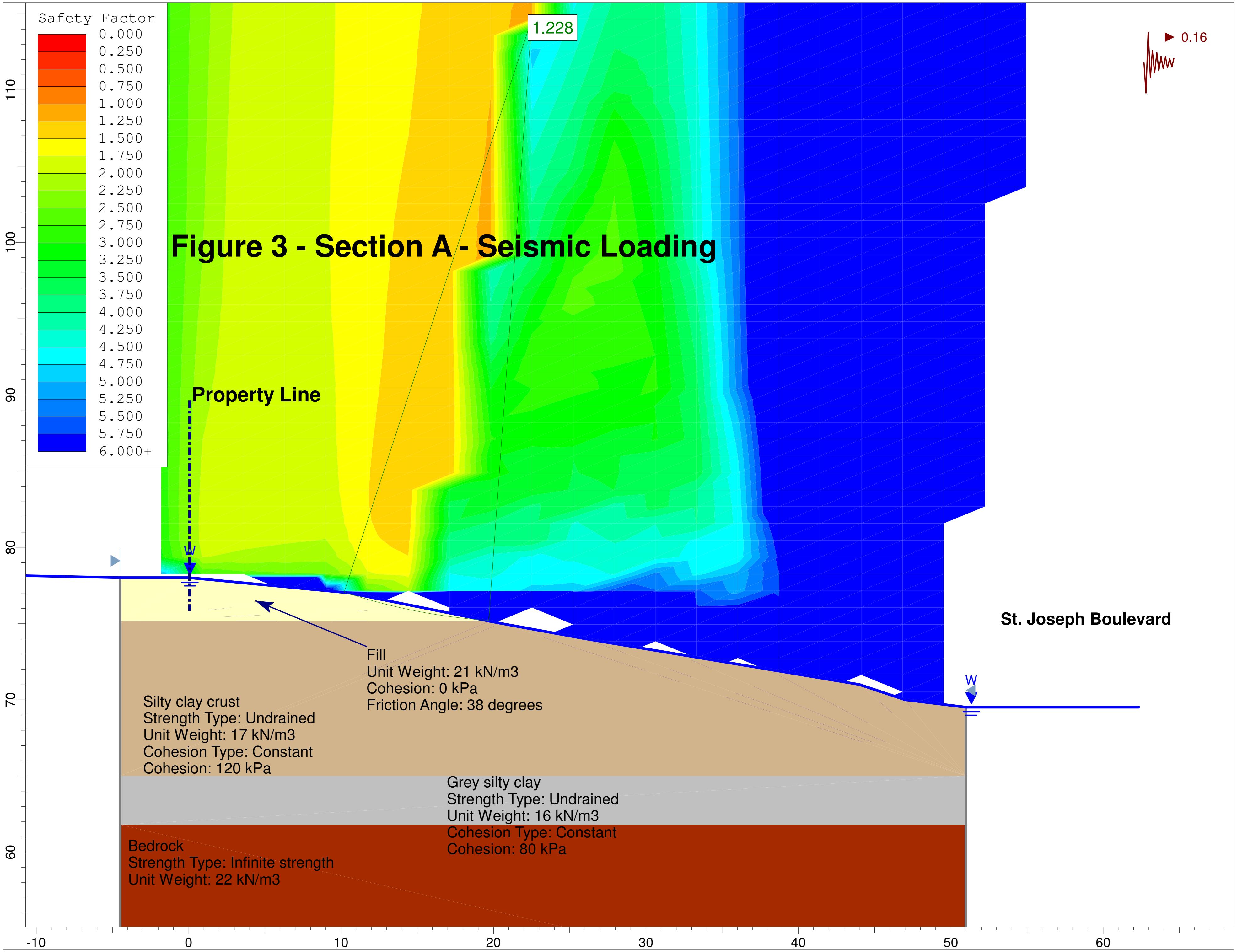
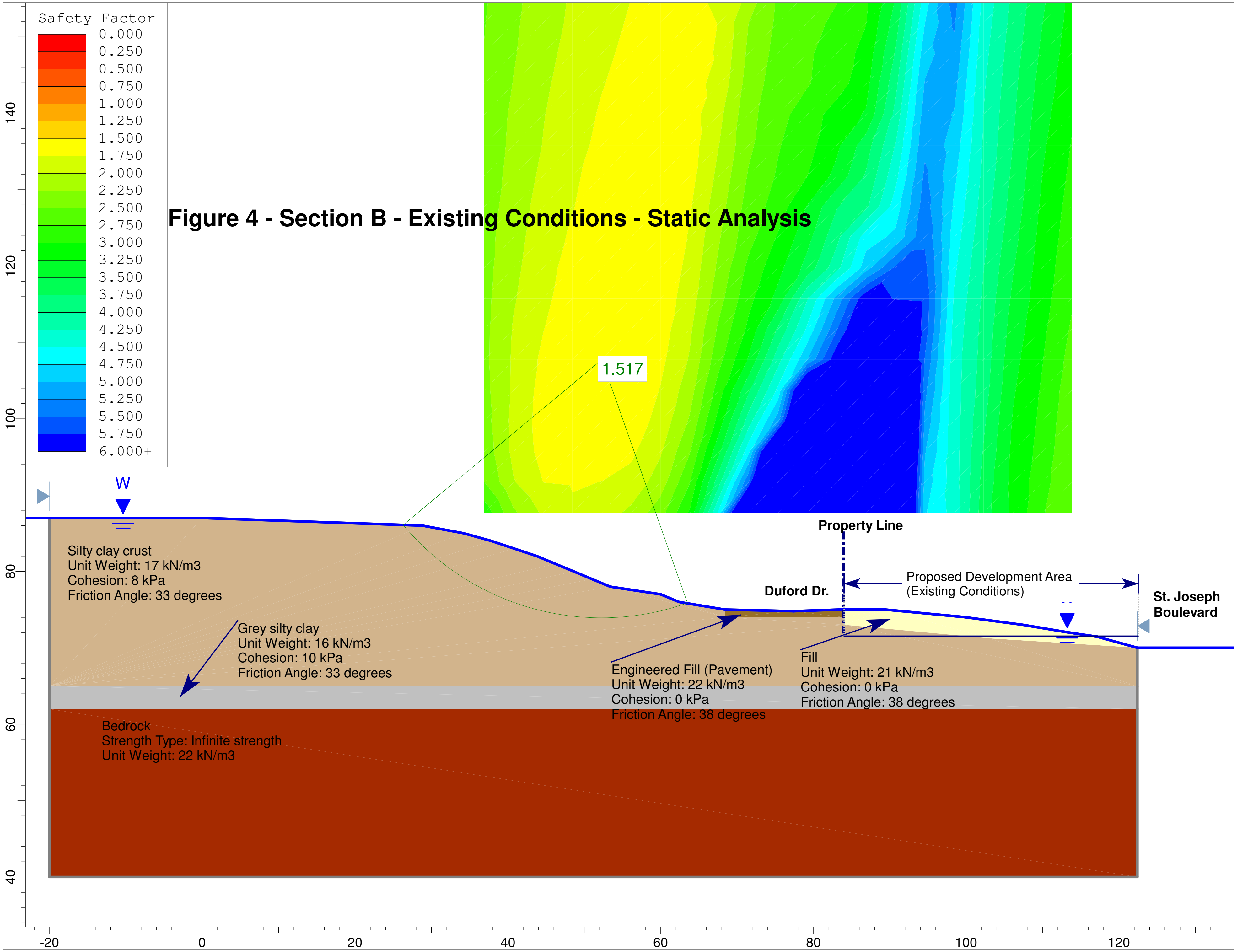


Photo 4: Street view image from Google Earth of the same former slope failure area presented in Photos 1 and 2. The ground surface is noted to be stable with no signs of slope instability.

Photo 5: Area of former slope failure noted in Photos 1 and 2. Subject site is property along the right side of the photograph.


Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
2.101
Figure 2 - Section A - Existing Conditions - Static Analysis
Property Line
St. Joseph Boulevard
W
Fill
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Silty clay crust
Unit Weight: 17 kN/m3
Cohesion: 8 kPa
Friction Angle: 33 degrees
Grey silty clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
110
100
90
80
70
60
-10
0
10
20
30
40
50
60


Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.228
0.16
Figure 3 - Section A - Seismic Loading
Property Line
W
St. Joseph Boulevard
Fill
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Silty clay crust
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 120 kPa
Grey silty clay
Strength Type: Undrained
Unit Weight: 16 kN/m3
Cohesion Type: Constant
Cohesion: 80 kPa
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
110
100
90
80
70
60
-10
0
10
20
30
40
50
60



Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Figure 4 - Section B - Existing Conditions - Static Analysis
1.517
W
Silty clay crust
Unit Weight: 17 kN/m3
Cohesion: 8 kPa
Friction Angle: 33 degrees
Grey silty clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
Property Line
Duford Dr.
Proposed Development Area
(Existing Conditions)
St. Joseph
Boulevard
Engineered Fill (Pavement)
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Fill
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
140
120
100
80
60
40
-20
0
20
40
60
80
100
120

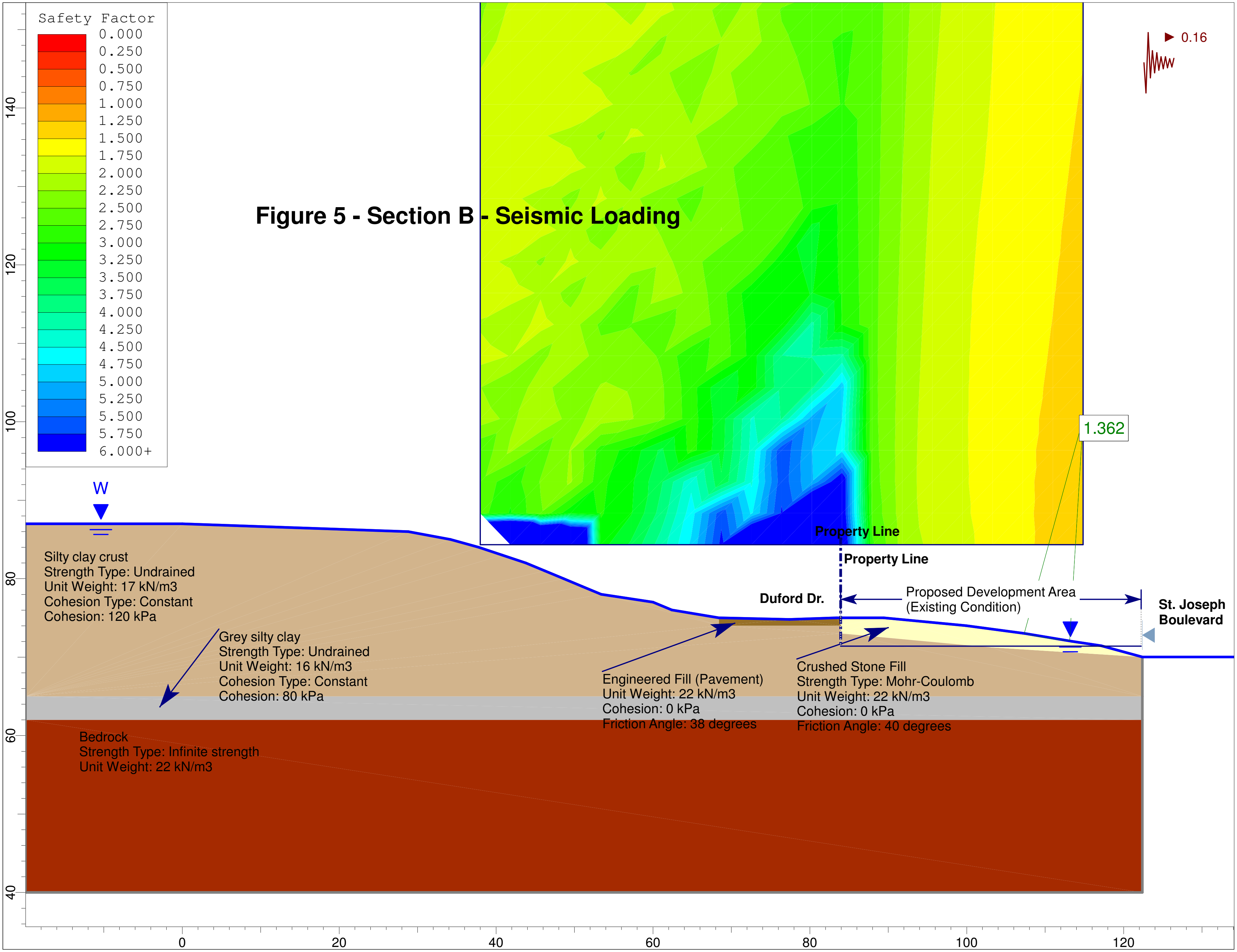


Figure 5 - Section B - Seismic Loading
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
1.362
W
Property Line
Property Line
Duford Dr.
Proposed Development Area
(Existing Condition)
St. Joseph
Boulevard
Silty clay crust
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 120 kPa
Grey silty clay
Strength Type: Undrained
Unit Weight: 16 kN/m3
Cohesion Type: Constant
Cohesion: 80 kPa
Engineered Fill (Pavement)
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Crushed Stone Fill
Strength Type: Mohr-Coulomb
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 40 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
140
120
100
80
60
40
0
20
40
60
80
100
120

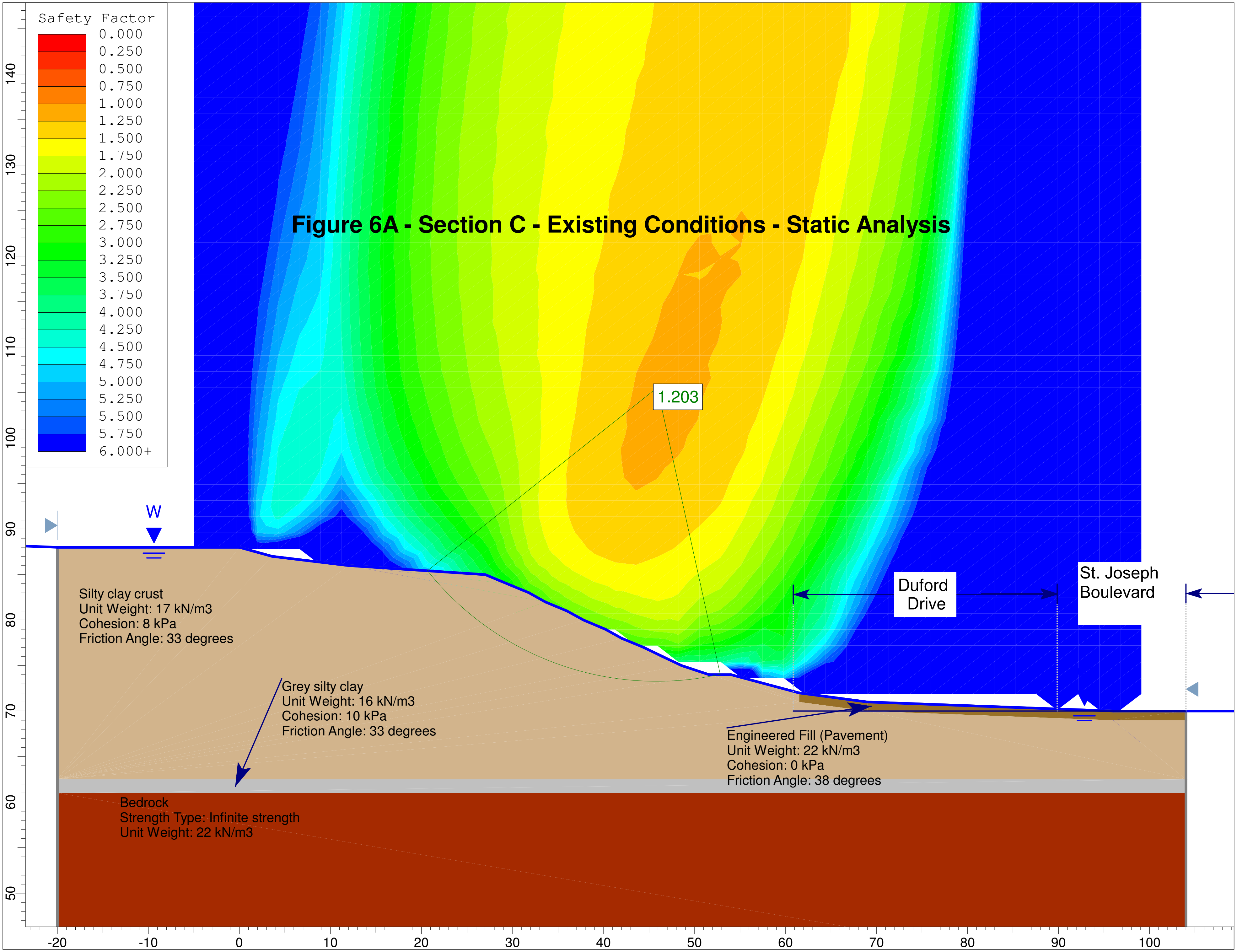
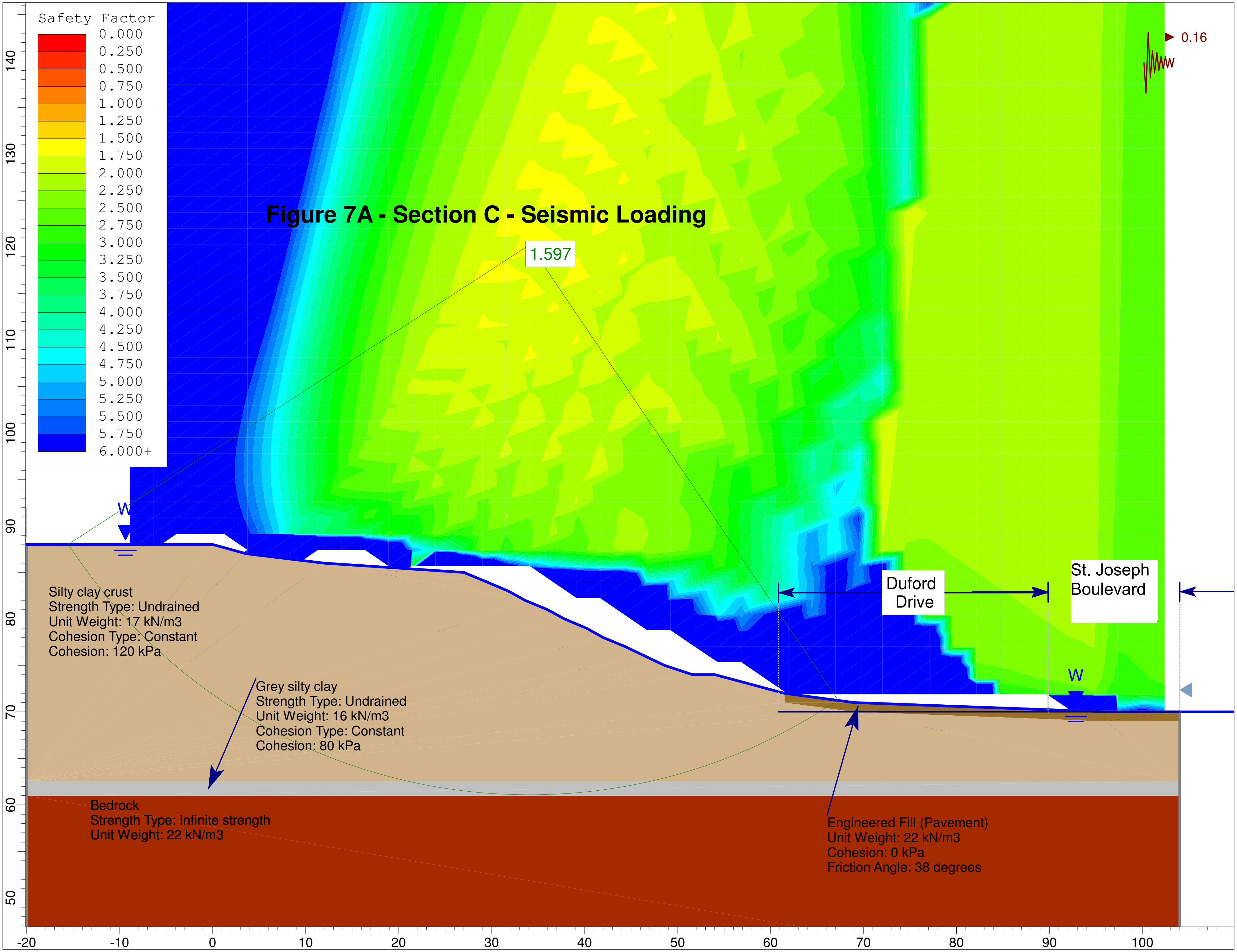
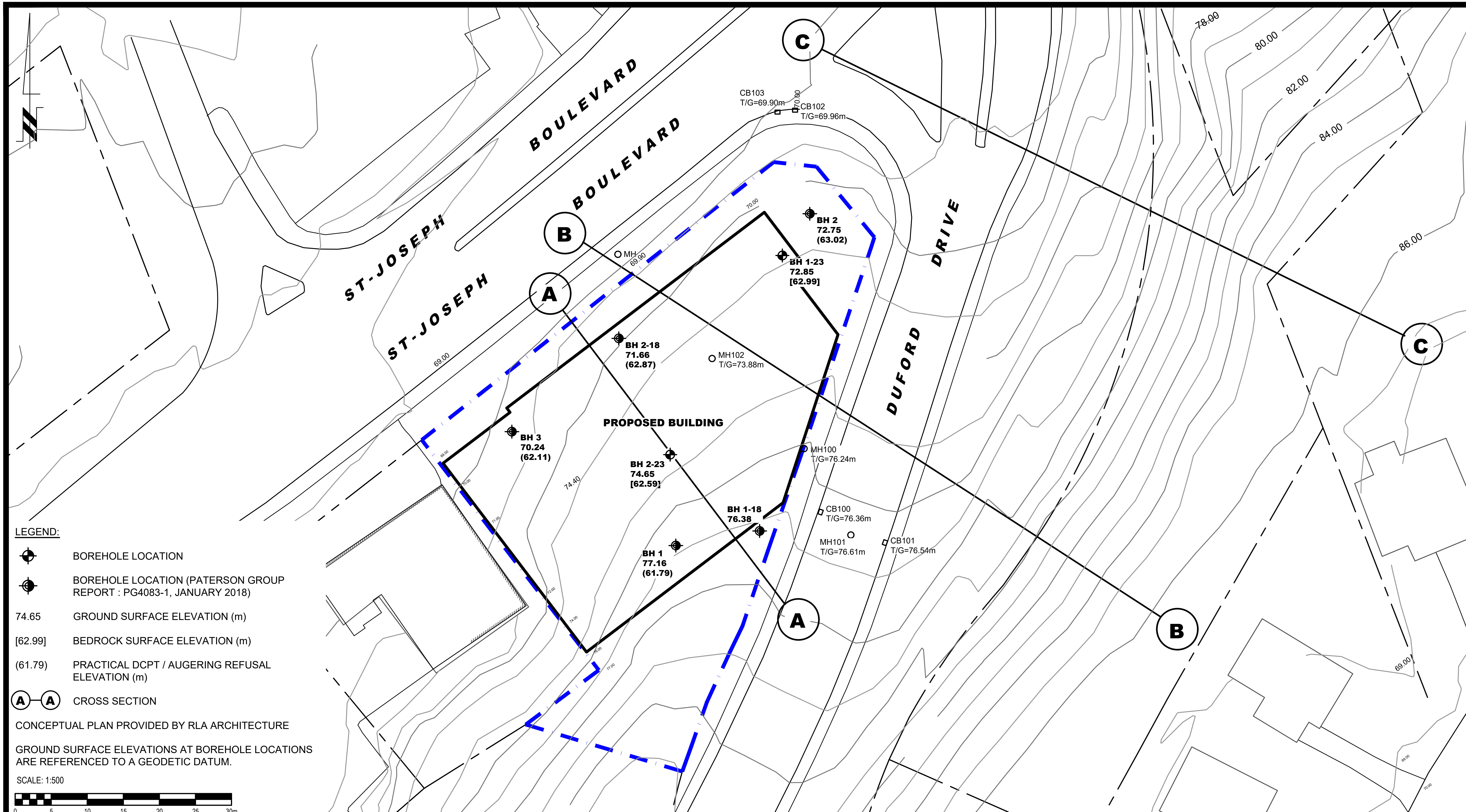


Figure 6A - Section C - Existing Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
W
1.203
Duford Drive
St. Joseph Boulevard
Silty clay crust
Unit Weight: 17 kN/m3
Cohesion: 8 kPa
Friction Angle: 33 degrees
Grey silty clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Engineered Fill (Pavement)
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
140
130
120
110
100
90
80
70
60
50
-20
-10
0
10
20
30
40
50
60
70
80
90
100


Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Figure 7A - Section C - Seismic Loading
1.597
0.16
W
Duford Drive
St. Joseph Boulevard
Silty clay crust
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 120 kPa
Grey silty clay
Strength Type: Undrained
Unit Weight: 16 kN/m3
Cohesion Type: Constant
Cohesion: 80 kPa
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
Engineered Fill (Pavement)
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
140
130
120
110
100
90
80
70
60
50
-20
-10
0
10
20
30
40
50
60
70
80
90
100



LEGEND:

- BOREHOLE LOCATION
- BOREHOLE LOCATION (PATERSON GROUP REPORT : PG4083-1, JANUARY 2018)

74.65 GROUND SURFACE ELEVATION (m)

[62.99] BEDROCK SURFACE ELEVATION (m)

(61.79) PRACTICAL DCPT / AUGERING REFUSAL ELEVATION (m)

A—A CROSS SECTION

CONCEPTUAL PLAN PROVIDED BY RLA ARCHITECTURE

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

SCALE: 1:500

0 5 10 15 20 25 30m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

VUZE CONSTRUCTION

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
222 DUFORD DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 05/2023
Drawn by: YA	Report No.: PG6609-1
Checked by: AB	Dwg. No.: PG6609-1
Approved by: JV	Revision No.:

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11x17