Landslide Hazard Assessment

Proposed Residential Development

Old Montreal Road
Ottawa, Ontario

Prepared for Taggart Investments

Report PG5201-2 Revision 3 dated October 18, 2024

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Appendices

1.0 Introduction

1.1 Purpose of Study and Scope of Work

Paterson Group (Paterson) was commissioned by Taggart Investments to conduct a landslide risk assessment for the proposed residential development considered Cardinal Creek Village South and located south and along Old Montreal Road, in the City of 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 risk 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 risk assessment 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 assessment 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. Further, Paterson compensated the subsurface information for the study area with a review of test hole information gathered for nearby sites in close proximity to the subject site which were investigated by Paterson as part of this assessment.

1.3 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of single and townhouse style residential dwellings with basement or slab-on-grade construction, attached garages, associated driveways, local roadways and landscaped areas. The construction of schools, a park, and a stormwater management pond are also included in the proposed development. It is further anticipated that the proposed development will be serviced by future municipal water, sanitary and storm services.

1.4 Review of Previous Geotechnical Investigations and Associated Studies

For this assessment, subsurface information was collected from a set of site-specific investigations and several previous investigations carried out by Paterson throughout the surrounding area of the subject site. The results of the previous investigations are presented in the following Paterson reports:

- ❑ Report prepared for Tamarack (Queen Street) Corp. - Geotechnical Investigation - Proposed Cardinal Creek Village Residential/Commercial Development - Old Montreal Road, Ottawa, Ontario - PG1796-4 dated September 19, 2014.

- ❑ Report prepared for Taggart Investments - Geotechnical Investigation - Proposed Residential Development Cardinal Creek Village South - Old Montreal Road, Ottawa, Ontario - PG5201-1 Revision 4 dated July 14, 2021.

❑ Memorandum report prepared for Taggart Investments – Slope Stability Assessment of Existing Slope Failure – 1320 Grand-Chene Court, Ottawa, Ontario - PG5201-MEMO.03 dated November 13, 2023.

Relevant test hole information and locations are presented on the Drawing PG5201-2 - Test Hole Location Plan in Appendix 2.

All reviewers of this report should understand that the geotechnical investigation undertaken in support of the proposed development has been undertaken in accordance with the City of Ottawa's *Geotechnical Investigation and Reporting Guidelines for Development Applications in the City of Ottawa*, that the slope stability fieldwork and analysis had been undertaken in accordance with the City of Ottawa's *Slope Stability Guidelines for Development Application in the City of Ottawa*, and that laboratory testing was undertaken in accordance with the above-noted guidelines and the *City of Ottawa's Tree Planting in Marine Clay Soils – 2017 Guidelines*.

It should also be noted that this report also considers the observations and findings provided by GEO Morphix in their report *Fluvial Geomorphological and Erosion Threshold Assessment, Tributary of Cardinal Creek, 1296 and 1400 Old Montreal Road* dated February 23, 2023. It is recommended that this report be read in conjunction with the project fluvial geomorphological report.

2.0 Background of Study Area

2.1 Field Investigation

Field Program

Paterson has undertaken several geotechnical investigations throughout the subject site. The initial portion of the geotechnical investigation for the overall development was carried out between January 22 and 26, 2009. At that time, seven (7) boreholes were advanced within the subject site to depths varying between 0.7 and 9.4 m below ground surface.

Supplemental investigations were completed in April and June 2012, and January and February 2013. At that time, thirty-two (32) additional boreholes were advanced to depths varying between 0.8 and 10.0 m below ground surface.

Additional geotechnical investigations were carried out in December 2012 and between February and March 2021 and consisted of excavating a total of fifty-four (54) test pits. The test holes were advanced within the subject site to depths between 0.7 and 6.5 m below the existing ground surface. A bedrock delineation program consisting of advancing probeholes to the bedrock surface was also carried out in November 2019 to assess the overburden thickness across the subject site.

The test hole locations were placed in a manner to provide general coverage taking into consideration site access, features and underground utilities. The test hole locations were determined by Paterson personnel and surveyed in the field by Paterson or Stantec Geomatics. It is understood that all test hole elevations are referred to a geodetic datum. The test hole locations for the investigations are presented on Drawing PG5201-2 - Test Hole Location Plan included in Appendix 2.

The boreholes and probeholes were completed using a track mounted drill rig operated by a two-person crew. The test pits were excavated using a rubber-tired backhoe or a hydraulic shovel.

All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the Geotechnical Division. The testing procedure consisted of augering or excavating to the required depths and at the selected locations sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon sampler, from the auger flights, or using a 73 mm diameter thin walled Shelby tubes in conjunction with a piston sampler. Grab samples were also collected along the excavated sidewalls of the test pits. All samples were visually inspected and initially classified on site. The auger, grab and split-spoon samples were placed in sealed plastic bags and the Shelby tubes were sealed at both ends on site and protected from disturbances over the entire process.

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

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

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

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

All boreholes were fitted with flexible polyethylene standpipes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

Geotechnical Laboratory Testing

The soil samples recovered from our field investigation were examined in our laboratory to collaborate the field findings. A fully sampled borehole (BH 89-13) was completed in June 2013 located within the north portion of the site. At the time, four (4) samples were submitted for unidimensional consolidation, five (5) samples were submitted for Atterberg limits testing and moisture content testing was completed on all recovered soil samples from BH 89-13.

Gradation and Atterberg limits testing were also completed on select samples obtained from the geotechnical investigations. The results of our testing are presented on Table 1, below, and on Grain Size Distribution and Hydrometer Testing and Atterberg Limit's Results sheets presented in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
TP 1-21	2.0	61	31	30	36.0	CH
TP 3-21	1.85	69	31	38	40.7	CH
TP 4-21	1.11	57	32	25	37.6	MH
TP 5-21	2.1	73	37	36	45	MH
TP 6-21	0.94	63	34	29	42.3	MH
TP 7-21	0.70	59	32	27	38.5	MH
TP 8-21	0.95	70	44	26	49.7	MH
TP 9-21	0.6	58	32	26	23.6	MH
TP 10-21	1.5	60	33	27	35.8	MH
TP 11-21	2.11	65	35	30	43.6	MH
TP 12-21	0.8	75	37	38	37.4	MH
TP 16-21	0.3	57	29	28	36.9	CH
TP 17-21	0.6	65	36	29	39.9	MH
TP 17-21	1.3	57	31	26	35.1	MH
TP 18-21	0.4	66	36	30	35.5	MH
TP 19-21	1.5	61	32	29	32.9	MH
TP 20-21	1.0	76	39	37	39.2	MH
BH 4B TW 1	4.19	61	28	33	70.8	CH
BH 57-12 SS 7	5.33	66	27	39	27.1	CH
BH 58-12 SS 6	4.57	63	23	40	23.2	CH
BH 67-13 SS 6	4.57	75	30	46	29.5	CH
BH 89-13 TW 2	4.97	79	30	48	70.7	CH
BH 89-13 TW 3	8.08	54	26	29	67.0	CH
BH 89-13 TW 4	12.65	46	26	20	70.0	CL
BH 89-13 TW 5	18.74	50	23	27	64.4	CL
BH 89-13 TW 6	24.20	43	20	23	n/a	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content; CH: Inorganic Clay of High Plasticity; CL: Inorganic Clay of Low Plasticity; MH: Inorganic Silts of High Plasticity

The results of the shrinkage limit test indicate a shrinkage limit of 22% and a shrinkage ratio of 1.71.

Grain size distribution (sieve and hydrometer analysis) was also completed on selected soil samples. The results of the grain size analysis are summarized in Table 2 and presented on the Grain-Size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
TP 2-21	G4	0.0	0.2	27.3	72.5
TP 7-21	G3	0.0	14.4	29.6	56.0
TP10-21	G4	0.0	14.4	31.2	67.5
TP12-21	G2	0.0	2.7	26.8	70.5
TP18-21	G1	0.0	15.4	34.1	50.5

2.2 Existing Conditions

Surface Conditions

The subject site consists mostly of undeveloped agricultural lands with several areas covered with trees and mature vegetation. The study area was observed to be intersected by a series of tributary ravines, which drain into Cardinal Creek. Piles of granular and crushed material have been observed on the west portion of the property and north of the main tributary ravine.

The ground surface across the subject site slopes downward gradually from east to west, and in general towards the tributary ravines. The slopes of the ravines were noted to be treed and stable based on our most recent site visit. Some signs of toe erosion were noted throughout where the watercourse is in close proximity to the valley corridor wall.

Due to the presence of the tributary ravines within the subject site, a slope stability assessment was carried out considering the slope conditions present in the subject site and along the sidewalls of the aforementioned watercourses. The results of the slope stability analysis are discussed further in Subsection 6.8 of this report.

Subsurface Conditions

Generally, the overburden profile consisted of topsoil, fill and/or asphaltic concrete underlain by a stiff to very stiff silty clay layer followed by a glacial till deposit. The fill was mostly encountered in the boreholes located next to Old Montreal Road.

Where encountered, the existing fill layer was observed to extend to ranges between 0.7 and 1.4 m in depth. The fill generally consisted of crushed stone followed by brown silty sand with clay, gravel, and cobbles.

The surficial layer of topsoil and/or fill was observed to be underlain by a silty clay deposit. The upper portion of the silty clay has been weathered to a brown desiccated crust. In situ shear vane field tests carried out within the silty clay crust yielded peak undisturbed shear strength values between 80 and 249 kPa. These values reflect a stiff to hard consistency in the silty clay crust.

Unweathered, grey silty clay was encountered below the brown silty clay crust. The silty clay deposit was observed to present a thickness in excess of 9 m at the west portion of the subject site and thinning out towards the east.

Glacial till was observed underlying the above-noted deposits at most locations at the subject site. The fine matrix of the glacial till generally consisted of silty clay with varying amounts of sand. Gravel, cobbles, and boulders were also present throughout the glacial till deposit and the tributary bed throughout the subject site.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the depth to bedrock across the site generally ranges from ground surface to 25 m. The depth to bedrock throughout the western portion of the tributary creek has been mapped to range between 15 to 25 m. Limestone of the Bobcaygeon formation is located throughout the majority of the subject site, with the exception of the western portion which is underlain by interbedded limestone and dolomite of the Gull River formation.

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 PG5201-1 - Bedrock Contour Plan in Appendix 2 for approximate bedrock contours based on refusal elevations.

Groundwater

Groundwater levels were recorded at each borehole instrumented with a piezometer and as noted upon completion of the test pits. The long-term groundwater can be estimated based on these observations, recovered soil samples' moisture levels, and observed colouring and consistency of the recovered samples. Based on these observations, the long-term groundwater level is anticipated at a depth of approximately **3 to 4 m** below ground surface.

The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1. It should be noted that groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

3.0 Slope Stability Analysis

3.1 Slope Conditions

The existing slope conditions were reviewed by Paterson field personnel throughout multiple site visits. The initial visit was completed on April 18, 2012, to document the conditions of the tributaries to Cardinal Creek (south tributary, Mid Branch 1 and Mid Branch 2). A second site visit was completed on September 10, 2022. At the time, it was concluded that the slopes along the tributaries to Cardinal Creek were observed to be stable. However, some toe erosion was noted throughout the watercourse, close to the valley corridor wall.

A third site visit was completed on July 12, 2023, to compare the current conditions with previous observations made at the subject site. Photos comparing the previous and current conditions have been included in Appendix 2.

The field review completed during the aforementioned visits generally consisted of observing surface conditions along the length of the tributaries, including identifying the presence of vegetation, erosion and other features associated with slope stability. Paterson field personnel verified subsurface information and in-situ shear strength of cohesive soils at select slope sections using a hand-auger and field vane apparatus, respectively, to compare them to the findings from our borehole and test pit test hole observations.

Water levels and flow within the watercourses were generally observed, including identifying signs of recent high-water marks or other signs of previous rises in the water levels.

Overall, a total of twenty-three (23) slope cross sections throughout the above-noted locations were analyzed as part of the slope stability analysis. Topographic surface elevation measured at the selected locations and LiDAR information were used to complete the slope stability analysis.

Based on the results of our field observations and slope stability analysis, a Limit of Hazard Lands was assigned from the top of slope for the above-noted sections of the study areas. The cross-section locations and topographic mapping information are presented on Drawing PG5201-1 - Test Hole Location Plan in Appendix 2.

3.2 Summary of Field Observations

The following section is a summary of our observations during the time of our field review of the subject slopes.

The subject site is intersected by a number of watercourses including the south tributary ravine that drains into Cardinal Creek and two branches opening from the aforementioned tributary designated in this report, from south to north, as Mid Branch 1 and Mid Branch 2. The south tributary was observed to flow in east to west direction throughout the central portion of the subject site and the two branches were observed to flow from a southeast to a northwest direction.

The general slope of the bank was observed to range between 3 to 15 m high and appeared to have a profile generally shaped 5H:1V with local sections with approximate steepness of up to 1H:1V. The watercourses were observed to be up to 6 m wide, with seasonal variations in water flow.

The majority of the slope appeared to consist of stiff, brown silty clay, which was underlain by firm, grey silty clay in close proximity to the water level. Some erosion of the toe of slope had been observed where the watercourse is located in close proximity to the valley corridor wall. This generally consisted of some erosion of the bank face resulting in some undercutting along the channels edge at some locations. Occasional shallow and low slip surfaces restricted to close proximity to the water level were observed in areas with sharp bends in the channel alignment. Overall, vegetation was observed to be intact and mature across the majority of the tributary valley. The bed of the water course generally observed to consist of glacial till and transition to stiff, grey silty clay further downstream along its footprint.

Reference should be made to Drawing PG5201-2 – Test Hole Location Plan in Appendix 2 which depicts the above-noted tributaries and the associated slope stability cross-sections and setback information.

3.3 Slope Stability Analysis

A slope stability assessment has been conducted to determine the applicable geotechnical Limit of Hazard Lands setback along the north bank of the main tributary to Cardinal Creek within the subject site. The analysis of the stability of the slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those 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 subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

A total of twenty-three (23) slope cross-sections were analyzed under static and seismic conditions. The cross-sections for existing and proposed conditions were analyzed utilizing the latest topographic mapping and proposed grading, respectively, and assuming the worst-case-scenario by assigning cohesive soils under fully saturated conditions.

Subsoil conditions at the sections were determined 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 soil parameters were determined for the slope soils based on subsoil conditions at the boreholes along the top of slope. The analysis was carried out in accordance with the City of Ottawa's standard guidelines prepared by Golder Associates titled Slope Stability Guidelines for Development Applications in the City of Ottawa, dated 2004.

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 3 on the following page.

Table 3 - Effective Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay Crust	17	36	9
Grey Silty Clay	16	36	12
Glacial Till	20	33	1
Bedrock	Impenetrable		

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the test holes completed at the time of our geotechnical investigation and based on our general knowledge of the area's geology. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 4 below.

Table 4 - Total Stress Soil and Material Parameters (Seismic Analysis)			
Soil Layer	**Unit Weight (kN/m^3)**	**Friction Angle (degrees)**	**Undrained Shear Strength (kPa)**
Brown Silty Clay Crust	17	-	100
Grey Silty Clay	16	-	80
Glacial Till	20	33	N/A
Bedrock	Impenetrable		

Static Conditions Analysis - Existing Conditions

The results for the existing static slope conditions at the slope stability sections are presented in Appendix 2. The slope stability factors of safety were found to be greater than 1.5 at all sections analyzed, except for Sections F and JJ, which require a 4.7 and 17 m setback, respectively from top of slope to obtain a factor or safety greater than 1.5.

Static Conditions Analysis - Proposed Conditions

The results for the analysis of the stability of the slope under proposed conditions are presented in Appendix 2. The slope stability factors of safety were found to be greater than 1.5 at all sections analyzed.

Seismic Loading Analysis - Existing Conditions

An analysis considering seismic loading was also completed as part of our slope stability assessment. A horizontal seismic acceleration, K_h, of 0.21g was considered for the analyzed section and discussed further in Section 4.2.5 of this report.

This acceleration is considered to be higher than half of the peak (horizontal) ground acceleration (PGA) of 0.312g (and near the regional PGA of 0.32g), specified in the National Building Code of Canada (NBCC 2015) Seismic calculator for the subject site. The above-noted specified PGA is considered to have a probability of exceedance of 2% in 50 years (i.e.,1:2,475 years) for the subject site. Based on a trendline considering the PGA values assigned for different probabilities of exceedance, a PGA equal to 0.21g may be approximately estimated to be equivalent to a probability of exceedance of 4% in 50 years (i.e., 1:1,250 years).

A factor of safety of 1.1 is considered to be satisfactory for stability analysis including seismic loading (i.e., pseudo-static) as per the City of Ottawa's Slope Stability Guidelines for Development Applications. The results of the analysis including seismic loading fully saturated conditions (worst-case-scenario) are shown in Appendix 2. The overall slope stability factor of safety at all slope cross-sections when considering seismic loading was found to be greater than 1.1 which is considered to be stable under seismic loading.

Seismic Loading Analysis - Proposed Conditions

An analysis considering seismic loading was completed for the proposed conditions as part of the slope assessment. A horizontal seismic acceleration, K_h, of 0.16g was considered for the slopes. This acceleration is considered as half of the peak (horizontal) ground acceleration (PGA) of 0.32g, specified in the NBCC 2015. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the analysis including seismic loading fully saturated conditions (worst-case-scenario) are shown in Appendix 2. The overall slope stability factor of safety at all slope cross-sections when considering seismic loading was found to be greater than 1.1 which is considered to be stable under seismic loading.

Limit of Hazard Lands

Based on our review, the slopes reviewed as part of this assessment are considered stable from a geotechnical perspective. Since the slopes are in close proximity to an active watercourse, erosion of the toe of slope is considered to be a notable factor in assessing slope stability. The banks abutting the watercourse were observed to be affected by minor signs of erosion. Signs of active erosion were mostly noted in the lower portion of the slopes and consisted of occasional small patches of loss vegetation or exposed root systems along the face of slope.

Generally, subsurface conditions at the toe of slope varied between in-situ, stiff, brown to grey silty clay. Based on the cohesive nature of the soils, the observed current erosional activities and the width and location of the current watercourse, the toe erosion allowance for the valley corridor slopes was determined.

Based on the above-note observations, and in accordance with the City of Ottawa's *Slope Stability Guidelines for Development Applications* (2004) and the Ministry of Natural Resource's *Technical Guide – River and Stream Systems: Erosion Hazard Limit* (2002), it is considered that a toe erosion allowance of 5 m is appropriate for the corridor walls confining the subject tributaries. The toe erosion allowance should be applied from the top of stable slope.

If portions of the slope were to be improved by the use of erosion protection methods, those portions would not be subject to the aforementioned toe erosion allowance as the toe of slope would no longer be susceptible to erosion that would impact the stability of the overlying slopes.

A stable slope allowance in accordance with the requirements outlined for Section F and Section JJ should also be taken from the top of slope, as required.

The limit of hazard lands, including a 6 m erosion access allowance, stable slope allowance (where required) and a 5 m toe erosion allowance, is presented on Drawing PG5201-2 - Test Hole Location Plan in Appendix 2.

3.4 Seismic Design Considerations

Based on the results of the geotechnical investigation, a seismic **Site Class D** 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:

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

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 Overburden and Clay Sensitivity

Based on the findings of the geotechnical investigation, the slope profiles throughout the subject site consist primarily of a silty clay deposit inferred to be underlain by bedrock. Based on geological mapping undertaken by the Geological Survey of Canada under Open File 5311, the local deposit is considered to be formed by offshore marine sediments in the form of preserved erosional terraces.

The clay deposit encountered throughout the subject site was observed to consist of a very stiff, weathered, brown clay crust extending to depths between 0.9 and 5.9 m below the ground surface. Shallower (i.e., less than 2.0 m in depth) deposits of clay were typically observed to be underlain by compact to dense deposits of glacial till, and not by unweathered, grey silty clay. Sand, with the exception of imported or re-worked site-generated soil fill material, was not encountered above the clay deposit to form a “sand cap” layer as has been documented throughout the Ottawa Valley.

Average undrained shear strengths for the upper and lower weathered and unweathered portions of the clay deposits were estimated to be approximately 210 and 60 kPa, respectively. The remoulded shear strength for the lower unweathered grey clay deposit ranged between 4 (BH 44-12) and 10 kPa. The clay deposits sensitivity (i.e., ratio between undrained and remoulded shear strength) was estimated to range between 4 and 16.

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. 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. Based on this, retrogressive landslides throughout the Ottawa region have historically occurred within clay soils, such as those encountered throughout the subject site.

It has also been documented that the retrogression of landslides might be predicted by the undrained and remoulded shear strength values measured in the silty clay unit. Mitchell & Markell (1974) studied the characteristics of landslides in silty clay soils associated with river valleys. Their study, based on 41 documented landslides located within Eastern and Northern Ontario, indicated that Taylor’s stability number can be used as an indicator to evaluate the susceptibility of landslides to occur. Taylor’s stability number (Ns) is defined as:

$$N_s = yH/S_u,$$

where y represents the bulk unit weight of soil (kN/m^3), H is the slope height (m), and S_u depicts the peak undrained shear strength of the silty clay (kPa). Mitchell & Markell (1974) determined that N_s should be greater or equal to 6 for the potential of retrogression to occur. Based on the in-situ field investigation testing information, the data suggests there is potential for retrogression of a slope failure throughout the subject site if a slope failure was triggered throughout the clay overburden.

It should be noted that remoulded shear strength was not measured to be less than 5 kPa, which is higher than the 1 kPa threshold commonly considered for identifying landslide susceptibility. While plasticity testing has not been complete throughout deeper portions of the clay deposit, there is sufficient information to generally characterize the underlying deposit as "sensitive" given the estimated sensitivity ratios and high Ns factors. The presence of sensitive clay throughout the subject size varies due to the current drift thickness and variable subsoil conditions between the western and eastern portions of the subject site. This is discussed further in subsequent sections of this report.

Based on the above, due to the presence of sensitive glaciomarine clay throughout the majority of the subject site (and along the subject tributary), there is potential for retrogression to occur should a slope failure be triggered. <u>Based on this, the baseline probability discussed in Section 4.3 - Hazard Assessment will be multiplied by a factor of 3.0</u>.

4.2.2 Slope Inclination, 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 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 below.

Topographic relief was interpreted using DEM provided by Google Earth. Relief was considered as the difference between the lowest and highest elevations and considering distances extending beyond a landslide footprint. Greater distances were considered where a landslide formed into a slope profile. Significantly large landslides could not be reasonably evaluated 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.

Table 5 – Summary of Drift Thickness Throughout Historic Landslide Footprints

Drift Thickness	Number of Incidences	%
0 to 1	0	0.0
1 to 2	0	0.0
2 to 3	0	0.0
3 to 5	0	0.0
5 to 10	8	7.0
10 to 15	7	6.1
15 to 25	34	29.8
25 to 50	49	43.0
50 to 100	16	14.0
Total Landslides Within Study Area	114	94.2
Total Landslides Documented by Open Files	121	

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.

In summary, more frequent incidences of landslides occur in areas with more than 15 m of overburden and 10 m of topographic relief throughout the study area. Based on the current test hole coverage and slope stability sections, it is anticipated that more than 15 m of overburden may be present west of Slope Stability Cross Section I and north of Cross Section NN.

Further, up to 20 m of relief may be observed between the western boundary of the subject site and Slope Stability Cross Section OO. Between 12 to 14 m of relief may be observed between Slope Stability Cross Section O and NN. Less than 10 m of relief may be observed throughout the remainder of the subject site, including the furthest extension of the tributaries mid-branch extending along the southwestern portion of the subject site (area of Slope Sections L and M).

Based on the above, the potential for a landslide as based on the above-noted factors is discussed in further detail in *Section 4.3 – Hazard Assessment* of this report. The baseline probability will be modified for each applicable combination of overburden thickness and topographic relief understood to be located throughout the subject site in that portion of the report.

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	

Slope inclination and shape are also factors associated with assessing landslide susceptibility and overall slope stability. While the majority of the slope analyzed as part of our slope stability assessment yielded factors of safety exceeded local requirements for development purposes, slope geometry throughout the subject site is indicative of terrain that has potential for instability.

Based on our review of LiDAR and topographic mapping, slopes are generally rectilinear and locally concave in shape, and range in inclination between approximately 5H:1V (11 degrees) and nearly 1H:1V (45 degrees). Based on this, it is suggested that the baseline probability discussed in Section 4.3 – Hazard Assessment will be multiplied by a factor of 1.5 to account for slope steepness and shape.

4.2.3 Groundwater, Surface Drainage and Toe Erosion

Groundwater

Groundwater is understood to be a factor contributing to landslide susceptibility. Landslides throughout the 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.

Fully saturated slope conditions have been considered as part of our slope stability assessment in Section 3.1 of this report. 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 at all sections analyzed with the exception of Sections F and JJ. An appropriate stable slope allowance has been incorporated as part of the Limit of Hazard Lands line depicted on Drawing PG5201-2 – Test Hole Location plan in Appendix 2 of this report.

While fully saturated conditions were considered as part of the slope stability analysis, this condition is considered to be a conservative estimate of the slope's stability given that it is interpreted the groundwater table is located at the interface between the weathered and unweathered clay layers for the clay deposit. It is expected a long-term local groundwater table dewatering of up to approximately 0.5 m will take place throughout the subject site as it becomes developed.

This localized long-term dewatering is anticipated as a result of the installation of buried services and a reduction in the amount of permeable surfaces through which surface water may infiltrate as a result of the proposed development. This local dewatering has been considered in our geotechnical report.

Based on this, given that groundwater conditions in the post-development condition are anticipated to be lower than in the pre-development condition, and that pre-development groundwater conditions did not impact the stability of the slopes form a geotechnical perspective, groundwater will not be considered as a factor in modifying the baseline probability discussed in Section 4.3 (i.e., factor of 1.0).

Surface Drainage

Surface drainage, or sheet drainage from the table lands towards the watercourse, can impact the stability of the subject slopes. Currently, in the pre-development condition, surface water generated from rain and snowmelt is handled by either ingress into the subsoils, or sheet drainage following local topography. The majority of the subject site is underlain by silty clay, which is a generally impervious material that does not permit high levels of ingress. Based on this, it is anticipated that the majority of the surface water is handled by sheet drainage, and likely across the table lands, valley corridor and into the watercourse.

In the post-development condition, the majority of the subject site is anticipated to be urbanized by buildings, roads and landscaped areas. The exception to this would be the area encapsulated by the Limit of Hazard Lands designation setback. This area forms an area where development would not be permitted given the nature of the slopes and results of analysis considering erosion potential, stability and temporary access for maintenance. The portion of the site beyond the Limit of Hazard Lands will be graded to promote drainage towards sewer infrastructure and infrastructure would be sized to attenuate peak runoff volumes that are currently handled by the subject site.

Based on this, sheet drainage between urban lots backing onto the Limit of Hazard Lands is expected to result in on-going sheet drainage between the urban area of the subject site and the valley corridor forming the creek. Since the majority of the subject site will be developed and sheet drainage would be primarily handled by the sewer infrastructure, the amount of sheet drainage the slope would handle in the post-development condition would diminish during peak events and normal conditions. However, since surface drainage will persist in the post-development due to the undeveloped nature of the area encapsulated by the Limit of Hazard Lands surrounding the tributary, the baseline probability discussed in Section 4.3 – Hazard Assessment will be multiplied by a factor of 1.5 to account for continued surface drainage in the post-development condition.

Toe Erosion

Landslides throughout the Ottawa Valley have been documented to occur most frequently adjacent to a watercourse. The formation of valley corridors by watercourses results in erosion along the toe of the slope and subsequent downcutting of the bank face by the erosional force of the watercourse. Sufficient downcutting, oversteepening and erosion of the slope can result in the instability of a slope and the potential for a landslide if a slope failure is triggered.

There is a relationship between stream flow (via flow accumulation) and landslide incidence such that larger landslides tend to be associated with larger watercourses (Quinn et al., 2010). In addition, the stream flow of a watercourse can be directly correlated to its stream order. Stream order is the degree of a tributary and branch streams with respect to an artery stream. Larger stream order values indicate the degree of closeness a stream is linked to the principal stream, whereas smaller values indicate the streams are considered to be distant tributaries from an artery stream. In summary, higher values of stream flow are correlated to higher degrees of stream order which are further correlated to older and fully developed watercourses. Smaller values of stream order are correlated to younger and less developed watercourses.

Generally, landslide density throughout the study area undertaken throughout NTS 31H was very low for streams up to order 3 and greater than or equal to order 9 (Quinn, 2009). The findings are similar for flow accumulation such that streams with less flow or smaller stream orders have a negative correlation with landslide incidence (Quinn, 2013). There is some evidence presented by a study area in Norway that younger streams have not fully developed their watercourse morphology and may be more erodible than larger, mature streams. However, the methodology undertaken to assess this for the study area of NTS 31H could not confirm this relationship for local and regional conditions at that time (Quinn, 2013).

Stream sinuosity was also explored as a variable impacting slope stability. Stream sinuosity is defined as the ratio of the total length along a stream segment to the shortest length between its endpoints (Quinn, 2013). Based on the review for the area of NTS 31H, it has been observed that landslides tend to be infrequent along streams with sinuosity lower than 1.338. Furthermore, channels with wider and more tightly spaced meander belts experience higher rates of erosion and are therefore more susceptible to landslides. Preferential occurrence of landslides in slopes situated on the outside of meander belts rather than in streams with low levels of sinuosity was similarly observed by Hugenholtz (2004).

A geomorphic study was undertaken by Geomorphic Solutions in 2007 for the Cardinal Creek watershed. The study considers the east-west running watercourse and its tributaries as part of this study. An excerpt of the study area and watershed footprint are depicted on Figure 1 below for reference:

Tributaries TRH1, TRH2, TRH3, TRH4 and TRH5 are located throughout the subject site and are considered the areas of interest with regards to landslide susceptibility. TRH1, TRH2 and TRH4 would be considered the "south tributary", "mid-branch 1" and "mid-branch 2" portions of the tributary discussed throughout this report, respectively. The study provided the following information with respect to the stream order and sinuosity for these reaches of the tributaries:

Table 7 – Summary of Geomorphic Assessment for Tributary H			
Reach	**Stream Order**	**Sinuosity**	**Length (m)**
TRH1	1	1.02	939
TRH2	1	1.02	790
TRH3	2	1.01	827
TRH4	1	1.00	361
TRH5	1	1.03	1285
Note: Results provided by Geomorphic Solutions for Geomorphic Assessment of Cardinal Creek Subwatershed (2007)			

The stream order and sinuosity of the south tributary and the mid-branches located throughout the subject site are considered less than values that would indicate an increase to landslide susceptibility.

However, there are notable signs of active erosion throughout the valley corridor during recent site visits by Paterson and GEO-Morphix. This includes active formation of localized slumps along the bank face, over-steepening of the bank and inability for vegetation to establish to reinforce against the active watercourse environment. Detailed observations pertaining to the active erosion and associated bank profile have been discussed in our Geotechnical Report and as noted in the Photographs from Site Visits in Appendix 1 of this report.

It should be understood that previous field reviews have included cursory field review of subsurface conditions by the use of visual observations of apparent subsurface conditions (i.e., areas where bare clay was present in valley floors, bank faces and areas of erosion) and by occasional hand-augers to confirm valley conditions. In general, the conditions observed throughout previous field visits is consistent with the information depicted on our slope stability cross-sections considered based on the subsurface information attained from previous rounds of test holes investigations.

Based on our review of recent and current LiDAR mapping (i.e., compare between 2006 and 2020 LiDAR), small, localized slumps continue to form throughout the valley in response to active erosion of the bank face. In the post-development condition flow conditions throughout the tributary are expected to slightly increase, however, peak flows are anticipated to be restricted and attenuated to minimize potential downstream impacts. Further, natural factors, such as beaver dams, which have formed downstream of the subject site, can result in unforeseen artificial raises in the creek levels and temporary saturation and subsequent draw-down along the bank face. Therefore, it is expected toe erosion will remain a notable factor impacting landslide susceptibility in the post-development condition.

It is expected the outfall area for the proposed stormwater management facility (SWMF) located at the southwestern portion of the northern side of the tributary will be provided with erosion protection for the channel that will be constructed between the headwall and the creek. The erosion protection, which will be advised by Paterson and the project geomorphologist, will be planned to provide sufficiently hydraulically sized to provide protection to the underlying soils from active erosion during peak discharge events, as well as to be provided such that long-term maintenance of the drainage channel would be minimal and be able to remain in place for the service life of the pond.

Since there are notable signs of erosion during each of our site visit, active erosion continues to result in the development of localized slumps, and toe erosion is one of the most common triggers for local slope failures and previous historical landslides, <u>the baseline probability will be multiplied by a factor of 2.0</u> to consider future toe erosion. This is discussed in further detail in *Section 4.3 – Hazard Assessment* of this report.

4.2.4 Proximity to Landslides

Landslide inventory mapping published by GSC indicates the presence of potentially up to 4 landslides in proximity to the subject site. 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 the likelihood of the nearest adjacent landslides being within a specified distance ranging between less than 50 and 2,000 m being between 49.2 and 96.7% (Quinn et al., 2011).

This pattern explains that future landslides are more likely in areas that have experienced previous landslides than in areas where no past landslides exist. 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.

It is understood up to potentially five landslides have been documented within 2 km of the subject site. Two of these landslides, Oln17 and Oln18, intersect the southwestern boundary of the subject site. Both Oln17 and Oln18 have been reported by GSC to have retrogressed into their respective sides of the incised valley of a tributary of Cardinal Creek (GSC OF8600, 2019), and are depicted on Figure 2 for reference.

Oln15 and Oln16 are located within 500 m of the westernmost boundary of the subject site. Oln15 is considered a "probable landslide" which may have retrogressed into the scarp slope above a terrace surface of the proto-Ottawa River (OF8600, 2019). Cmb1 is located 2 km from the northeastern boundary of the subject site and retrogressed into a scarp slope along the south side of the Ottawa River.

The area of Oln15 and Cmb1 was measured to have approximately 19 to 21 m of topographic relief and a relatively steep (i.e., over 20 degrees) slope along their flank. The areas adjacent to Oln17 and Oln18 were measured to have up to approximately 11 degrees of slope inclination and up to 9 m of relief along their footprints.

Oln16 retrogressed into the western side of the incised valley of Cardinal Creek and has been heavily altered by urban development (OF8600). The area of Oln16 experiences approximately 14 to 16 m of topographic relief and is incised by a creek identified as having a stream order of 4 and sinuosity of 1.39 (Geomorphic Solutions, 2007). Drift thickness throughout the area of Oln15 and Oln16 range between 25 to 50 m. Oln17 and Oln18 has been documented by GSC OF5311 as having a drift thickness ranging between 15 to 25 m.

Based on our review of subsoil information for the area around Oln17 and Oln18, it is not anticipated subsoils conditions throughout those scars is representative of conditions throughout the entirety of the subject site. Based on our current test hole information and associated mapping, drift thickness is higher throughout these scars than throughout the majority of the subject site.

The higher drift thickness is typically characteristic of areas with deeper deposits of sensitive clay, greater depth to bedrock and glacial till (i.e., formations that are not as sensitive to erosion and landslide susceptibility) and greater topographic relief. These conditions are generally only similar for the portion of the subject site located throughout and west of the proposed SWMF and throughout the northern portion of the subject tributary.

While topographic relief may be similar throughout the southern half of the tributary, drift thickness is measured to be decreasing towards the south of the tributary, and consequently results in shallower overburden along the southern half of the tributary. Based on that, surface and subsoil conditions similarities throughout the subject site and areas of previous failures are anticipated to be concentrated throughout the south-western corner of the northern half of the tributary within the subject site.

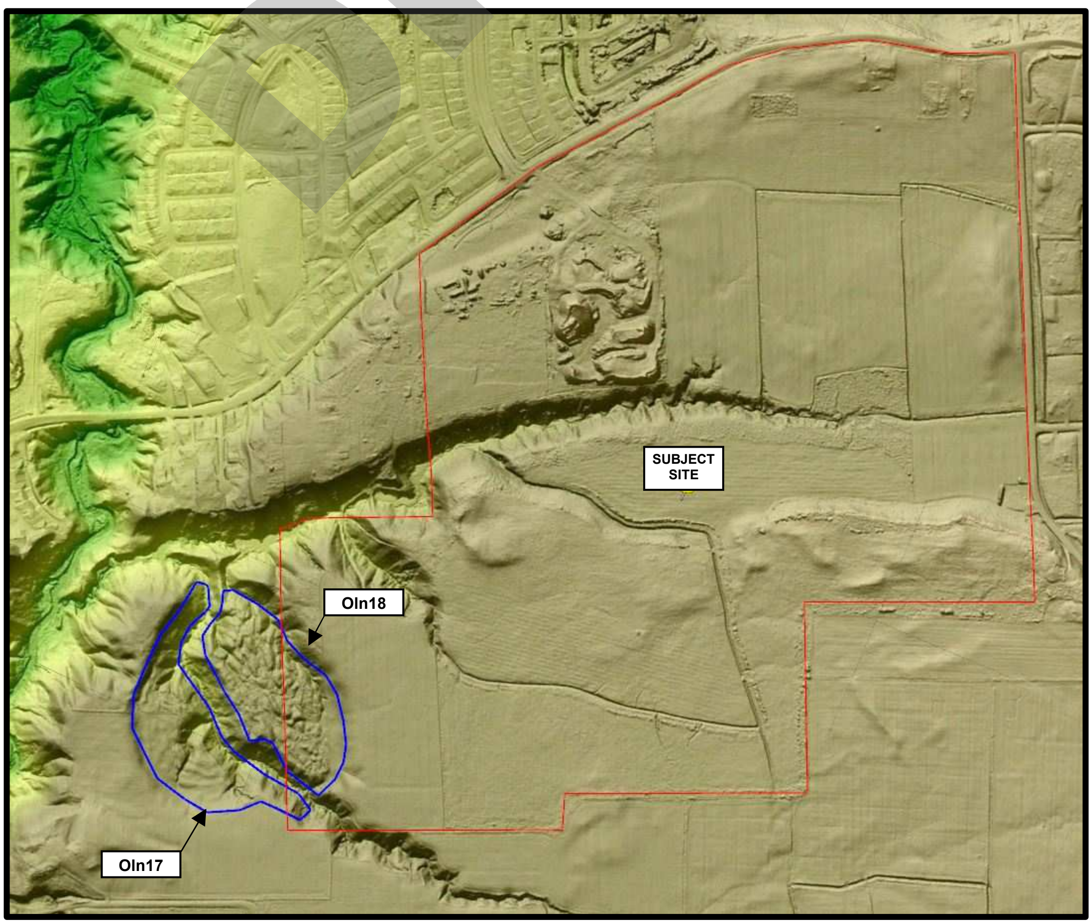

Figure 2 – LiDAR Image of Subject Site and closest landslide and slope failure.

Further to the above-noted large-scale landslides that have occurred within the vicinity of the subject site, recent small-scale and localized slumps have formed in recent years in response to active erosion and sheet drainage. Given the high concentration of local small- and large-scale landslides throughout the area and within the subject site, it is considered appropriate to increase the baseline probability for landslides to occur throughout the subject site by a factor of 2 in *Section 4.3 – Hazard Assessment* of this report.

4.2.5 Earthquakes and Seismic Hazards

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 0.21g 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 1:1,250-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 exceed 5 kPa within the subject site and are therefore above the threshold associated with landslide susceptibility. 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. Further, local hazard peak-ground acceleration values for the subject site for a 2% exceedance is considered to be 0.316, which is less than the regionally accepted value of 0.32. This suggests that the area of the subject site has a marginally reduced seismicity than would be considered for the Ottawa region in general.

Given the above, earthquake loading is not anticipated to be a significant impact on local landslide susceptibility and will be considered a slightly notable factor in the calculation of the baseline probability (<u>i.e., multiplied by a factor of 1.1</u>).

4.2.6 Sources of Anthropogenic/Construction Vibrations

It is anticipated that the underlying clay deposit will experience vibrations from several sources during the construction phase of the proposed development. Since the currently proposed development is anticipated to consist of a residential development, foundations for the proposed low- to mid-rise structures are anticipated to consist of conventional spread footing foundations. These types of foundations will be able to be constructed using conventional construction methods and will not require the use of equipment that may cause larger vibrations, such as pile drivers and caissons.

It is expected some higher amounts of vibrations may result from site servicing works that would be located below the bedrock surface throughout portions of the subject site where the bedrock formation is within approximately 2 to 3 m of the existing ground surface. In these areas, Once the excavation reaches the bedrock surface, bedrock removal would be required using a combination of line-drilling, hoe-ramming and blasting. In the City of Ottawa, vibrations resulting from blasting measured at nearby structures is limited to a peak particle velocity of 25 mm/s. Attenuating and limiting vibration potential in accordance with local guidelines is expected to provide limitations against the potential for blasting to generate significantly high magnitudes of vibration that could trigger localized slope failures.

Other sources of vibration throughout the construction phase would be much lower than those associated with blasting and surficial. These would be generated from heavy-truck traffic, operation of heavy-machinery, displacement of material and compaction of fill by vibratory compactors. It is expected the vibrations associated with these efforts would be dampened by the subsoils such that they would not result in a meaningful impact to the stability of slopes along the tributary.

The above-noted peak particle velocity ranges suggest that the vibrations that would be experienced by the clay deposit during the construction program would be comparable to those considered for a seismic hazard with an annual probability of 1% of exceedance. Based on this, the vibrations associated with the construction program would be comparable to an earthquake yielding a horizontal peak ground velocity of 0.025g, and far less than those considered in our slope stability analysis for seismic conditions (considered PGA of 0.21g, over 8 times higher).

There is potential for earthworks to require the use of land located in close proximity to the edge of the table lands, and as has been observed in recent years and as is observable on aerial images taken of the subject site. These types of works are anticipated to consist of occasional and temporary stockpiling of soil fill and other materials. This type of work will induce stress onto the underlying slope if stockpiling occurs in close proximity to the table lands and would be periodically monitored and advised upon by Paterson during the construction phase to minimize impacts onto the underlying slopes.

The location of the future SWMF is located in an area that is considered favorable with regards to reducing landslide susceptibility throughout the subject site. The SWMF will provide a significant stress reduction to the underlying soils supporting its footprint since it would consist of removing approximately 6 m of soil across the pond's footprint. The area of the proposed SWMF is in an area where drift thickness is in the higher range (i.e., 12 to 15 m) and would result in a condition that is less comparable than in the area of previous failures.

Based on our review of slope stability analysis results considering finished grades and the post-development condition (also considering fully saturated conditions), the post-development condition is not anticipated to result in affecting the stability of the subject slopes or increasing susceptibility for landslides to occur throughout the subject site. Recommendations will be provided in the geotechnical report to limit the types of features rear yards backing onto the table lands may consider (i.e., limiting grade raises, above-ground structures such as pools and decks, drainage features, etc.) and to mitigate enabling conditions that would affect the stability of the subject slopes.

Based on the above-noted discussion, while the construction phase is not anticipated to be a major contributor to landslide susceptibility given the above-noted discussion, the baseline probability will be adjusted by a factor of 1.5 to consider unknown potential effects the construction program could impose onto the slopes along the tributary during that phase of the development.

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 3 below.

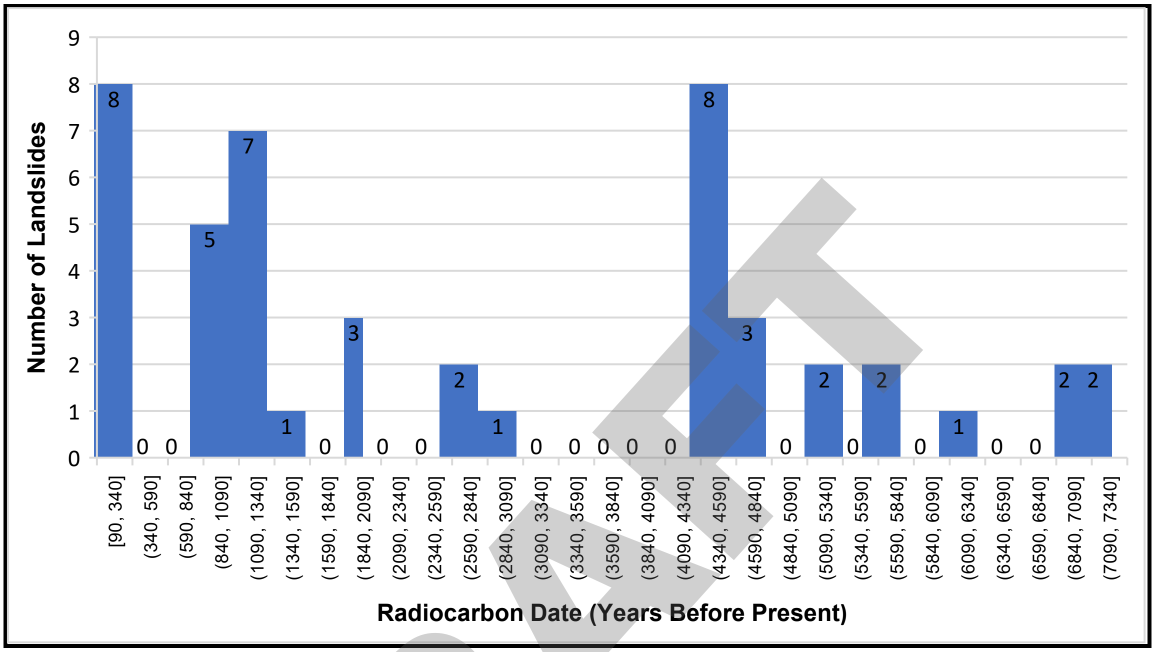

Figure 3 - Summary of Carbon-dated Landslides (GSC OF7432)

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 75-80,000 km^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 2,500 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 and annum is calculated over the study area defined by the GSC files. Considering the area of the subject site (approximately 1.2 km^2), this baseline probability may be reduced to <u>a site-specific baseline probability of 5.47x10^{-4} landslide per year</u>. The baseline estimate would be then adjusted based on our assessment of site-specific factors that are known to have resulted in large, catastrophic landslides.

Based on our review, surface relief and drift thickness are considered notable factors affecting landslide susceptibility, as discussed in Subsection 4.2.2 of this report. Hugenholtz (2004) observed in their review of Green's Creek that the concentration of landslides to re-occur throughout concentrated areas along the creek alignment. They had observed as many as 52 landslides of varying sizes and classes over the period of 1928 and 2001 (73 years) using digital photogrammetric techniques. The conditions observed throughout Green's Creek are considered comparable to the area of Cardinal Creek nearby the subject site, and an example of the potential for more frequent smaller-scale landslides beyond the larger-scale retrogressive slides that are more readily identified. Given this, a return period of one tenth of the large landslide return-period (i.e., 5.4-year return period) is considered appropriate for the relief and drift thickness variables throughout the subject site.

Based on our review of site-specific factors identified throughout this report, additional factors have been considered for adjusting the baseline probability to provide a site-specific landslide probability. Table 8 presents a summary of the above-noted Ottawa-wide probability being reduced to a site-specific probability, omitting the factors associated with drift thickness and topographic relief (which are summarized in the preceding paragraphs).

Table 8 – Summary of Site-Specific Baseline Probability Modification Factors	
Baseline Probability for Landslide to Occur Throughout Subject Site	$5.5\text{x}10^{-4}$
Section 4.2.1. – Overburden and Clay Sensitivity	3.0
Section 4.2.2. – Inclination, Bedrock Depth and Surface Relief	
Inclination	1.5
Bedrock Depth	Estimated Subsequent Tables
Surface Relief	
Section 4.2.3. – Groundwater, Surface Drainage and Toe Erosion	
Groundwater	1.0
Surface Drainage	1.5
Toe Erosion	2.0
Section 4.2.4. – Proximity to Landslides	2.0
Section 4.2.5. – Earthquakes and Seismic Hazard	1.1
Section 4.2.6. – Sources of Anthropogenic/Construction Vibrations	1.5
Modified Site-Specific Baseline Probability (Not Considering Drift Thickness and Surface Relief Factors)	**$2.4\text{x}10^{-2}$**

TABLE 8 WAS UPDATED TO INDICATE THE FACTORS DISCUSSED IN THE SECTIONS ABOVE

The probabilities for landslides to occur throughout the subject site considering drift thickness (Table 9) and surface relief (Table 10) are estimated accordingly in Table 11 and calculated by multiplying the values tabulated in Table 9 and Table 10, and the modified probability presented in Table 8.

Table 9 – Summary of Drift Thickness Throughout Historic Landslide Footprints

Drift Thickness (m)	Number of Incidences	Probability (5.4-year return period)
0 to 10	8	0.013
10 to 15	7	0.011
15 to 25	34	0.055
25 to 50	49	0.079
50 to 100	16	0.026

Calculated as: (#incidences for events(landslide

Table 10 – Summary of Relief Throughout Historic Landslide Footprints

Surface Relief (m)	Number of Incidences	Probability (5.4-year return period)
0 to 4	3	0.005
4 to 6	2	0.003
6 to 8	2	0.003
8 to 10	6	0.009
10 to 12	8	0.013
12 to 14	11	0.018
14 to 16	16	0.026
16 to 18	8	0.013
18 to 20	5	0.008
20 to 25	21	0.034
25 to 30	12	0.019
30 to 40	13	0.021
> 40	6	0.009

Two scenarios will be evaluated for landslide probability calculations for the subject site. The first scenario represents the eastern portion of the subject site where surface relief is less than 12 m, and drift thickness is less than 15 m. The second scenario considers the western portion of the subject site, where the surface relief is up to 20 m and drift thickness up to 25 m. The calculations for each scenario are presented in the following paragraphs.

2 SCENARIOS ARE DISCUSSED, AS DESCRIBED IN THE DOCUMENT

Scenario A - Eastern Portion of Subject Site

For the eastern portion of the subject site, the observed surface relief ranges between 0 and 12 m, and the drift thickness ranges between 0 and 15 m. The landslide probability calculations considering the above-mentioned conditions are included in Table 11 in the following page.

Table 11 - Estimate of Site-Specific Landslide Probability Based on 5.4-yr return period	
Factor	**Probability**
Site-Specific Baseline Probability (Table 8)	2.4×10^{-2}
Probability of Landslide – Drift Thickness up to 15 m	2.4×10^{-2}
Probability of Landslide – Surface Relief up to 12 m	3.4×10^{-2}
Site-Specific Probability	**2.0×10^{-5}**

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Based on our assessment, the probability for a landslide to occur throughout the eastern portion of the subject site has been estimated to be less than **1:48,948 per year**. The above-noted probability was calculated as the product of the baseline probability, the probability of landslide occurrence based on cumulative drift thickness up to 15 m, and the cumulative probability based on surface relief up to 12 m, considering a return period of 5.4 years. Based on the above, the annual probability of a large landslide occurring at or directly impacting the eastern portion of the subject site is estimated to be less than 1:10,000 per year.

Scenario B - Western Portion of Subject Site

For the western portion of the subject site, the observed surface relief exceeds 12 m, and the drift thickness exceeds 15 m. The landslide probability calculations considering the above-mentioned conditions are included in Table 12 below.

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Table 12 - Estimate of Site-Specific Landslide Probability Based on 5.4-yr return period	
Factor	**Probability**
Site-Specific Baseline Probability (Table 8)	2.4×10^{-2}
Probability of Landslide – Drift Thickness up to 25 m	8.0×10^{-2}
Probability of Landslide – Surface Relief up to 20 m	1.0×10^{-1}
Site-Specific Probability	**1.9×10^{-4}**

Based on our assessment, the probability for a landslide to occur throughout the western portion of the subject site has been estimated to be **1:5,158 per year**. Based on the above, the annual probability of a large landslide occurring at or directly impacting the subject site is estimated to be more than 1:10,000 per year for portions of the subject site where drift thickness exceeds 15 m and topographic relief exceeds 12 m. Based on this, the area of the subject site corresponding to a probability exceeding 1:10,000 is located at the western boundary of the subject site, and as depicted in Figure 4 below.

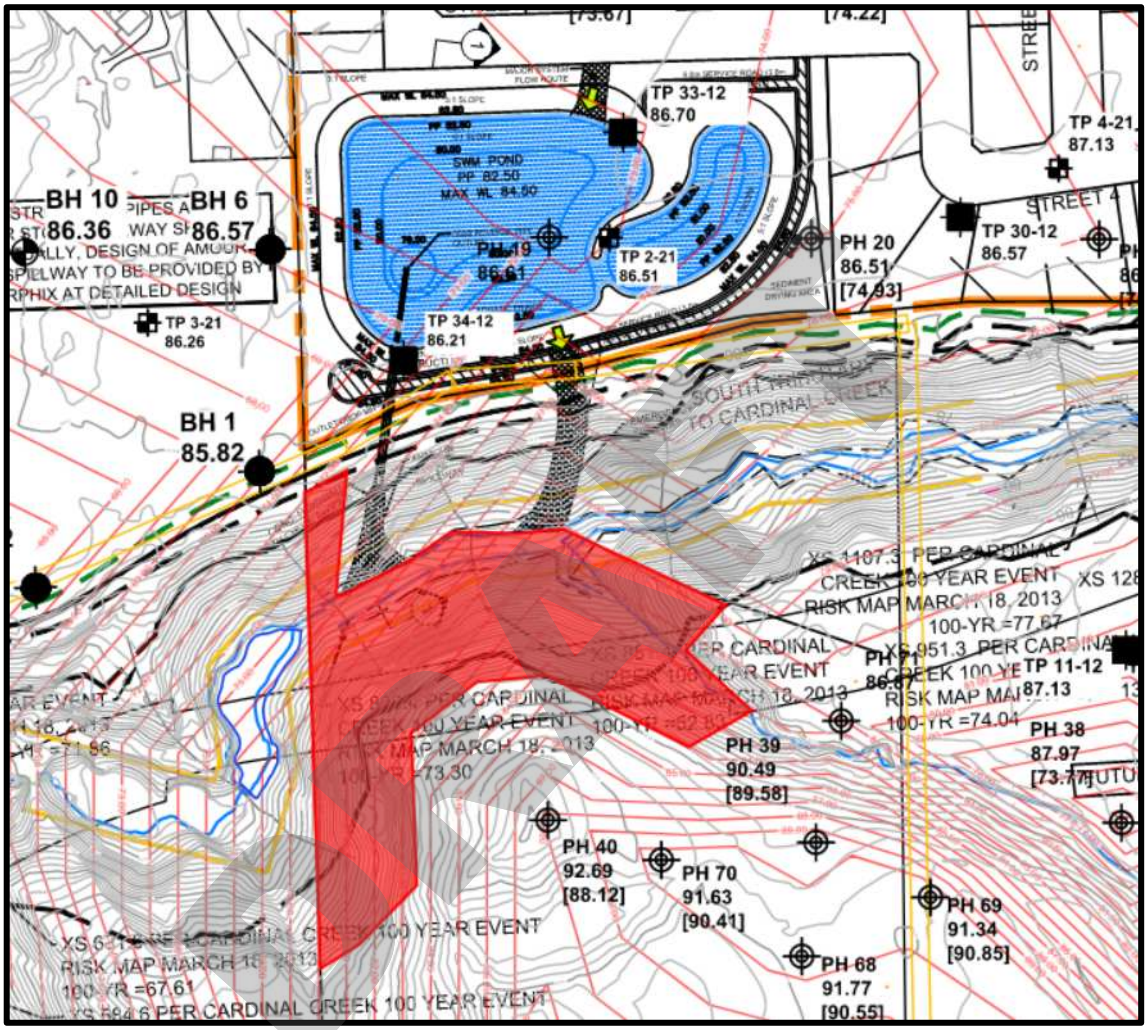

Figure 4 – Area of Subject Site Above 1:10,000-Annual Probability Threshold

Based on our review, pre-development and post-development conditions throughout this portion of the subject site are such that the probability for a landslide to occur is considered to be more than 1:10,000 per year.

Landslide Susceptibility Mitigation

Reviewing the above-noted parameters affecting susceptibility, the most prevalent trigger in initiating slope failures that can result in landslides and further in retrogressive landslides is considered to be toe erosion.

The remaining factors are considered to be either innate to the property (i.e., sensitivity of clay deposit, depth to bedrock, proximity to landslides and watercourse, sensitivity to vibrations from construction and earthquakes) or uneconomical to sufficiently reduce the hazard probability (i.e., slope inclination and surface relief). Based on this, toe erosion protection could be considered a suitable strategy to reduce erosion potential of the bank face supporting the overlying slope and further reduce the potential for a landslide to be triggered by a slope failure.

Toe erosion protection is commonly used throughout the Ottawa and Gatineau regions for mitigating erosion potential for soils in contact with active watercourses. The implementation of erosion protection would be advised by Paterson and the project geomorphologist to ensure the strategy would be implemented in a manner that is hydraulically compatible with the local drainage features, to mitigate downstream effects in increasing erosion potential to slopes located downstream of the subject site, and to minimize maintenance of the erosion protection measure.

Typically, the erosion protection measure would consist of a relatively thick layer of erosion protection stone, such as “rip-rap”, placed upon a layer of non-woven geotextile fastened directly into the slope profile. The erosion protection would extend between the base of the channel and to a pre-designated elevation (typically considered as 300 mm above the 1:100-year water level within the watercourse) and be dressed with bioengineering features to permit reinstatement of vegetation and naturalization of the channel in conjunction with providing features that would minimize degradation of the control measure over time.

At this time, Paterson has not prepared a detail outlining a site-specific recommendation for erosion protection that would be implemented throughout this subject site. If consideration is given to implementing the detail by the City of Ottawa and RVCA, Paterson can collaborate with GEO-Morphix and associated stakeholders in providing an acceptable solution to mitigating on-going toe erosion and reducing long-term landslide risk throughout the subject site.

In the event that toe erosion measures could be implemented at the base of the above-noted portions of the subject slopes, hazard probability estimates would be considered to be revised as indicated in Table 13 in the following page.

Table 13 – Summary of Site-Specific Baseline Probability Modification Factors	
Baseline Probability for Landslide to Occur Throughout Subject Site	5.5×10^{-4}
Section 4.2.1. – Overburden and Clay Sensitivity	3.0
Section 4.2.2. – Inclination, Bedrock Depth and Surface Relief	
Inclination	1.5
Bedrock Depth	Estimated Subsequent Tables
Surface Relief	
Section 4.2.3. – Groundwater, Surface Drainage and Toe Erosion	
Groundwater	1.0
Surface Drainage	1.5
Toe Erosion	**1.0**
Section 4.2.4. – Proximity to Landslides	2.0
Section 4.2.5. – Earthquakes and Seismic Hazard	1.1
Section 4.2.6. – Sources of Anthropogenic/Construction Vibrations	1.5
Modified Site-Specific Baseline Probability (Not Considering Drift Thickness and Surface Relief Factors)	**1.2×10^{-2}**

Modifying the toe erosion factor from 2.0 to 1.0 (1.0 signifying that erosion protection is in place and mitigates the trigger as a potential cause for slope failures and slumps, as is currently being observed throughout the subject site) would result in the following adjusted probabilities provided in Table 14:

Table 14 - Estimate of Site-Specific Landslide Probability Based on 5.4-yr return period

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Factor	Probability
Site-Specific Baseline Probability (Table 8)	1.2×10^{-2}
Probability of Landslide – Drift Thickness up to 25 m	8.0×10^{-2}
Probability of Landslide – Surface Relief up to 20 m	1.0×10^{-1}
Site-Specific Probability	**9.7×10^{-5}**

Table 14 demonstrates the use of toe erosion protection measures would reduce the probability for a landslide to occur throughout portions of the subject site that are currently most susceptible to being affected by a landslide if a slope failure was triggered. If toe erosion protection was considered throughout the western portion of the subject site where slope heights exceed 12 m, the probably for a landslide to occur throughout the subject site would be considered **1:10,317 per year.**

The portion of the subject site that is most susceptible for this to occur is generally located along the western-third of the site along the tributary. Throughout this area, there are sufficient factors present in conjunction with overburden thickness and slope height. While the remainder of the subject site has the majority of the same attributes (sensitive clay overburden, active toe erosion, experiencing surface drainage, etc.), either overburden is too shallow and/or the height of the slopes are too low to be susceptible to triggering a large landslide.

In the event that toe erosion protection is not considered as an acceptable solution to reduce the potential for a landslide to occur throughout the subject site, the portion of the subject site identified on Figure 4 would be subject to a risk assessment based on the current RVCA guidelines. The remainder portions of the subject site located east of the area of higher susceptibility would remain below the threshold requiring a risk assessment.

Based on this review, Paterson anticipates that the area located downstream of the subject site and west of the area identified in the above-noted figure has higher chances for a landslide to occur. This is based on the trend and expectation for the overburden thickness to increase towards the west of the subject and slope heights also increasing towards the west and as the tributary joins the main Cardinal Creek artery.

However, since this area is located beyond the subject site and where site-specific geotechnical information is currently known, additional studies by others would be required to confirm the hazard probability beyond the subject site. Paterson suggests that the western portion of the tributary be investigated further by the City of Ottawa and RVCA, as recommended in our memorandum PG5201-MEMO.03 dated November 13, 2023, given recent failures that have been documented and observed by the City of Ottawa west of the subject site and since estimates provided in this report indicate the probability would be higher throughout those areas. Similar to the subject site, it is suggested that the toe erosion protection measures be explored as a potential solution to minimize the probability for a landslide to occur west of the subject site.

While toe erosion protection will lower the probability for a landslide to occur throughout the subject site it is not considered required throughout the length of tributary throughout the subject site. The area east of the highly susceptible area identified on Figure 4 is considered to below the threshold which would require a risk assessment and does not require the use of toe erosion protection to reduce the potential for a landslide to occur at this time.

Further, since the proposed development will be phased, and likely phased in several sub-phases located on the north and then south side of the tributary, it is not considered a requirement to implement toe erosion protection along the south side of the tributary until consideration will be given to developing the portion of the site located south of the tributary and supported by the area identified in Figure 4.

At the time of preparing this report, the current area of the proposed development is located on the northern half of the tributary, and the area that would be recommended to be improved upon by toe erosion protection in advance of that development would be the portion of the site downstream of the proposed SWMF. The location of the SWMF is considered suitable given our findings, despite it being in an area that is considered more prone to a landslide occurring.

However, since the area of the SWMF will be constructed by lowering the ground surface around the pond, removing a significant volume of overburden and depth of subsoil (approximately 6 m of soil removal throughout the pond footprint) and providing dedicated erosion-protected outlet channels for pond outflow, the probability for a landslide to occur throughout this area would be improved from the estimates provided in Table 11 despite the depth to bedrock and pre-development slope height in this area.

Based on this, Paterson estimates that the post-development condition associated with the current grading and location of the proposed SWMF would reduce the probability for a landslide to occur as being below the 1:10,000 annual threshold. Therefore, while the area of the SWMF may be located in an area where the pre-development condition results in a probability exceeding the 1:10,000 threshold, the implementation of the currently understood SWMF design would reduce the probability sufficiently such that the currently proposed location is considered as preferential from a landslide-hazard perspective. However, since peak-volumes of post-development discharge is anticipated to be higher throughout this portion of the tributary within the subject site, the erosion protection measure recommended herein would offset the potential toe erosion that would result in the location and implementation of the proposed SWMF.

Summary and Conclusion

Based on Peterson's review, the current pre-development landslide hazard probability throughout the subject ranges between 1:5,158 and 1: 48,948 per year. However, Paterson has demonstrated that an acceptable solution to lower the probability throughout portions of the site where the hazard probability is greater than 1:10,000 per year could consist of toe erosion protection. Implementing this technique throughout these portions of the creek in advance of any type of potential development, and in an effort to make the subject slopes safer and limit the potential for landslide to occur throughout the subject site, the hazard probability would instead be considered less than **1:10,317.**

Based on this, Paterson recommends toe erosion protection be considered as part of maintaining the existing tributary to minimize erosion in areas that are susceptible to a landslide if a slope failure were triggered. Undertaking this measure would yield safer slopes and minimize the risk for a landslide to occur throughout the subject site.

The majority of the subject site is considered to be safe and suitable in consideration of the proposed development, however, some efforts involving enhancing the existing creek are recommended to be undertaken in advance of development-works to improve the safety of the subject slopes and reduce the potential for slope failure to result in a landslide throughout the subject site.

5.0 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 Taggart Investments 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.

Oct. 18, 2024

Fernanda Carozzi, PhD. Geoph.   Drew Petahtegoose, P. Eng.

Report Distribution:

- ❑ Taggart Investments
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