Landslide Hazard and Risk Assessment

Final Report

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Prepared for:

DCR/Phoenix Group of Companies
Ottawa, ON

Prepared by:

Stantec Consulting Ltd.
Ottawa, ON

LANDSLIDE HAZARD AND RISK ASSESSMENT

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LANDSLIDE HAZARD AND RISK ASSESSMENT

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Piraux, Olivier

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Prepared by____

(signature)

Olivier Piraux, M.Sc.

Prepared by ____

(signature)

Gwangha Roh, P.Eng., Ph.D.

All sections except Chapter 5: Stability Analysis

2021-12-14

PROFESSIONAL GEOSCIENTIST
RICHARD GUTHRIE
PRACTISING MEMBER
3084
ONTARIO

Reviewed by ____

(signature)

Rick Guthrie, M.Sc., Ph.D., P.Geo. MASME

2021-12-14

Approved by ____

(signature)

Raymond Haché, P. Eng. M.Sc.

Table of Contents

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

Exp Services Inc. (hereafter EXP) is under contract with DCR/Phoenix Group of Companies (hereafter Phoenix Homes) to look for opportunities for a proposed residential development at 1154 – 1208 Old Montreal Road in Ottawa, Ontario. As part of the proposed development application for that area, the Rideau Valley Conservation Authority (RVCA) required that a quantitative landslide hazard and risk assessment be completed. Specifically, RVCA requires that a professional assessment demonstrate that the landslide hazard, including a large “catastrophic landslide”, has an annual probability less than 1:10,000, or that mitigation is possible such that the risk can be reduced to below 10^{-5} per year.

In May 2021, Stantec was retained by EXP to conduct that assessment.

1.1 SCOPE OF WORK

Contracted to EXP, and authorized by Phoenix Homes, Stantec carried out a landslide hazard and risk assessment wherein the scope of work was designed to meet the requirements of the RVCA for the proposed residential development at 1154 – 1208 Old Montreal Road.

The following landslide hazard and risk assessment is based on Stantec’s understanding of site conditions, the nature of the proposed development, available and acquired subsurface data, our knowledge of the area, and the published geotechnical properties of landslides in marine clays.

The reader is referred to the following chapters for a summary of the scope of work, information about our investigative approach and findings of the overall assessment:

- Literature and existing data review (Chapter 2)
- Desktop interpretation and mapping (Chapter 3)
- Field investigation (Chapter 4)
- Stability analysis (Chapter 5)
- Risk assessment (Chapter 6)

1.2 PROJECT AREA

This landslide hazard and risk assessment study focuses on a proposed residential development located at 1154 – 1208 Old Montreal Road in Ottawa, Ontario (Figure 1-1). Within this report, the following spatial references are used to present site conditions:

- The “Project Area” is used to describe an approximated 500 m buffer applied to the general footprint of the proposed residential development, in addition to the Project Site itself. This extent was selected to capture significant historical landslides located within a reasonable range of the Site, as presented in the various thematic maps and figures presented throughout the report.
- The “Project Site” (or the “Site”) is used to describe the limits of the land parcels forming the proposed residential development layout as displayed on the Site Plan previously developed for the project (see Appendix B).
- Of particular interest is the assessment of the “South-facing Slope”, which consist of the slope segment bordering the south boundary of the Site, immediately downslope from the proposed development area.
- The “Unnamed Creek” is used to describe the west-flowing tributary located at the base of the south-facing slope. This creek and small tributary gullies flow into Cardinal Creek some half a kilometer southwest from the Site.

These locations are presented in Figure 1-2 below.

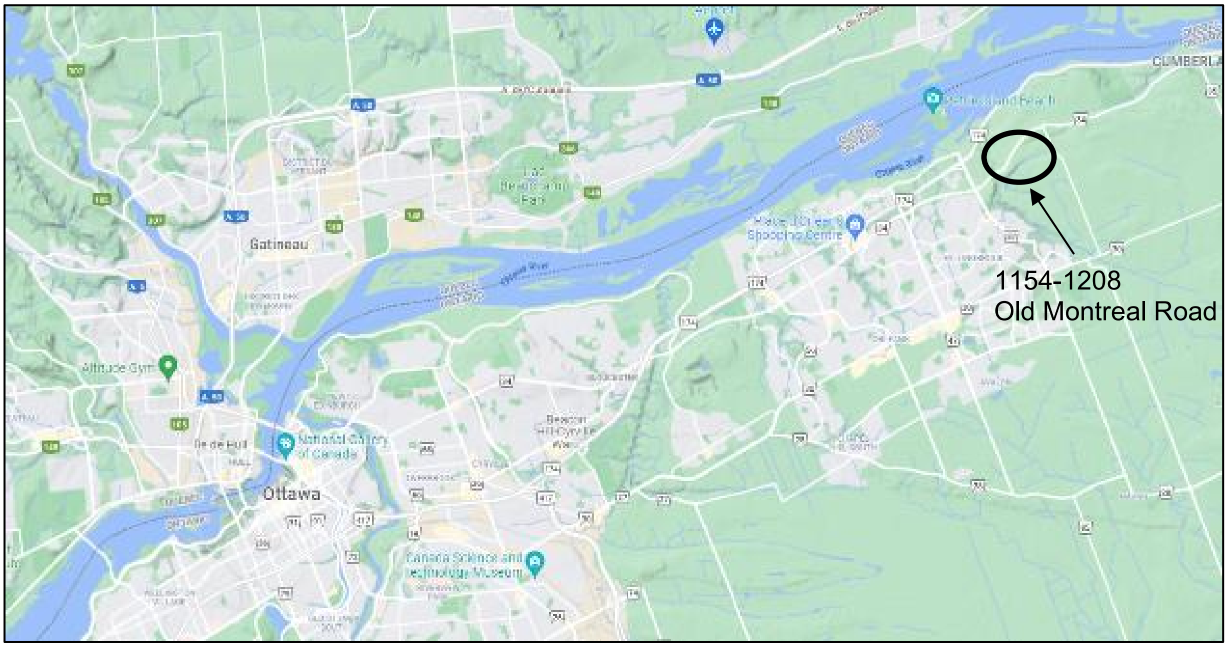


Figure 1-1 General Project Location

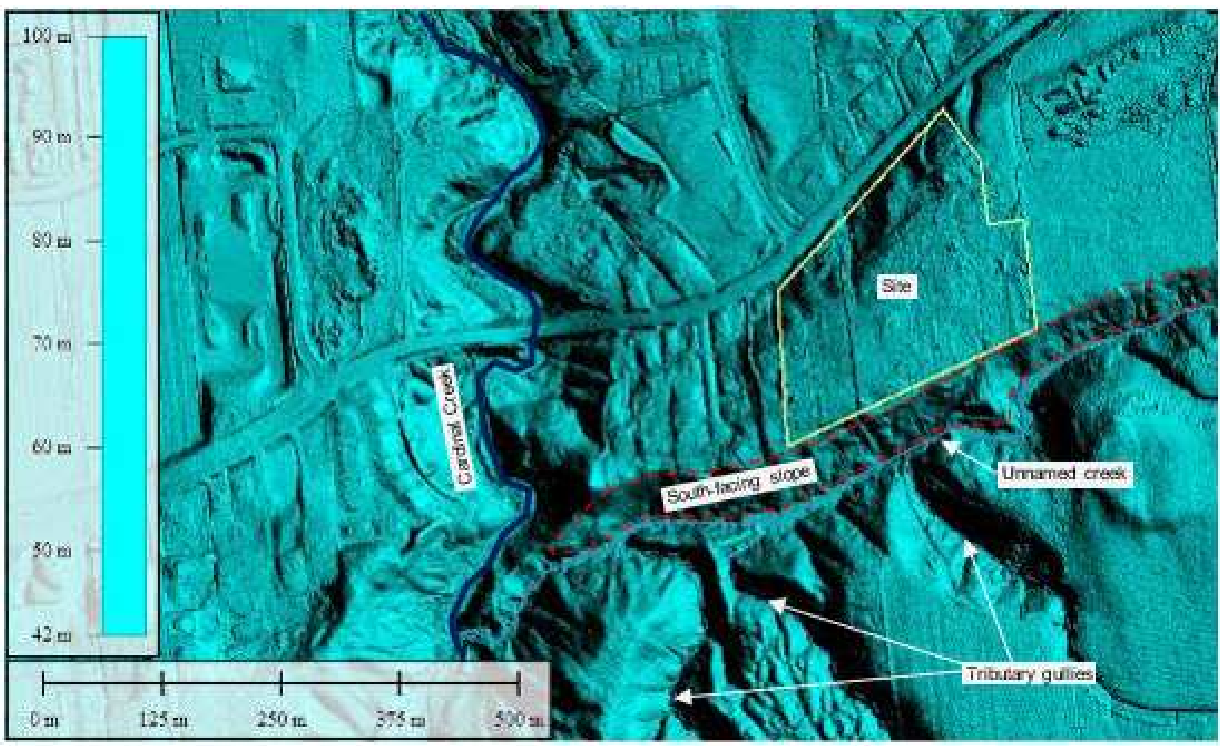


Figure 1-2 Overview of Project Area and Site

2.0 LITERATURE AND EXISTING DATA REVIEW

2.1 DATA SOURCES AND LIMITATIONS

Key data sources utilized as part of the assessment include the following:

- Project-specific information and data received from the client, including EXP's geotechnical investigation report (EXP, 2021) and copy of the Phoenix's proposed subdivision layout (Appendix B)
- Air photos, satellite imagery and Light Detection and Ranging (LiDAR) data, as listed in Table 2-1
- Provincial geospatial data including contour intervals, watershed delineation, stream layer and other natural environment geospatial data
- Ontario Geological Survey (OGS) maps and spatial data relevant to bedrock geology, surficial geology, bedrock topography and overburden thickness
- Ontario Water Well Information System (WIS) for water well records including static levels and depth-to-bedrock
- Other existing data, reports and literature known to Stantec regarding the occurrence of landslides in sensitive clay in the Ottawa region (see references listed in Chapter 8)

Geotechnical drilling, field, and laboratory testing was undertaken by EXP in 2017/2018 (EXP, 2021). A field reconnaissance and supplementary laboratory testing on samples collected by EXP was carried out by Stantec in the summer of 2021. The background information presented herein is, therefore, based on data compiled from both programs as well as upon Stantec's interpretation of surface/subsurface conditions present within the more general project area based on other available data sources.

The use of several different data sources means that the accuracy and precision of the data will vary. Although surface/subsurface conditions are expected to be relatively consistent across the Site, localized variations may not be captured in existing data. Extrapolation of in-situ conditions can only be made for a limited extent beyond the locations referenced. The extent depends on variability of the soil, superficial materials, bedrock, soil moisture and groundwater conditions as influenced by geological processes and local land use. For these reasons, caution should be exercised when interpreting data obtained from regional dataset or from locations not specific to the current Site.

Refer to Stantec's Professional Services Terms and Conditions in Appendix A for additional information.

Table 2-1 Air Photos, Satellite Imagery and LiDAR Data used as part of the Assessment

Data type (Source)	Acquisition date / Air Photo ID	Scale/point density[1]
Air Photos (NAPL)	1951 / A13283 0028-0029	Scale 1:50,000
	1960 / A17262 0134-0135	Scale 1:25,000
	1973 / A23188 0199-0200	Scale 1:25,000
	1981 / A31267 0048-0049	Scale 1:25,000
	1990 / A31579 0052-0053	Scale 1:55,000
	1998 / A28361 0004-0005	Scale 1:22,800
Satellite Imagery (Google Earth Pro™)	2003/05/28	N.A.
	2004/11/19	
	2008/09/30	
	2012/03/30	
	2013/09/24	
	2014/05/06	
	2015/06/18	
	2016/04/19	
	2017/05/19	
	2018/06/01	
LiDAR (City of Ottawa)	2006 (month/day unknown)	2.3 pts/m2
	2014, Nov. 11 to Dec. 7	14.7 pts/m2
	2018, Apr. 18 to May 21	27.9 pts/m2
[1] Corresponds to unclassified LIDAR point count density (i.e., bare-earth point density is lower)		

2.1 GEOLOGICAL SETTING

The project area is located within the Great Lakes - St. Laurence Lowlands physiographic region of Canada, within the Central Lowland subregion (Government of Canada, Natural Resources Canada, 2017). This subregion was heavily modified by Pleistocene glaciations and postglacial processes, resulting in often thick sequences of unconsolidated materials deposited overtop of sedimentary bedrock.

Large areas of the Ottawa Valley are underlain by glaciomarine (marine) silty-clay and clayey-silt (or ‘clay’) deposits that accumulated in the Champlain Sea, a post-glacial sea that inundated the area following the retreat of the Laurentide Ice Sheet between 14.0 and 10.5 thousand years before present (Dyke, 2004). As the Champlain Sea receded during the early Holocene as a result of regional uplift, the postglacial stream network developed and incised into the marine deposits.

Within the project area, existing surficial geology maps (Richard et al. 1978, Richard 1982, St-Onge 2009, OGS 2010) identified the presence of offshore marine deposits (predominantly silt and clays but can include beds of sand and gravels), till, colluvium and bedrock.

The marine soils are, also referred to informally as ‘Leda Clay’ (Gadd, 1975), are known to be geotechnically sensitive, being the source of over 250 mapped landslides (predominantly earth flows) in the National Capital Region (Brooks and Aylsworth 2011). These landslide deposits represent most of the mapped colluvial deposits in the area. The till deposits, although exposed in some areas, are often buried below marine soils. A figure presenting the main geomorphological units found within proximity of the Site is presented in Figure 2-1. Summary description of surficial materials are included.

Development at the Site itself is proposed to occur on a flat to gently undulating topographic high associated to a former marine terrace. The surface of the terrace gently slopes towards the west (less than 1 degree), with natural ground surface elevations ranging from approximately 86 m to 81 m above sea level (asl). The terrace is bound to the south by a gully system whose catchment area (approximately 3 km^2, including the Site itself) is a subset of the Cardinal Creek sub-watershed. The gully system is contained within the marine deposit and does not appear to be bedrock controlled. Moderate to steep escarpment slopes are present along the gully sidewalls, including on the south-facing slope marking the southern limit of the Site. Immediately south of the Site, the south-facing slope is 12 to 20 m in height, 40 to 60 m in length, and presenting gradients generally ranging between 10 and 20 degrees. Steeper slopes in the range of 25 to 35 degrees are found locally; however, limited to small escarpments.

An unnamed creek flows westerly at the base of south-facing slope, merging into Cardinal Creek some 500 m downstream from the Site. The flow regime of this second-degree stream is unknown; however, understood to be low-energy and potentially intermittent. The stream channel itself is less than two meters wide, sinuous and characterized by a gradient generally lower than two degrees. A few ephemeral tributary channels and gullies feed into the unnamed creek from the south, including two gully system where landslides have been identified. Refer to Figure 1-2 and Figure 2-1 below for an overview of Cardinal Creek and of the tributary gullies south of the Site.

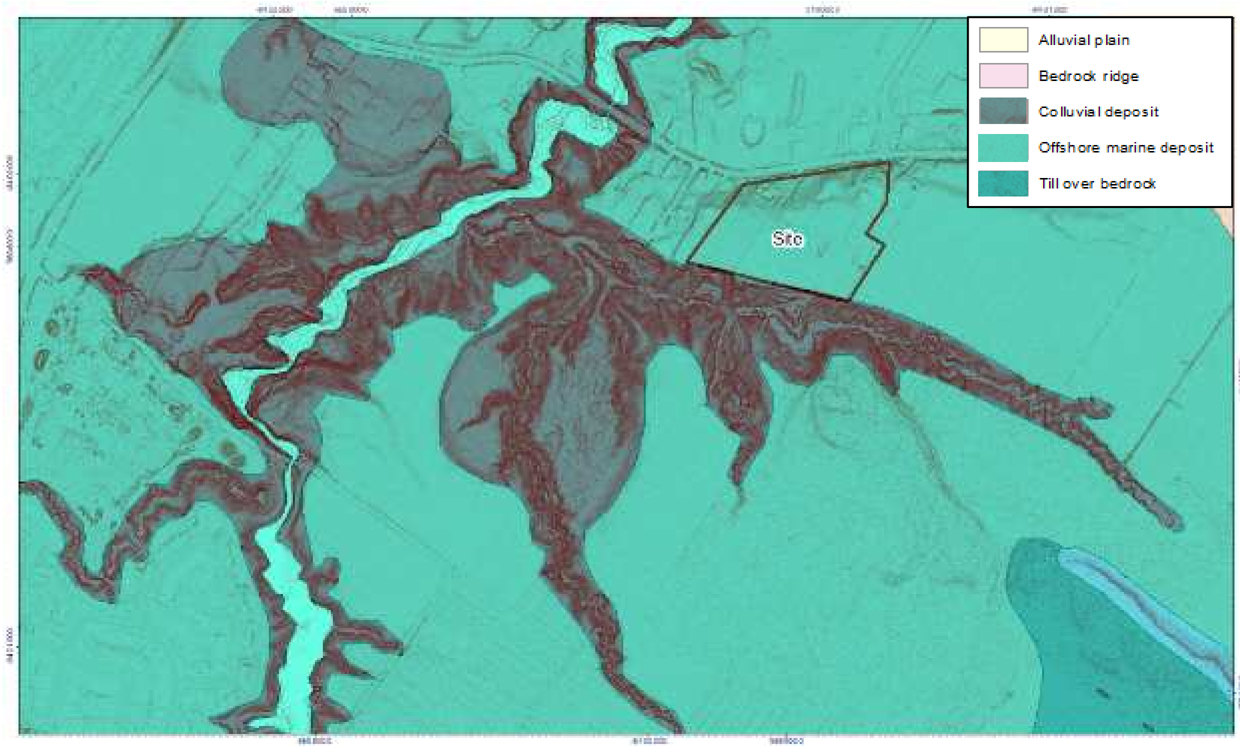
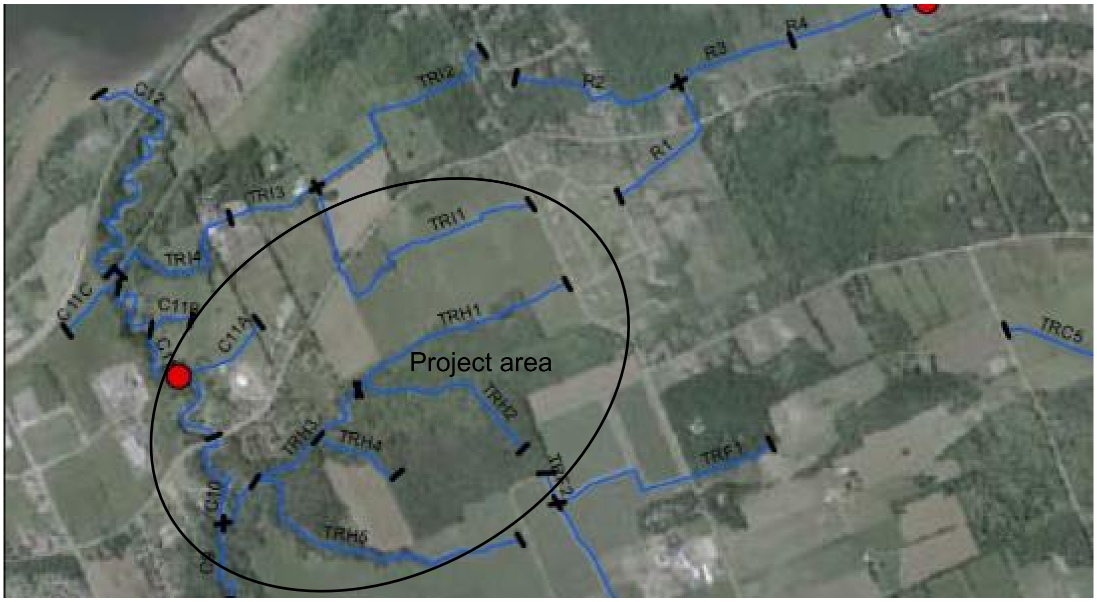


Figure 2-1 Geomorphological Units – Old Montreal Road Area

Surficial Material[1]	Description
Offshore marine deposit	• Deposit associated to the Champlain Sea • Includes clay and silt underlying erosional terraces; upper part of marine deposits removed to variable depths by fluvial erosion so in places clay is uniform blue-grey[1].
Colluvium and landslide deposit	• Most colluvial materials found within the project area are related to mass movement processes that have impacted the marine silt and clay deposits associated to the Champlain Sea. Landslide deposits include the ones identified by Brooks (2019) as well as a number of smaller features not yet documented but visible on bare-earth LiDAR images (see Chapter 3). The thickness of these deposits is expected to vary depending on the size of the various landslides, from a few meters for small landslides, to up to ~10m for large retrogressive landslides. Colluvial materials resulting from slow downslope movements along gully side walls are expected to be thin (<1 m).
Till	• Sandy and silty compact diamicton, grey at depth but brown where oxidized; calcareous where derived from sedimentary rocks and not leached; consists dominantly of lodgment till (sandy to silty sand). In areas that lie below the marine limit, the till it is often overlain by a discontinuous layer of marine deposit.
Bedrock	• Bedrock underlying the Site is expected to consist of sedimentary rocks associated to the Rockcliffe Formation[2] (limestone, dolostone, shale, arkose, sandstone). • The review of available background information suggest that bedrock is over 10 m deep at the Site[3]. No visible outcrops are expected to be present along the unnamed Creek.
[1] OGS 2010 (map MRD128), [2] OGS 2011 (map MRD126-REV1), [3] Ontario Water Well Information System (WIS)	



Source: Geomorphic Solution 2007.
North-flowing Cardinal Creek (visible as C9 to C10) and tributaries. The unnamed creek is identified as TRI1 to TRI4.

Figure 2-2 Cardinal Creek and Tributaries near the Project Area

2.2 CHARACTERISTICS OF CLAY LANDSLIDES IN THE OTTAWA REGION

2.2.1 Main Landslide Types

Landslides in sensitive clay deposits of Eastern Canada are generally located along active river or stream valleys as well as the margins of paleochannels and terraces of former streams. Such features are common in the Ottawa Region, where several large retrogressive landslides are present along the margins of paleochannels and upper terraces of the Ottawa River (Russell et al. 2011). The majority of these large failures are prehistoric and of unknown age, although some significant recent landslides also occurred.

The four most common types of slope failures observed within the project area consist of:

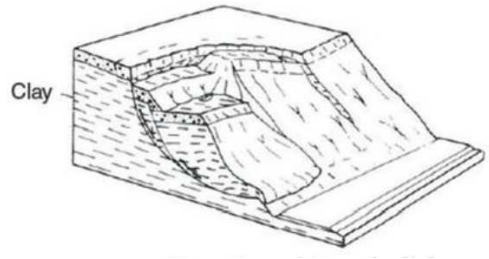
- Surficial slope failures
- Rotational landslides
- Large retrogressive landslides, subcategorized into flowslides and spreads

2.2.1.1 Surficial Slope Failures

Surficial slope failures occur within the face of the slope. These types of landslides are often caused by soil saturation during/following heavy rains or snowmelt. Given that the failures are shallow (typically less than 1.5 m), only the dryer surface crust which overlies the deeper and unweathered clay is impacted by these failure types; however, in many cases there may be no clay crust in the middle of the slope. Debris generated by surficial slope failures can spread over significant distances. Based on professional experience, the debris can spread to distances equal to twice the slope height. It is anticipated that hundreds of these types of landslides occur every year in the provinces of Ontario and Quebec.

2.2.1.2 Rotational Landslides

In general, a rotational slide is characterized as soil movement along a curved rupture surface, typically resulting in relatively little internal deformation. The slide movement is roughly rotational about an axis that is parallel to the ground surface and transverse across the slide. Rotational slides exhibit characteristic features such as vertical head-scarps, back-tilted blocks, internal ponds and a succession of secondary scarps. Unlike translational slides, rotational slides can be self-stabilizing when the steep driving blocks transition to passive resting blocks at the toe of the slope, or when the resisting force from the passive resting bocks exceed the driving force of the active block in the head of the landslide. However, head-scarps also remove support along their vertical face creating new active blocks above the landslide head and causing it to retrogress into the slope along usually well-define concave rupture surface.

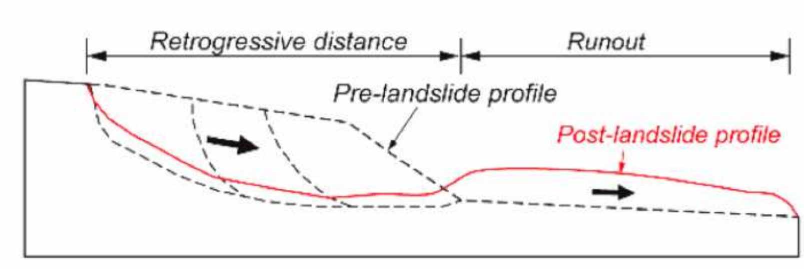


Source: Varnes 1978

Figure 2-3 Schematic of Rotational Clay Landslide

2.2.1.3 Flowslides

In case of flow slide, a rotational landslide initially occurs and followed by a succession of rapid rotational failures under undrained conditions. the source material is almost entirely remoulded or liquefies and flows out of the slide area. Flow slides often exhibit extensive internal deformation when the material being deformed is composed of extra-sensitive or quick, fine-grained soils.

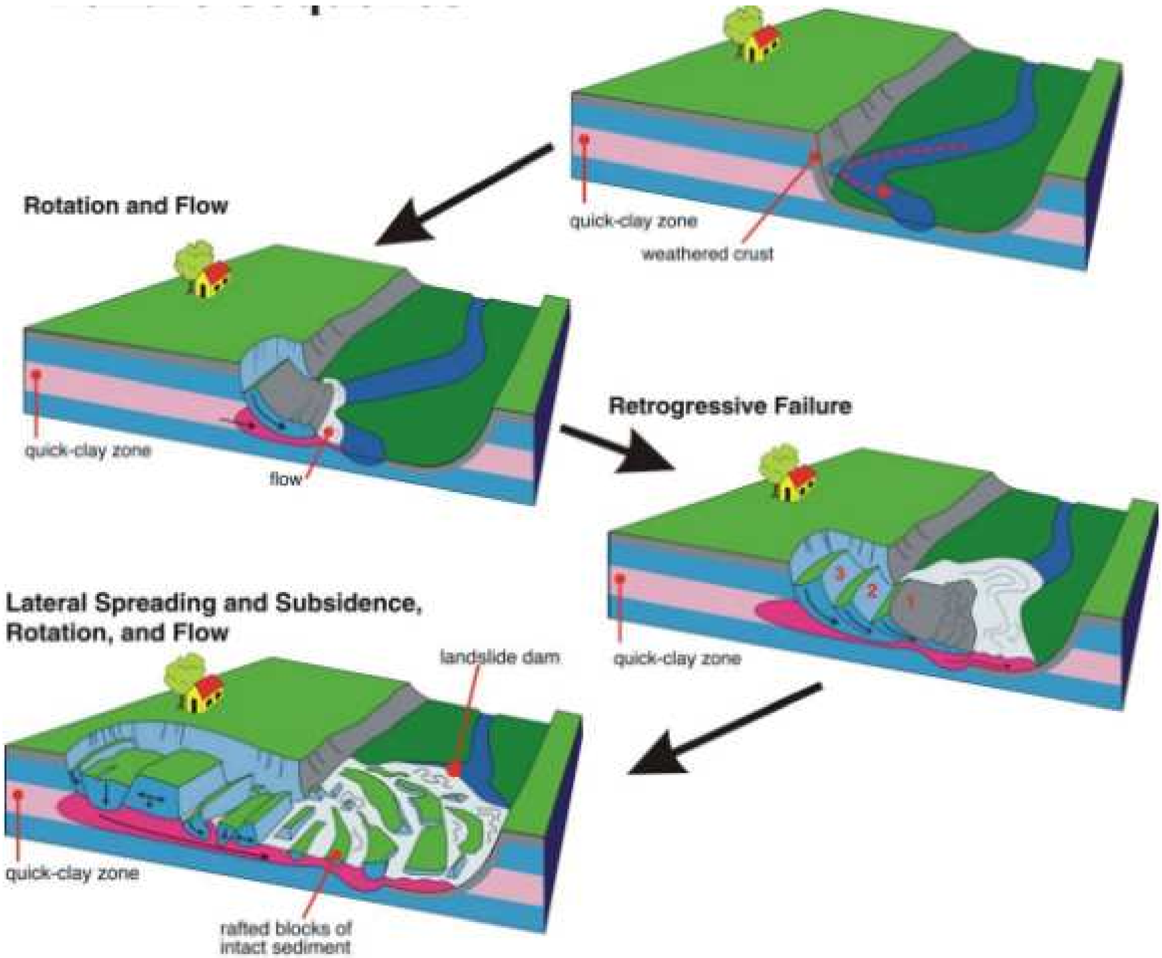


Source: Locat et al. 2011

Figure 2-4 Schematic of Flowslide

2.2.1.4 Spreads

Spreads are the lateral displacement and rotation of intact blocks of cohesive soil supported by, or floating on, a layer of intensely deformed weaker material (Hungr & Picarelli, 2014). They are found to occur on flat to very gentle slopes, where the dominant mode of movement is lateral extension accompanied by shear or tensile fractures. According to (Cruden & Varnes, 1996), spreads result because of the extension and dislocation of the soil mass above the failure surface. The failure is caused by “liquefaction” of a specific layer, the process whereby saturated, loose, cohesionless sediments (usually sands and silts) are transformed from a solid into a liquefied state. Failure is usually triggered by rapid ground motion, such as that experienced during an earthquake, but can also be artificially induced. When coherent material, soil, rests on materials that liquefy, the upper units may undergo fracturing and extension and may then subside, translate, rotate, disintegrate, or liquefy and flow. Lateral spreading in fine-grained materials on shallow slopes is usually progressive. The failure starts suddenly in a small area and spreads rapidly. Often the initial failure is a rotational slide, but in some materials movement occurs for no apparent reason. Spreads can be distinguished from other retrogressive landslides because of their unique geomorphologic shape, where blocks of more-or-less intact clay having horst and graben shapes result in a series of upward pointing ridges.



* Modified from Geoscape Ottawa-Gatineau

Figure 2-5 Schematic Failure Sequence of Large Retrogressive landslide (Spread) in Clay

2.3 PREVIOUS LANDSLIDE STUDIES

Significant background information is available regarding landslides in the Champlain Sea clay. Digital inventory of landslides, as part of ongoing efforts to map and communicate landslide hazards have been presented by several authors and as mentioned earlier, some 250 mapped landslides (predominantly earth flows) are present within an approximate 60 km from the National Capital (Brooks and Ayslworth 2011). Quin et al. (2007, 2008) also presented an inventory of landslides in the Champlain clay.

Information relevant to the current assessment is the recent landslide inventory map and database produced for the Ottawa region (Brooks 2019), where at four distinct retrogressive landslides are present within a one-kilometer distance from the Site (two within 400 m). Refer to Table 2-2, and Figure 2-6 and 2-7 for a subset of the database information compiled by Brooks.

Aylsworth et al. (2000) interpreted that a number of large landslides presenting a common age of about 4550 years before present (yr B.P.) were likely indicative of the occurrence of a prehistoric seismic event. Aylsworth and Lawrence (2003) further presented radiometric-based evidence suggesting that the Ottawa region experienced two geologically destructive earthquakes: one at 4550 yr B.P., caused widespread landsliding in sensitive marine clays, and the second at 7060 yr B.P., causing irregular surface subsidence, lateral spreading, and sediment deformation in thick deposits of marine clay. Bases on worldwide studies of earthquake-induced landslides (e.g., Keefer 1984; Obermeier 1996), Aylsworth and Lawrence (2003) estimated that the magnitude of earthquakes having impacted the region probably exceeded 6.5, which may even be larger than the 2475 years return period. An updated compilation of radiocarbon dates relative to the age of sensitive clay landslides in the Ottawa region was released by the GSC in 2021 (Brooks et al. 2021); however, this update did not provide new information related to the two historical landslides located across the unnamed creek some 400 m south of the Site.

Table 2-2 Subset of landslide database by Brooks (2019)

Site code	Geographical coordinates		Feature	Morphology	Scar Area (km2)	Age	Comments
Oln15	45.48871	-75.47487	Landslide, probably	Truncated source area	0.081	Unknown	The morphology of the feature is consistent with a landslide (or possibly two adjacent landslides) retrogressing into the scarp slope above a terrace surface of the proto-Ottawa River within Orleans. The site is heavily altered by urban development. An assessment of feature on 1945 aerial photographs (NAPL A9612-15 to -19) was inconclusive. Other mapped landslides are located nearby.
Oln16	45.48586	-75.47170	Landslide	Debris field within a narrow stream valley	0.074	Unknown	The landslide retrogressed into the western side of the incised valley of Cardinal Creek, Orleans. The site is heavily altered by urban development. The landslide origin and location were verified on 1945 aerial photographs (NAPL A9612-15 to -19).
Oln17	45.48877	-75.46692	Landslide	Debris field within a narrow stream valley	0.040	Unknown	The landslide retrogressed into the western side of the incised valley of a tributary of Cardinal Creek, Orleans, opposite Oln18. The debris field consists of irregular topography that is incised by the creek. The landslide origin was verified on 1945 aerial photographs (NAPL A9612-15 to -19). Mapped as separate feature from Oln18 on the basis of morphology of the main scarp.
Oln18	45.49016	-75.46497	Landslide	Debris field within a narrow stream valley	0.025	Unknown	The landslide retrogressed into the eastern side of the incised valley of a tributary of Cardinal Creek, Orleans, opposite Oln17. The debris field consists of preserved, irregular topography that is incised by the creek. The landslide origin was verified on 1945 aerial photographs (NAPL A9612-15 to -19). Mapped as separate feature from nearby Oln17 on the basis of the wider main scarp.

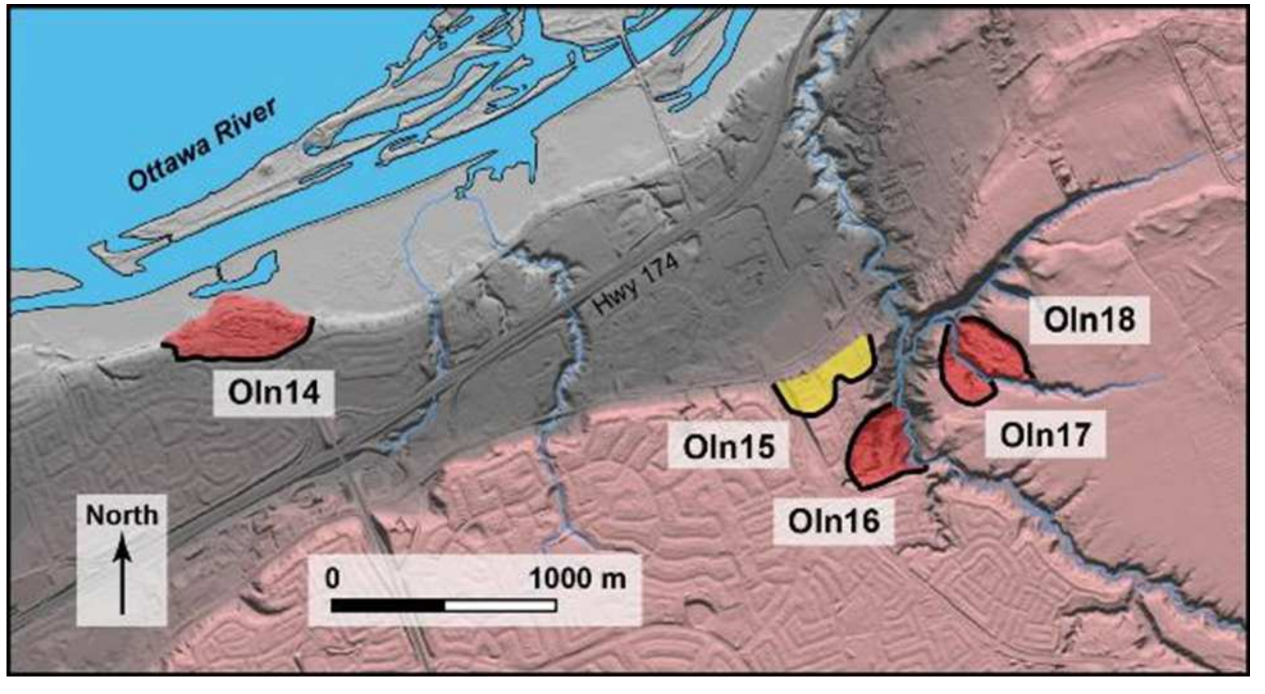


Figure 2-6 Overview of landslide units listed in Table 3 (from Brook 2019)

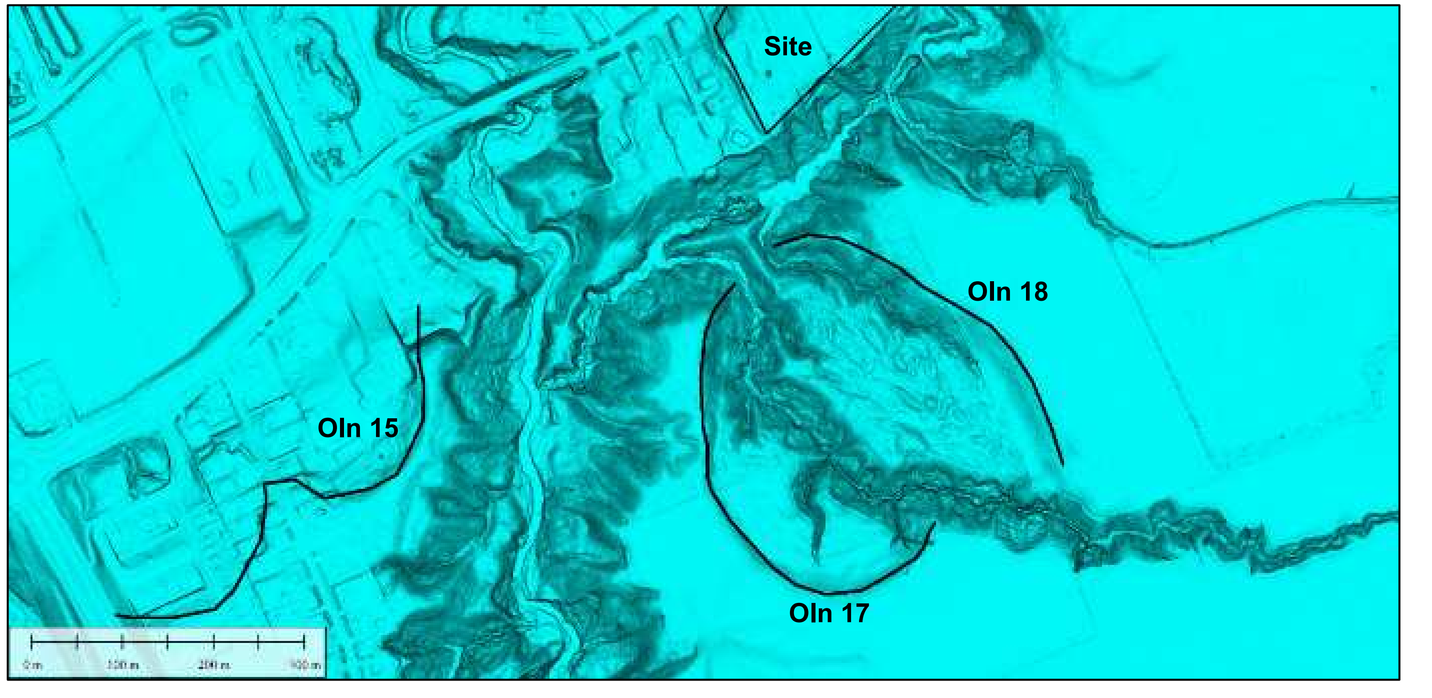


Figure 2-7 2019 bare-earth LiDAR of landslide units OLn 16, 17 and 18

2.4 EXP GEOTECHNICAL INVESTIGATION REPORTS REVIEW

EXP was retained by Phoenix Homes Group of Companies to carry out the preliminary and updated preliminary geotechnical investigations for the proposed residential subdivision. The field work for the preliminary geotechnical investigation was undertaken in two stages (2016 and 2018). A total of 12 boreholes were advanced at the Site to depths ranging from 7.2 m to 23.3 m below the existing grade. DCPT and field vane tests were also carried out as part of EXP investigation. Groundwater levels were measured in the open boreholes on completion of drilling and in four (4) monitoring wells installed in Boreholes 1, 3, 7 and 12. The locations of boreholes are shown on the Figure 2-8 below; however, refer to the geotechnical report presented by EXP (2021) for more details.

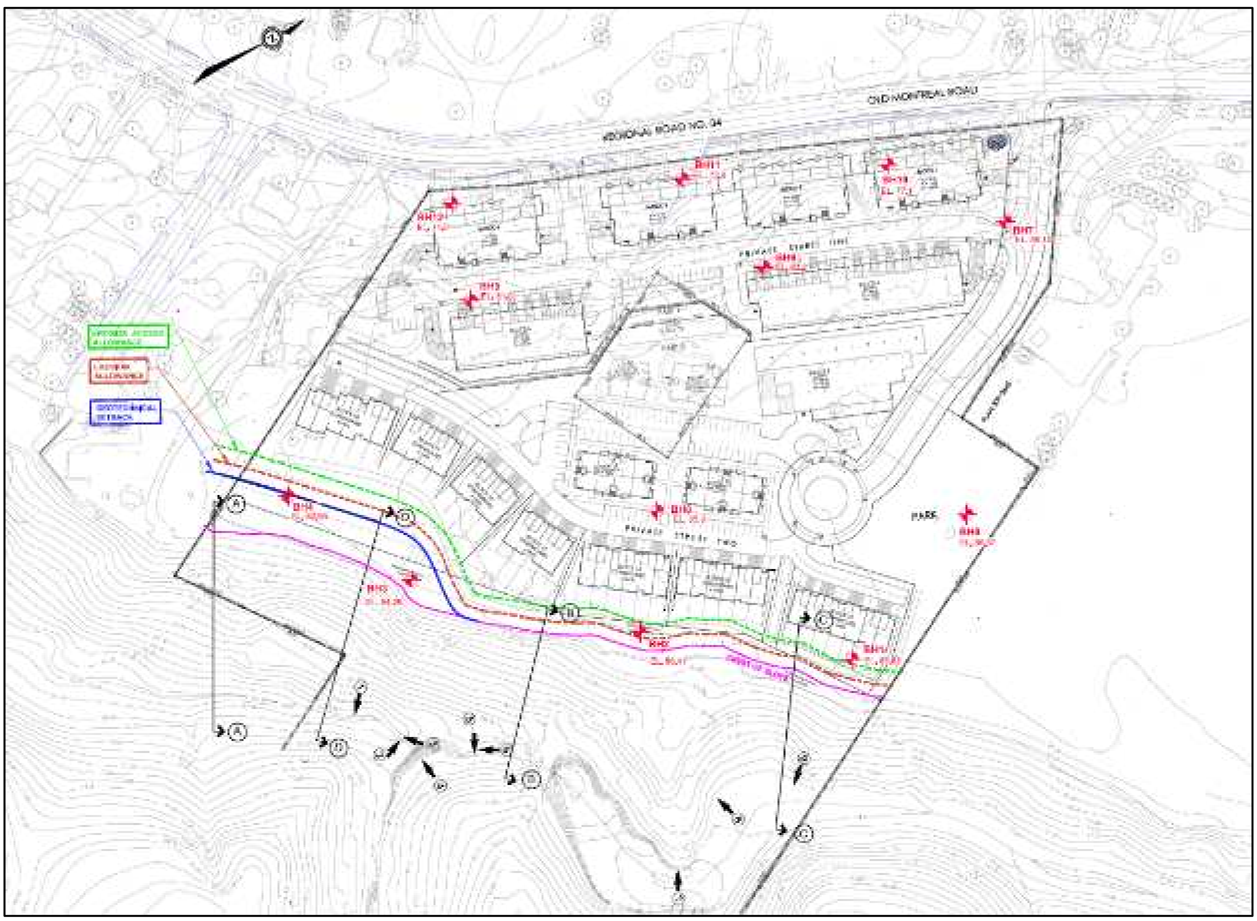


Figure 2-8 EXP Borehole Location Plan (EXP 2021)

The two EXP investigations were carried out mainly for the proposed subdivision assessment and only two boreholes (BH1and BH3) were advanced deep enough along the south boundary of the Site to assess the slope stability. It should be noted that limited field and laboratory tests were originally performed.

The stratigraphy recorded at the Site consisted of:

- Topsoil underlain by;
- Silty sand and gravelly sand / Sand fill underlain by;
- Silty clay crust underlain by;
- Massive silty clay underlain by;
- Silty sand till

Bedrock was inferred based on the DCPT and SPT refusal. Shallow groundwater conditions were measured in the open boreholes upon completion of drilling and in the monitoring wells. Table 2-3 summarizes the field geotechnical testing, laboratory geotechnical testing and the groundwater condition.

Table 2-3 Summary of Subsurface Condition (EXP 2021)

Soil Type	MC (%)	γ (kN/m^3)	SPT N Values	Cu (kPa)	Atterberg Limit Test Results			Hydrometer Test Results (%)			
					MC (%)	PL (%)	LL (%)	Clay	Silt	Sand	Gravel.
Fill	21-30		5 - 14	-	-	-	-	-	-	-	-
Silty Sand	21		6 - 7	-	21-30	-	-	-	-	-	-
Clay Crust	30-63	17.2 - 19.1	2 - 23	106 - >250	38	26.1	64.5	72	26	2	-
Clay	48-74	15.5 - 17.3	HW - 6	50 -220	54-70	24.6-28.3	52.7-62.7	61-69	30-37	1-2	-
Silty Sand Till	4-10	-	11 - 64	-	-	-	-	3	12	46	39

Mc = Natural Moisture Content, Cu= Undrained Shear Strength, HW = Hammer Weight, PL= Plastic Limit, LL= Liquid Limit

Table 2-4 Groundwater Measurements (EXP 2021)

Borehole No.	Date Drilled	Observation Date	Groundwater Depth (m)	Elevation to Groundwater Table (m)
1	August 15, 2016	September 10, 2016 February 15, 2018	1.5 1.3	84.3 84.5
3	August 17, 2016	September 10, 2016 February 15, 2018	2.5 1.5	81.8 82.8
7	August 16, 2016	September 10, 2016	1.3	83.8
8	February 7, 2018	February 15, 2018	0.8	85.1
9	February 6, 2018	February 15, 2018	1.0	81.2
10	February 6, 2018	February 15, 2018	0.7	76.4
11	February 5, 2018	February 15, 2018	1.1	71.8
12	February 7, 2018	February 15, 2018	1.2	70.7

Two-dimensional limit equilibrium slope stability analyses (total, effective and pseudo-static conditions) were carried out for the four slope sections generated from the 2014 Lidar (Table 2-5).

Table 2-5 Natural Slope Inclination of Cross-sections Analyzed (EXP 2021)

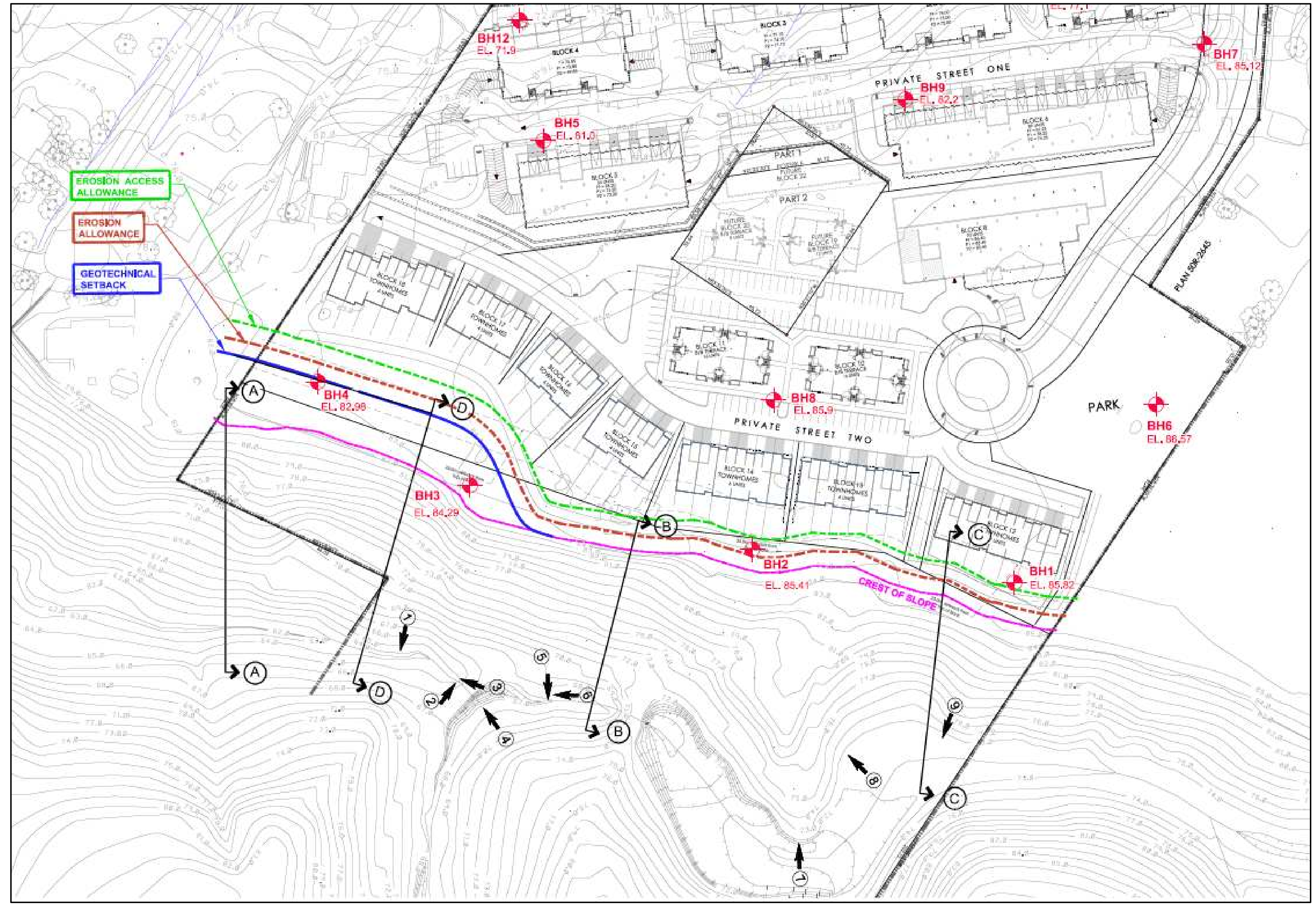
Section	Crest of Slope (m)	Toe of Slope (m)	Height of Slope (m)	Overall Slope Inclination
A-A	81.75	62.0	19.75	3.83H:1V
B-B	84.0	68.0	16.0	3.12H:1V
C-C	85.0	72.25	12.75	3.69H:1V
D-D	83.0	64.0	19.0	3.42H:1V

Liquefaction potential of clayey soil using criteria from Bray et al. (2004) and flow slide potential using Mitchell and Markall (1974) approach were assessed. Results of the slope stability analyses are summarized in Table 2-6. Using these results, EXP concluded that the Site was not prone to flow slides.

Ministry of Natural Resources (MNR) limit of hazardous lands assessment was carried out to provide the development setbacks. These are displayed on Figure 2-9 below.

Table 2-6 Results of Slope Stability Analyses (EXP 2021)

Section	Condition Analysed	Factor of Safety	Required Geotechnical Set Back
A-A	Effective stress analysis and setback determination	1.50	24 m
	Total stress analysis	1.94	-
	Total stress analysis with seismic loading	1.20	-
B-B	Effective stress analysis	1.75	0
	Total stress analysis	2.55	-
	Total stress analysis with seismic loading	1.63	-
C-C	Effective stress analysis	1.59	0
	Total stress analysis	2.21	-
	Total stress analysis with seismic loading	1.49	-
D-D	Effective stress analysis	1.46	0
	Total stress analysis	1.64	0
	Total stress analysis with seismic loading and setback determination	1.10	19 m



3.0 DESKTOP INTERPRETATION AND MAPPING

3.1 METHOD

A desktop review of historical air photos, satellite imagery and LiDAR data was carried out by Stantec to increase the understanding of local terrain conditions and document landslides located within the Project Area.

Desktop terrain mapping was completed using ESRI ArcGIS® software for interpretation of LiDAR data (2006, 2014 and 2019) and satellite imagery. The LiDAR (LAS format) was acquired from City of Ottawa and processed by Stantec to create 0.5 m digital elevation models (DEMs) from which bare-earth hillshade images, slope raster and 1 m contours were generated. Satellite imagery consisted of open source orthoimages available from the Google Earth platform. Refer to Table 2-1 for the list of imagery and data used as part of the assessment.

3.1.1 Historical Review of Landslides and other Geomorphic Processes

Review and interpretation of historical aerial photographs provides important insight into land use changes, both from natural and human induced processes. The end-goal of this review was to identify and record visible landslides that occurred within the project area. Hardcopy air photos covering the 1951-1998 period were obtained from the Natural Resources Canada's National Air Photo Library and reviewed using a 2x optical stereoscope. Satellite imagery and other data were reviewed on a high-resolution computer monitor using ArcGIS and/or the Google Earth Pro online platform at scales ranging from 1:10,000 to 1:1000. Identification of new or recent landslides on the stereo air photos and satellite imagery was based on the absence of vegetation (e.g., trees and shrubs where there would typically be vegetation), apparent exposure of fresh mineral soil where that would not normally be the surface condition expected, and observation of characteristic landslide features such as, for example, curved headscarps and gullies, colluvial cones or fans, splays, levees and lobes of fresh sediment; and irregular (hummocky and undulating) topography.

The use of bare-earth LIDAR images provided the advantage of displaying ground surface without the vegetation cover, therefore allowing for visualization of topographic features otherwise hidden by the canopy layer. This includes the identification of landslides that occurred before 1951, where the delineation of unstable or potentially unstable terrain relied primarily on the morphology of erosional and depositional features noted above. For this, we used the 2006, 2014 and 2019 LiDAR hillshade images, but also the slope map and contour data to refine the morphology exhibited by the LiDAR hillshade images.

3.1.2 Topographic Change Detection

A topographic change detection analysis was conducted following the visual assessment of air photos, satellite imagery and LiDAR data. This remote sensing technique consisted in comparing bare-earth Digital Elevation Models produced from LiDAR datasets acquired over time (in this case 2006, 2014 and 2019) to inform on patterns of loss and gain of material during this timeframe. The resulting dataset is referred as a DEM of Difference (DoD), where in the context of a terrain stability assessment, a negative change in elevation could be indicative of a mass movement process (e.g., landslide or soil erosion), and a positive change in elevation could be indicative of sedimentation or material deposition (e.g., accumulation at the toe of a landslide).

3.2 FINDINGS

Reviewing available data suggest the gully system located south of the Site has developed by basal incision and headward erosion, with contribution of relatively small landslides features (generally under 5,000 m^2 in size) that have developed in gully headwalls, side slopes and/or along the edges of the creek and tributary gullies over time.

Historical air photos indicated that most of the terrain alongside the unnamed creek has remained forested since at least 1951, with agricultural activities first occurring predominantly on flat ground north of the unnamed creek (i.e., current Site), then progressively south of gully system in the early 1960s. Two distinct access trails currently crossing the gully are visible on the 1951 photos.

A series of small landslides are visible alongside the banks of Cardinal Creek on the 1951 air photo, mainly because the vegetation was cleared, making the features visible. Stantec expects that other similar landslides are present alongside the forested portion of the riparian zone. This includes the sidewalls of the unnamed creek and tributary gullies where the thick vegetation canopy is likely masking the footprint of landslides.

Review of recent satellite imagery (2005-2020) suggests that slope stabilization occurred at a number of locations along Cardinal Creek, including two areas located within 0.5 km from the Site. No information specific to these remediation efforts was available for review at the time of conducting this assessment; however, it appears that the activities were carried out to procure stabilization and toe protection required to address shoreline instability.

Further south and west of the Site, the footprints of four large historical landslides are visible on historical photos and LiDAR data reviewed as part of the current assessment. These features, much larger than the majority of landslides observed within the gully system, were included in a wider regional database (Brooks, 2019). Reviewing air photos and LiDAR data revealed no apparent ground movements within those landslide bodies.

3.2.1 Landslide inventory mapping results

A landslide inventory was used to support the desktop terrain mapping and landslide initiation/reactivation hazard mapping and to gain a general sense of how far future landslides may travel within the project area. Summary observations compiled from the desktop interpretation and mapping include the following:

- A total of 65 different landslides were mapped within the Project Area (i.e., an approximate 500 m buffer around the footprint of the Site). These landslides range from small surficial slope failures mainly found along mid- to lower-slope areas (<50 m^2) to large retrogressive failures reaching over 50,000 m^2 in size. Several additional landslides are located immediately beyond the area investigated as part of the current assessment.
- Landslide activity status estimated from geomorphic indicators and/or from LiDAR data mostly range from landslides showing activity sometime in the last 5 years to landslides that are about 200 years old. The two large retrogressive landslides south of the Site are expected to date as far back as several thousand years, potentially under different geomorphological conditions.
- Four landslides ranging in size from 800 m^2 to approximately 2,600 m^2 are present on the south-facing slope immediately below from the Site. These landslides extend from the crest of the slope, down to the unnamed creek bed. Other smaller failures (50 m^2 to 300 m^2) are present; however, these are apparently shallower and restricted to the base of the slope, immediately alongside the unnamed creek.
- Longitudinal cross sections through mapped landslides units (i.e., from the head scarp to the toe bulge position) suggest overall displacement depth in the order of 1 to 5 m (example on Figure 3-2). Cross sections across the retrogressive landslides south of the Site (example on Figure 3-3) suggest spread-type failures may have reached a minimum depth of approximately 10 m.

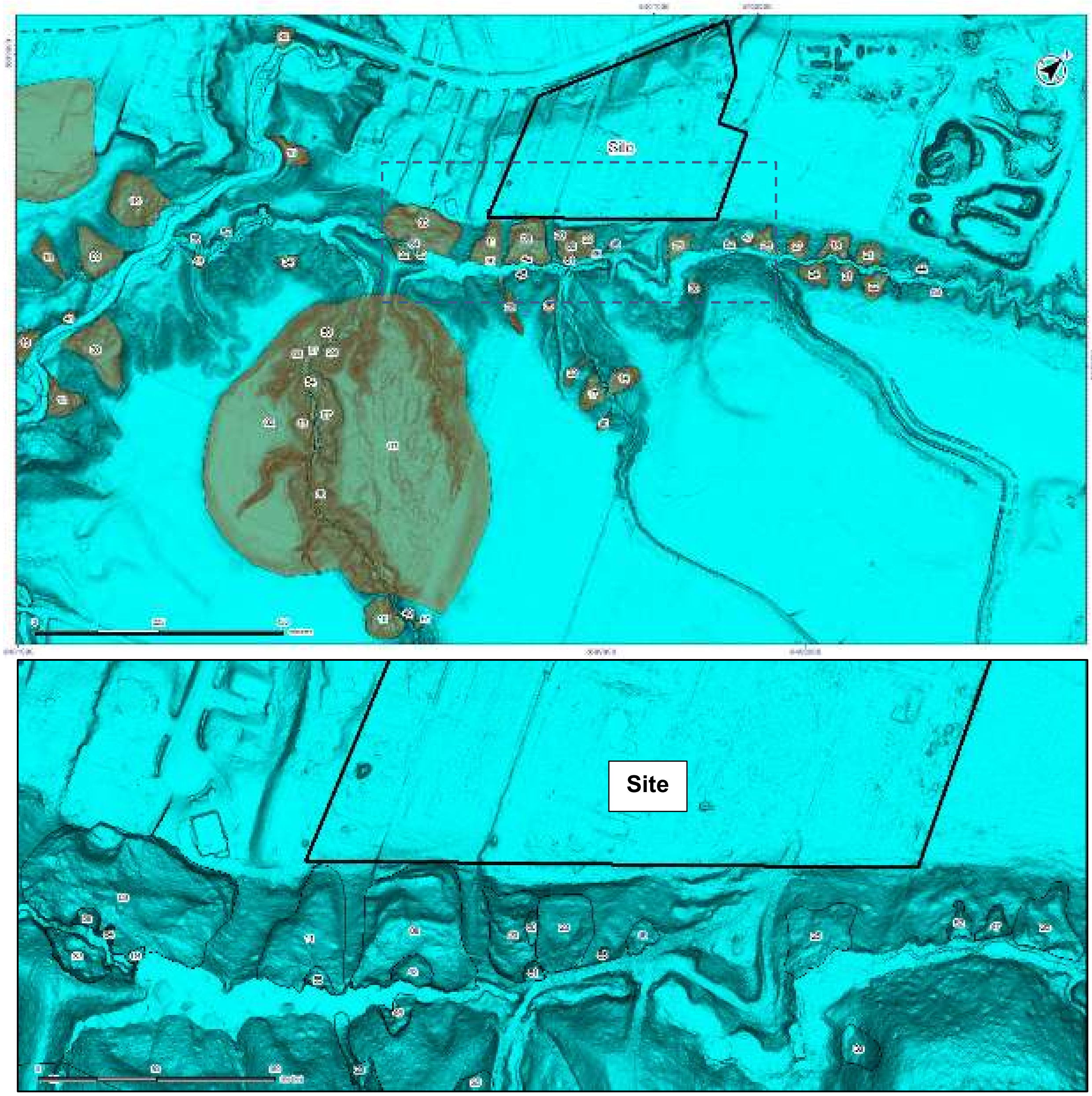


Figure 3-1 Old Montreal Road Landslide Inventory

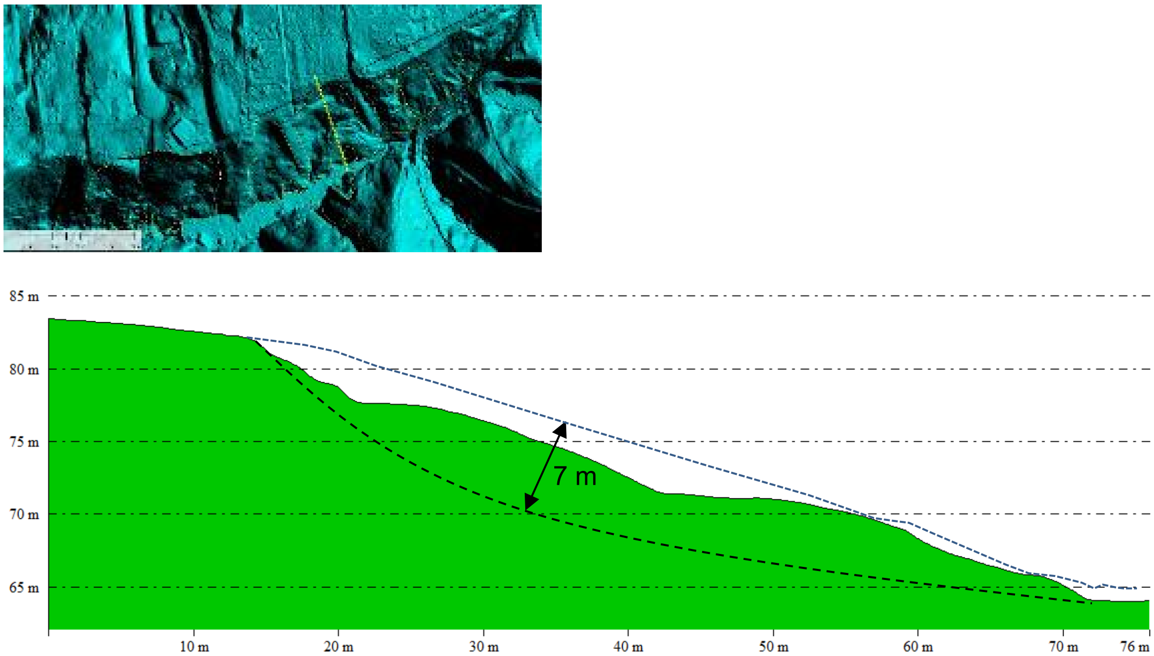


Figure 3-2 Topographic cross section across rotational landslide south of the Site

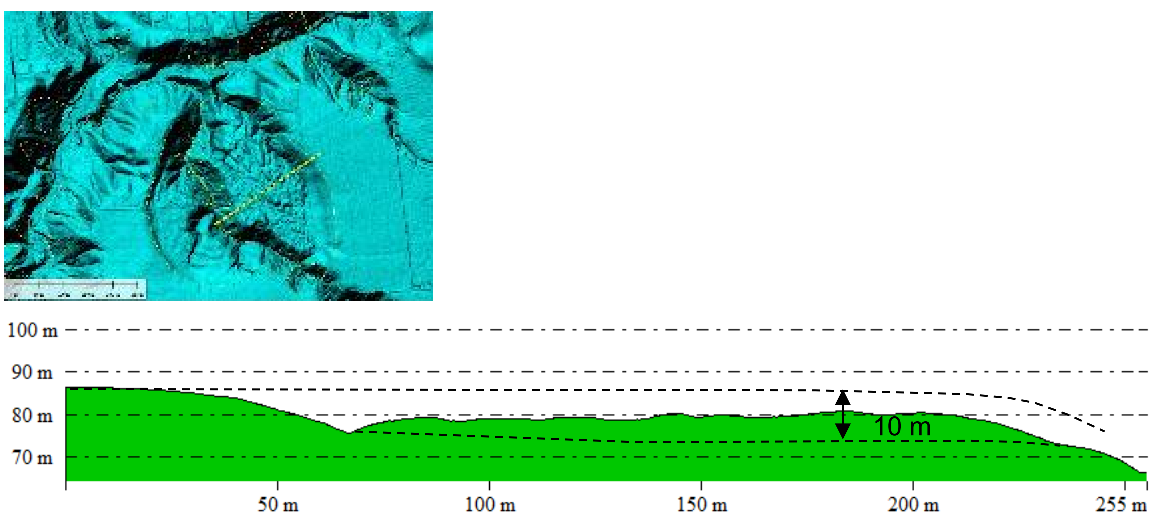


Figure 3-3 Topographic cross section across retrogressive landslide 400m south of the Site

3.2.2 LiDAR Change Detection

The availability of three distinct LiDAR dataset allowed for the identification of topographic changes within the Project Area. Visual representations of the DoD produced from comparing the 2006, 2014 and 2019 LiDAR DEMs are presented in Figure 3-4 to 3-6. On these figures, areas of positive elevation changes are shown in shades of blue, and areas of negative elevation changes are shown in shades of red and yellow.

A summary of identified topographic changes is presented below:

- Negative topographic changes associated with the initiation, reactivation or increased activities of existing landslides. Of interest, a landslide showing topographic changes indicative of ground movement was observed approximately 100m outside the southwest boundary of the Site (see Figure 3-5 and 3.6a). Another landslide showing recent movement is located 200m south of the Site (see Figure 3.6b).
- Positive and negative topographic changes associated to erosion/sedimentation processes along the different creeks and gully, including positive change associated to the accumulation of standing water due to a beaver dam causing localized flooding along the unnamed creek (see Figure 3-5).
- Other positive and negative topographic changes related to anthropogenic activities (e.g., earthworks and construction activities).

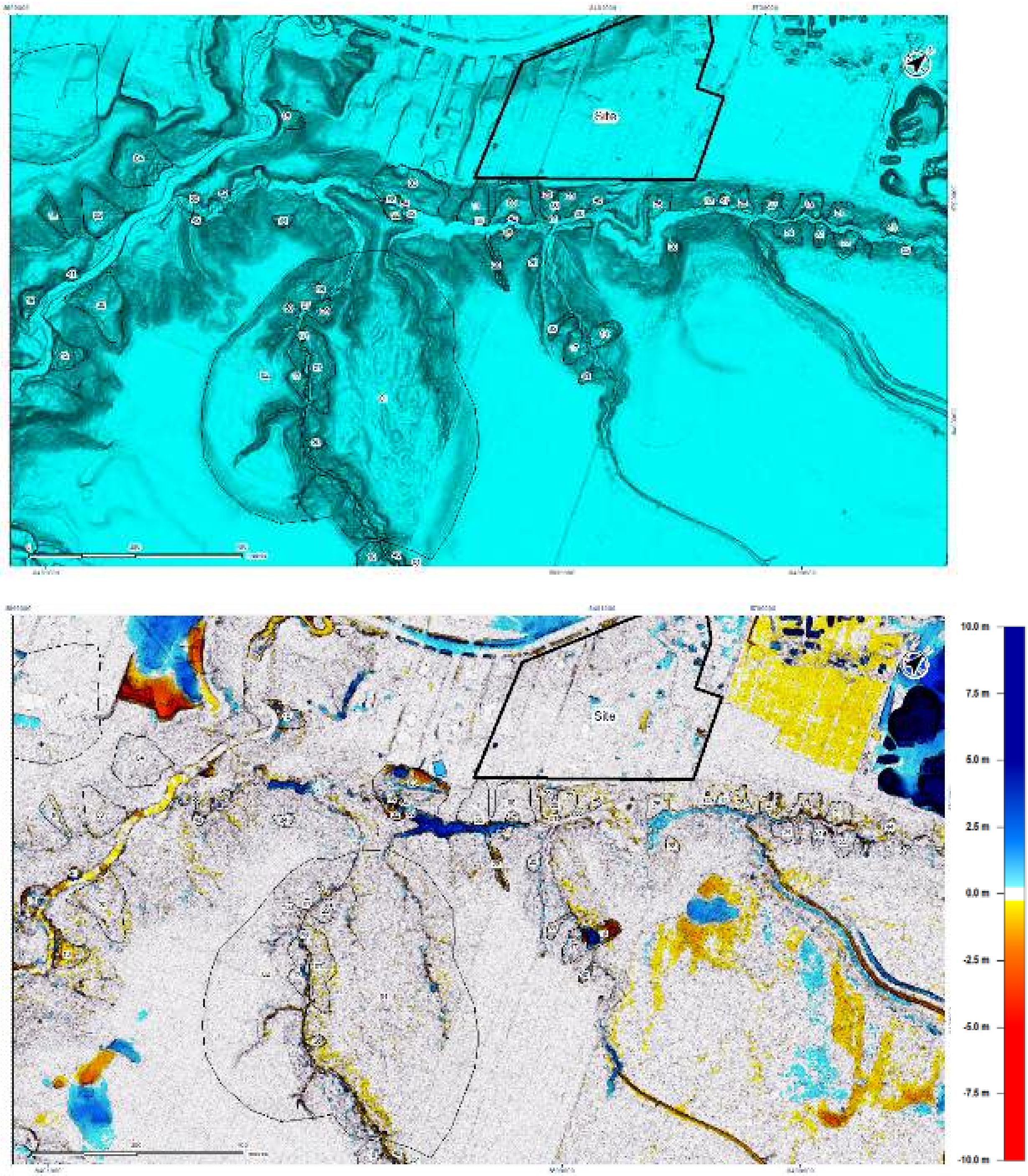


Figure 3-4 Overview of 2006-2019 LiDAR Change Detection

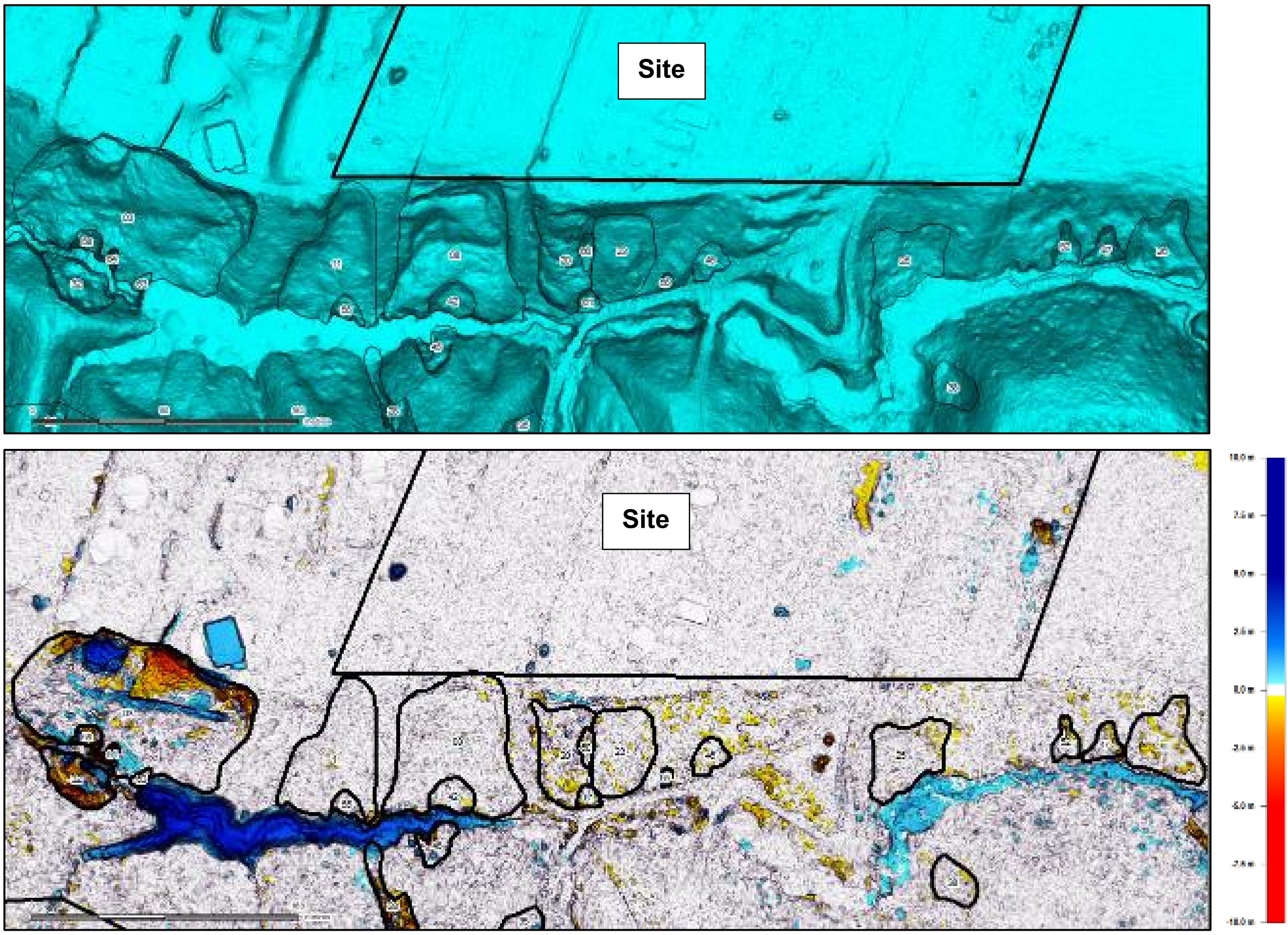
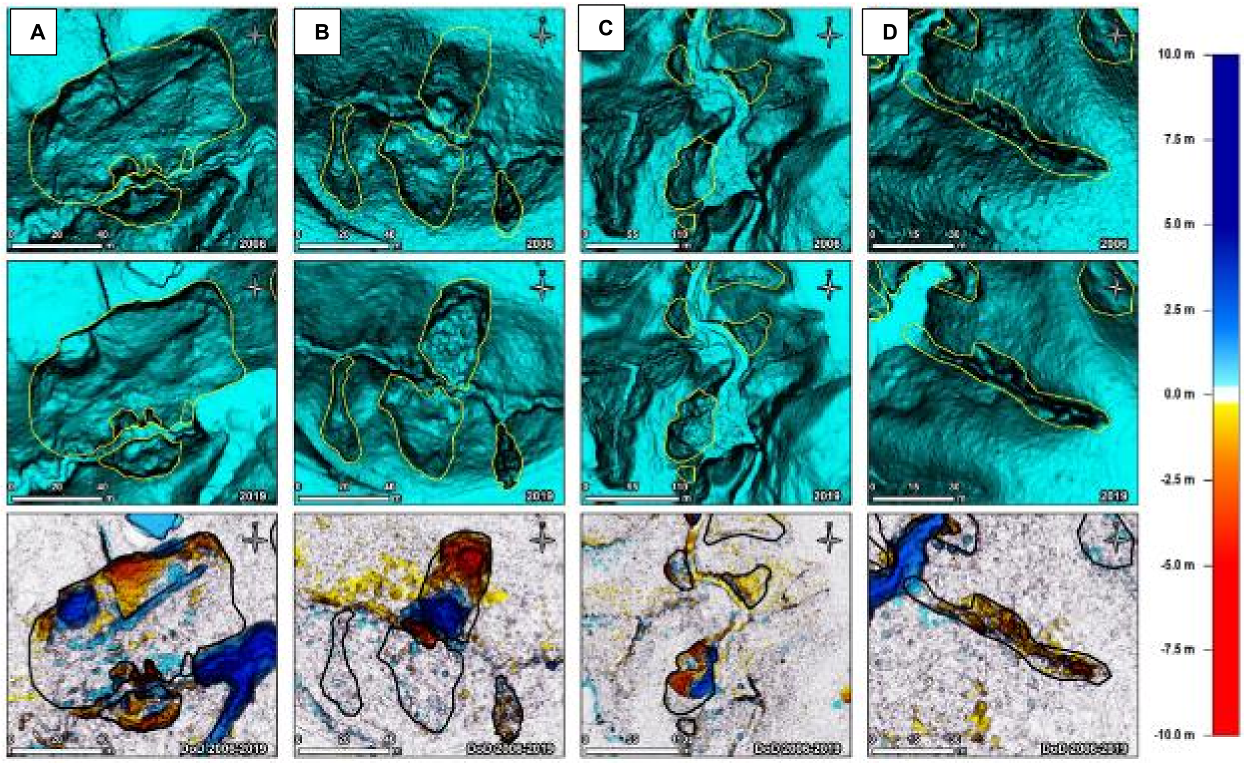


Figure 3-5 Recorded ground movements along the south-facing slope

Notes: LiDAR suggest some ground movements within a landslide some 100 m southwest of the site. Three recent failures are present in that same area. The “blue” color along the stream channel, upstream of from the recent landslides corresponds to standing water that has accumulated due to the presence of a beaver dam. Note that some of the negative changes are assumed related to changes in vegetation density (e.g., the “yellow” areas along the south-facing slope), and therefore not representative of actual ground movements.



Notes:

A	Group of landslides showing recent movement (same as displayed in Figure 3.5)
B	Reactivation of a landslide along a tributary gully 200m south of the Site
C	Example of landslides showing recent movement (Cardinal Creek area)
D	Small but active gully that has develop in the clay deposits along a slope across from the site.

Figure 3-6 Example of other landslides showing recent movements (2006 – 2019)

4.0 FIELD INVESTIGATION

4.1 SITE RECONNAISSANCE

A reconnaissance site visit was completed by two geohazard specialists (i.e., a geomorphologist and a geotechnical engineer) with previous experience conducting field investigations of clay landslides. Key goals of this visit were to:

- Ground truth key findings of the desktop interpretation (e.g., confirm surficial geology and geomorphological processes),
- Identify potential signs of ground movements postdating the imagery and LiDAR data used to conduct the terrain mapping,
- Collect data on identified landslides and other geoprocesses (e.g., flooding and gullying),
- Identify locations for subsequent geotechnical drilling.

While the visit focused on unstable/potentially unstable terrain located downslope from the proposed subdivision (i.e., the south-facing slope), special attention was given to visit nearby slope segments showing recent (<20 years) movements (e.g., across from the unnamed creek and the proposed subdivision). Information on surficial materials was gathered by examining soil exposures within tributary gullies, along the streambed as well as along road cuts. These opportunistic observations were complemented by a few shallow test pits and auger hole conducted using manual tools. Material textures were visually assessed and classified based on particle size. Signs of mass movements were documented

A map displaying the location of foot traverses conducted as part of the site visit is presented in Figure 4-1. Select photographs taken during the site visit are also presented.

4.2 SUMMARY OF FIELD OBSERVATIONS

During this site reconnaissance, special attention was given to record indicator slope instabilities (either past or recent). These indicators were related to one or more of the following factors: slope morphology, surficial materials, vegetation and drainage condition.

A summary of key observations made as part of the site reconnaissance is presented below:

- It was possible to visually identified all previously mapped landslides while in the field, which confirmed the added value of using bare-earth LIDAR as part of the assessment. Although an often-dense vegetation was present within the gully system, the distinct topography of landslides' head scarps, slide blocs and/or undulating to hummocky deposit was still recognizable.
- The majority these features showed no obvious recent signs of mass movements (e.g., tension cracks and active scarp faces denuded of any vegetation); however, the overall topography, vegetation and drainage conditions did inform on a few unstable slope sections.
- Observed indicators of previous slope instability include:

  - Subtle change in drainage and vegetation conditions including poorly drained lower slope position
  - Poorly-define scarp faces and step-like benches, pressure ridges and curved or concave slope sections
  - Exposed soil along slopes along small gullies and stream channels
  - Debris piled on lower slopes including mixed and buried soil profiles
  - Partially revegetated landslide scars and scarp faces
  - “Jack-strawed” trees and trees tilted in various directions.
- The absence of large trees in several of the smaller landslides and the presence of “dead standing” trees at the toe of slope suggest these smaller landslides may be 20-50 years old. At several locations, “jack-strawed” or tilted trees were observed, further suggesting that some movement occurred as the trees were growing.
- No signs of recent movements were observed along the few larger landslides located immediately south of the Site (i.e., on the south-facing slope above from the unnamed creek), where poorly distinct landslide scars and mature (~30cm diameter) hardwood trees were observed to grow immediately below or within former landslide scarps. Analysis of LiDAR data and field observations suggest those landslides are in the 50 to 100 years range or more.
- A landslide located along the south-facing slope immediately west of the Site did show signs of recent activities (i.e., <20 years), which coincide with the findings of the LiDAR change detection analysis (i.e., ground movement during the 2006-2019 period. Field observations include the presence of a poorly developed step-like bench and unvegetated head scrap (~1m in height) near the top of the slope, much younger vegetation than adjacent similar area as well as undulating toe slope and mixed soil profiles.
- Another recent landslide was observed along the tributary gully some 200 m south of the Site. The feature consisted of a small flowslide that has initiated immediately below an existing access trail. This landslide, approximately 145 m^2 in size did show signs of active movement along it toe (mainly soil creep), likely influence by the slow but progressive removal of colluviated material by fluvial erosion.
- Only marine soils, colluviated marine soils and fluvial materials were observed in the field. No till material, nor bedrock exposures were observed at the Site, including at the bottom of the gully system.
- An inactive beaver dam was observed at the base of the gully system along the unnamed creek. This beaver dam, visible on the 2019 LiDAR data, was observed to be 15 m wide and up to 2.5 m in height. No water was present upstream from the dam; however, observations of water mark on trees suggested previous flooded reaching up to 150 m upstream. These observations coincide with the findings of the LiDAR change detection analysis, suggesting that the area (approximately 2,000 m^2 in size) was flooded sometime between 2014 and 2019.

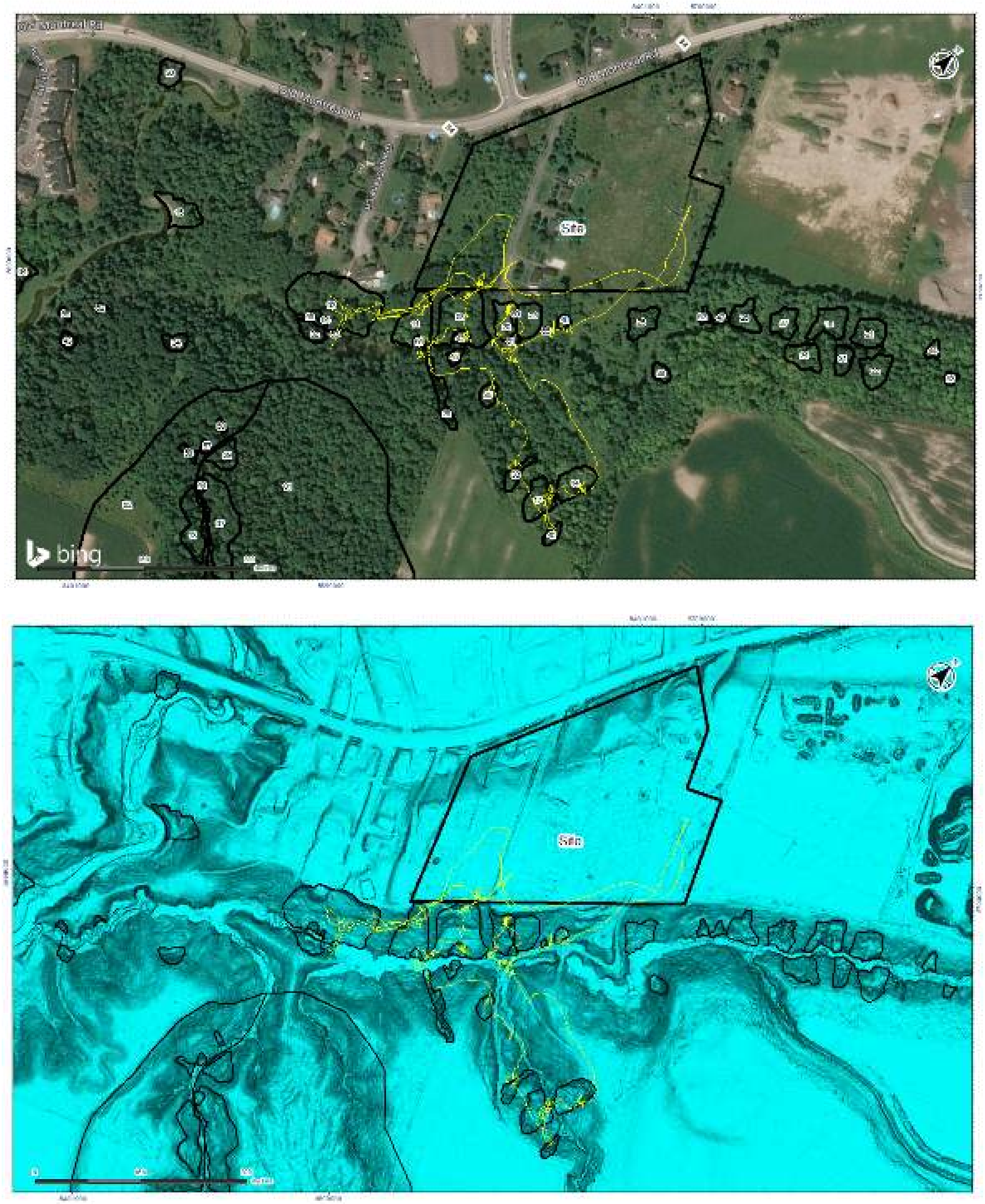


Figure 4-1 Field reconnaissance transect and observed landslide units


Photo 1: View of upper limit of the south-facing slope at the edge of the plateau (looking west, the slope is on the left).


Photo 2: Downslope view of landslide ID 11. Approximate landslide limit in white.


Photo 3: Poorly defined headscarp at landslide ID 11.


Photo 4: Undulating to hummocky terrain along the base of landslide ID 55. Movement from right to left.


Photo 5: Lateral view of stable headscarp at landslide ID 08. The feature matches the crest of the south-facing slope along the southwest limit of the Site.


Photo 6: View of a section of the headscarp observed at landslide ID 03, in the area showing movement on the LiDAR change detection.

Photo 7: View of the headscarp (white) at landslide ID 14. The landslide is less than 15 years and showing signs of recent movement.

Photo 8: Small landslide (ID 42) along the unnamed creek at the based of the south-facing slope (looking upstream).

Photo 9: Base of landslide ID 14 showing back-tilted trees (movement from right to left, approximate landslide limit in white).

Photo 10: Upstream view of the beaver dam across the unnamed creek.

Photo 11: Base of active gully along north-facing slope across from the site (ID 28).

Photo 12: Upstream view of previously flooded area at the base of gully system (south-facing slope on the left side). Note: Maximum water level visible from tree bark.

4.3 SUPPLEMENTARY GEOTECHNICAL INVESTIGATION

Reviewing the EXP updated preliminary geotechnical report available for the Site indicated that the available information was limited to screen the soil deposit for the purpose of determining the risk against retrogressive failure. For this reason, Stantec recommended for a supplementary geotechnical investigation to further assess the potential risks for large retrogressive slide at the Site. Supplementary geotechnical investigation recommendations were provided to EXP on August 28, 2021, and included the following:

<u>Additional Borehole drilling:</u>

- Two geotechnical boreholes: BH-13 (Lat 45.492744°; Long -75.467035°), and BH-14 (Lat 45.493032°; Long -75.466505°). Continuous Split Spoon sampling with at least 3 undisturbed Shelby tube samples in clays should be collected. Borehole should be advanced until the refusal to bedrock.
- Install nested piezometers (standpipes) at 5 m, 10m, 15m and 20 m depth plus one in the till. It is understood that a second hole will be drilled adjacent to BH 13 to accommodate multiple piezometers. Stantec prefers to install vibrating wire piezometer (nested) instead of standpipe.
- Complete field shear vane tests adjacent to the proposed borehole using Nilcon vane apparatus. Determine peak shear strength for every meter and remolded shear strength for every 2 m interval to obtain the strength profile along the depth.
- Complete one Cone Penetration Tests (CPT-1) adjacent to proposed borehole BH-13 and field vane test locations. The CPT data will be used to interpret and compare the information obtained from borehole and vane tests.
- Two additional CPT tests were proposed: CPT-2 (Lat 45.493032°; Long -75.466505°) and CPT-3 (Lat 45.493273°; Long -75.465902°).

<u>Lab Testing:</u>

- Moisture content test, peak and remolded shear strength tests (using Swedish fall cone) for all (continuous) samples collected. This will provide water content and strength profile throughout the depth.
- Atterberg limit tests for at least 6 samples collected per borehole.
- Gradation including hydrometer tests for 2 samples per borehole.

As summarized above, two boreholes, two Nilcon vane holes and three soundings (CPTs) were initially proposed for the supplementary geotechnical investigation for a large-scale retrogressive slide risk evaluation as shown on Figure 4-2. It is our understanding that borehole BH-13 and CPT-1 were cancelled due to site access restriction (vegetation conditions). The coordinates and elevations of borehole BH-14 and CPTs-2 and 3 are provided in Table 4-1. The table includes details on piezometers installed in borehole BH-14.

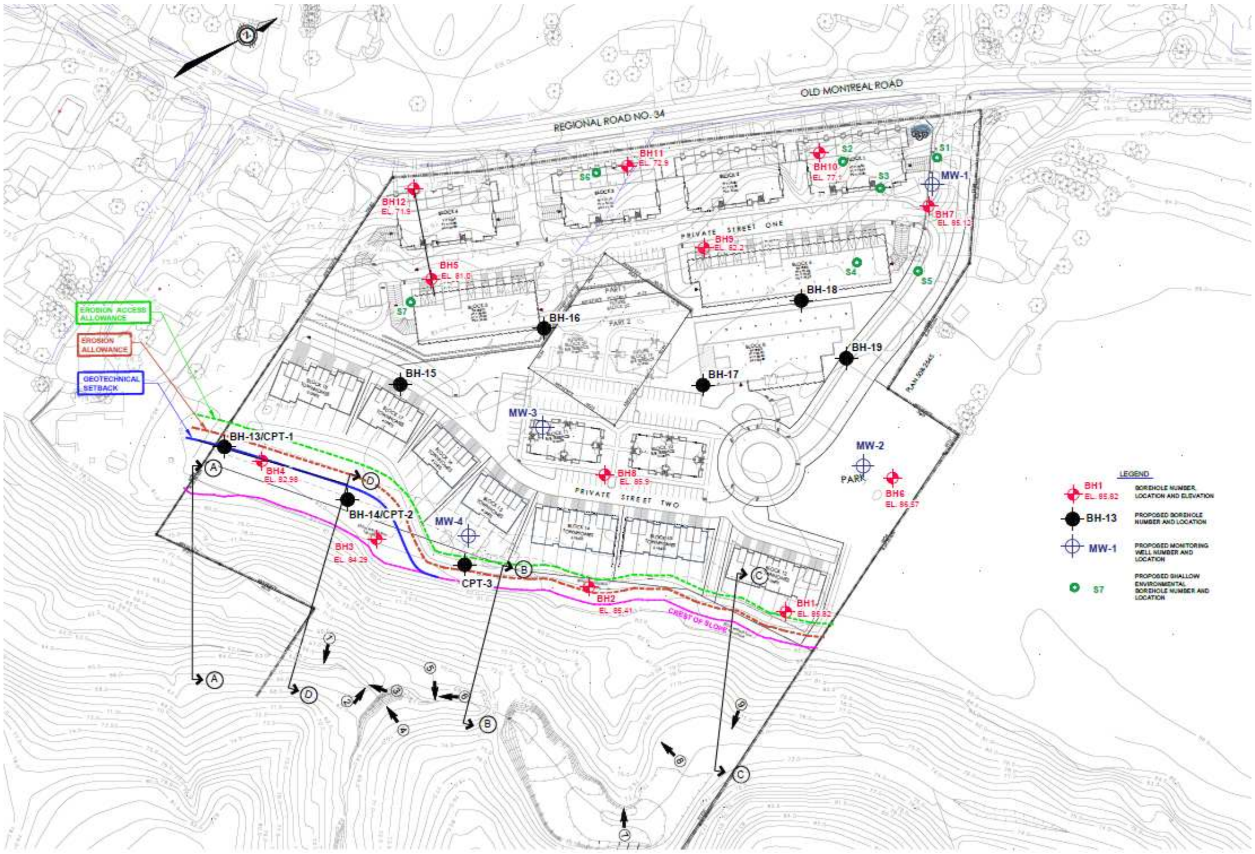


Figure 4-2 Supplementary Geotechnical Investigation Location Plan (drawing by EXP)

Table 4-1 Supplementary Geotechnical Investigation Borehole, Sounding and Piezometer Locations and Elevations

Point No.	Northing (m)	Easting (m)	Elevation	Remarks
BH-14/CPT-2	5039762	385595	84,04	-
CPT-3	5039788	385623	82,10	-

BH	Elevation	WL Depth	WL Elev	Remarks Depth of Slotted Portion/Sand Pack and Seal
BH14A	84,04	3,24	80,80	16 ' depth, Screen 13'-16', Sand 1' above/below, bentonite to top
BH14B	84,04	5,39	78,65	33' depth, Screen 30'-33', 1' sand above/below, bentonite to 17'
BH14C	84,04	15,13	68,91	50' depth, Screen 47'-50', 1' sand above/below, bentonite to 34'
BH14D	84,04	19,46	64,58	65' depth, Screen 62'-65', 1' sand above/below, bentonite to 51'
BH14E	84,04	19,84	64,20	75' depth, Screen 72'-75', 1' sand above, bentonite to 66'

Borehole BH-14 encountered the following subsurface condition (see borehole log in Appendix C);

- Granular fill underlain by;
- Clay crust underlain by;
- Massive clay underlain by; and
- Gravelly sand till.

Borehole BH-14 was terminated at auger refusal at a depth of 23 m below ground surface. CPT2 was carried out in the vicinity of Borehole BH-14 and a similar stratigraphy was identified. CPT 3 was carried out about 50 m east of the CPT 2 and also encountered a similar stratigraphy.

Continuous undrained shear strength and remolded strength profiles were obtained from two CPT soundings based on the cone tip and sleeve friction resistances. Figure 4-3 and Figure 4-4 below present results obtained at CPT 2 and CPT 3.

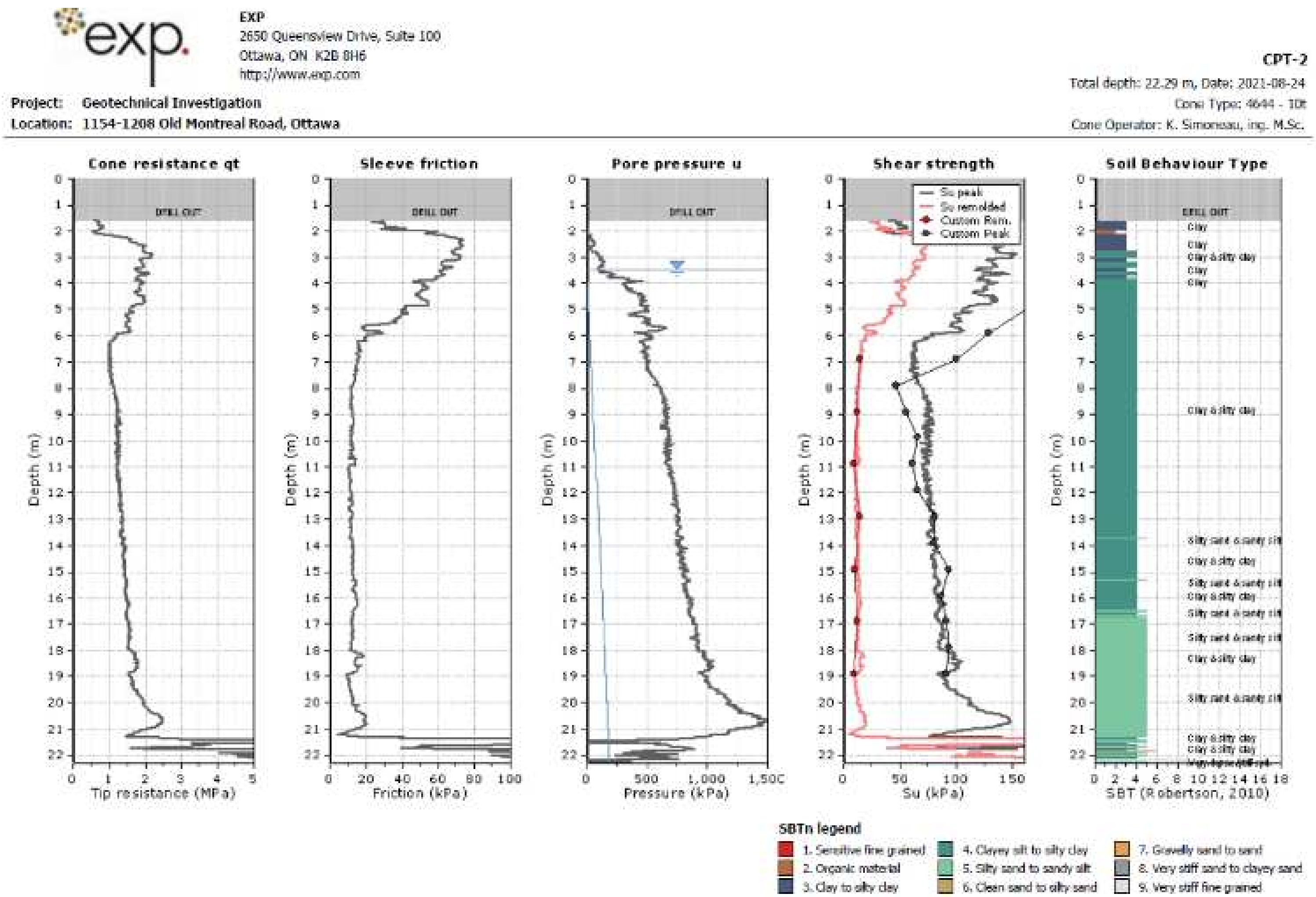


Figure 4-3 CPT2 data plot

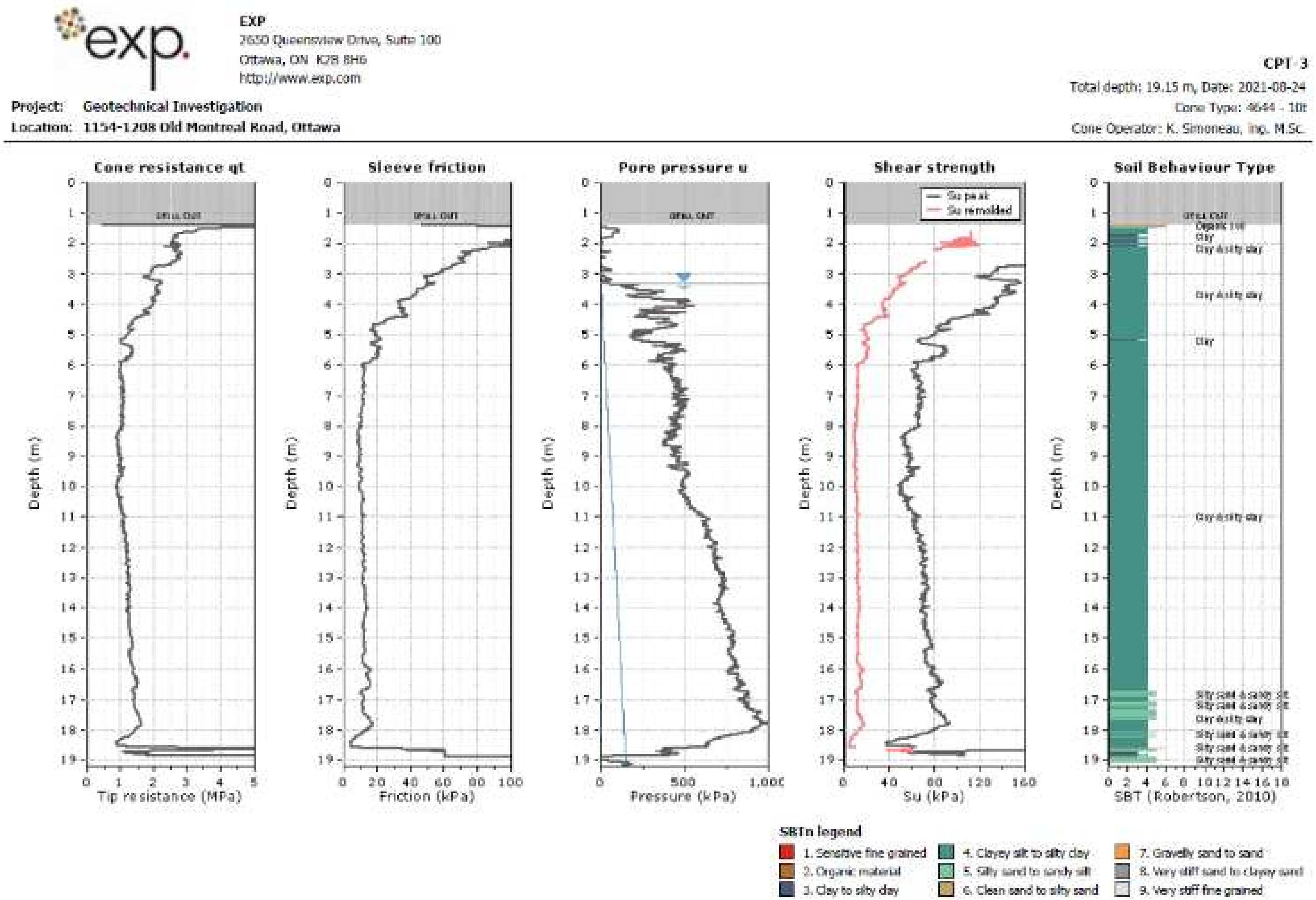


Figure 4-4 CPT3 data plot

Both CPTs have very similar trend and interpreted undrained and remolded shear strength parameters are as below (Table 4-2).

Table 4-2 CPT summary

CPT #	Soil Layer	Average Undrained Shear Strength (KPa)	Average Remolded Shear Strength (KPa)	Average Sensitivity
2	Clay Crust	110	50	2.3
	Massive Clay	80	12	6.2
3	Clay Crust	120	60	2.1
	Massive Clay	70	12	5.6

A series of Nilcon vane test was also performed in the vicinity of Borehole BH-14 mostly in the massive clay layer and measured undrained and remolded shear strength are summarized in Table 4-3.

Table 4-3 Shear Strengths

exp.

RÉSISTANCE AU CISAILLEMENT NON-DRAINÉ

NQ-2501-200

DOSSIER:	OTT-234493	Grandes ailettes:	c = 0,05	C_u=9,80665 x K x c x (M_s - M_f)	Indice de sensibilité
PROJET:	1208 Old Montreal Road	Moyennes ailettes:	c = 0,1	K= 1.192	
CLIENT:	Unknown	Petites ailettes:	c = 0,2		

Forage no	Profondeur (m)	c	M_f (mm)	M_s (mm)	M_s - M_f (mm)	C_u (kPa)	M_{fr} (mm)	M_{sr} (mm)	M_{sr} - M_{fr} (mm)	C_{ur} (kPa)	C_u / C_{ur}
BH-14	4.90	0.2	12	83	71.0	166.0					
	5.90	0.2	16	71	55.0	128.6					
	6.90	0.2	9	52	43.0	100.5	7	13	6.0	14.0	7.2
	7.90	0.2	7	27	20.0	46.8					
	8.90	0.2	12	36	24.0	56.1	7	12	5.0	11.7	4.8
	9.90	0.2	15	43	28.0	65.5					
	10.90	0.2	14	40	26.0	60.8	17	21	4.0	9.4	6.5
	11.90	0.2	16.5	44.5	28.0	65.5					
	12.90	0.2	18.5	53	34.5	80.7	19	25	6.0	14.0	5.8
	13.90	0.2	20	54	34.0	79.5					
	14.90	0.2	23	63	40.0	93.5	16	20.5	4.5	10.5	8.9
	15.90	0.2	24	61	37.0	86.5					
	16.90	0.2	29	68	39.0	91.2	20	25	5.0	11.7	7.8
	17.90	0.2	32	72	40.0	93.5					
	18.90	0.2	32	71	39.0	91.2	23	27	4.0	9.4	9.8

The average undrained and remold shear strengths of massive clay measured by Nilcon vane are 78 KPa and 12 kPa and average sensitivity is about 7 which are quite comparable to the CPT findings.

Swedish fall cone tests were also carried out to measure undrained and remolded shear strengths of split spoon soil samples and Shelby tube samples retrieved from Borehole BH14 and measurements are summarized in Table 4-4. The average undrained and remolded shear strengths of massive clay are about 80 kPa and 5.5 kPa which are also comparable to CPT and Nilcon vane test results.

Table 4-4 Swedish Fall Test Results

SWEDISH FALL CONE TEST

Project Name	Exp. File# OTT-234493-AO	**Borehole No.**	BH 14
Project No.	121623638	**Sample No.**	
Location	1208 Old Montreal Road,Ottawa,ON	**Depth**	

Test No.	Cone Mass (g)	Cone Angle (deg)	K	Cone Penetration (mm)	Undrained Shear Strength (kPa)	Comments
1	400	30	0.10	1.0	3920.0	BH14 ST1@ 10-12ft-Intact
2	60	60	0.03			BH14 ST1@ 10-12ft-Can't remold (Pocket Pen > 4.5)
3	400	30	0.10	3.0	435.6	BH14 ST2@ 20-22ft-Intact
4	60	60	0.03	2.0	44.1	BH14 ST2@ 20-22ft-Remold
5	60	30	0.10	6.0	16.3	BH14 SS12@ 26-28ft-Remold
6	400	30	0.10	6.5	92.8	BH14 ST3@ 30-32ft-Intact
7	60	60	0.03	5.0	7.1	BH14 ST3@ 30-32ft-Remold
8	60	60	0.03	6.0	4.9	BH14 SS16@ 36-38ft-Remold
9	400	30	0.10	8.0	61.3	BH14 ST4@ 40-42ft-Intact
10	60	60	0.03	5.0	7.1	BH14 ST4@ 40-42ft-Remold
11	60	60	0.03	5.0	7.1	BH14 SS19@ 44-46ft-Remold
12	60	60	0.03	6.0	4.9	BH14 SS21@ 48-50ft-Remold
13	60	60	0.03	5.0	7.1	BH14 SS23@ 52-54ft-Remold
14	60	60	0.03	6.0	4.9	BH14 SS25@ 56-58ft-Remold
15	60	60	0.03	5.0	7.1	BH14 SS27@ 60-62ft-Remold
16	60	60	0.03	10.0	1.8	BH14 SS29@ 64-66ft-Remold
17	60	60	0.03	12.0	1.2	BH14 SS31@ 68-70ft-Remold

Tested By:	Daniel Boateng	Date:	08-Sep-21	Checked By:	R. Hache	Date	8-Sep-21

The measured natural moisture contents of the clay crust and massive clay are about 45% and 65%, respectively.

Six (6) Atterberg limit tests were carried out on split spoon samples and Shelby tube samples retrieved from Borehole BH-14 and results are presented in Figure 4-5. Average liquidity index about 1 is obtained for the massive clay.

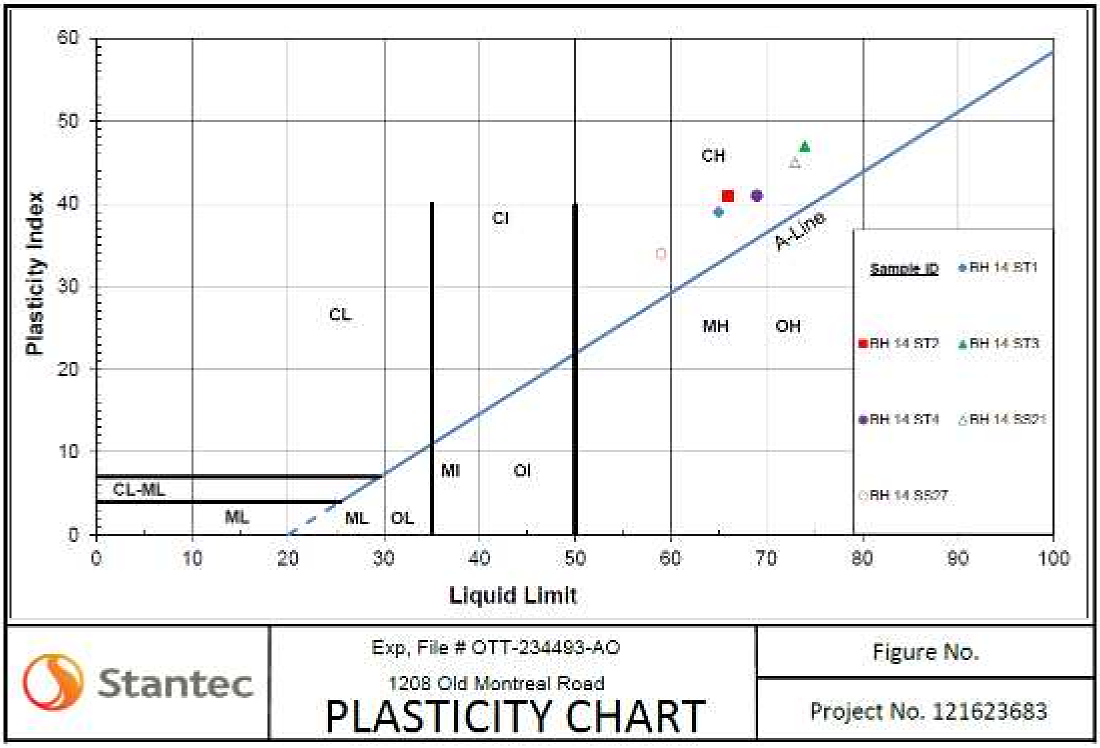


Figure 4-5 Atterberg Limit Test Results

The supplementary geotechnical investigation findings are comparable to the EXP updated preliminary geotechnical investigation findings which are summarized in Table 4-5.

Table 4-5 EXP Updated Preliminary Geotechnical Investigation Findings

Soil Layer / Soil Parameters	Clay Crust	Massive Clay	Remarks
Average Natural Moisture Content (%)	40 to 50	60 to 70	
Average Liquid Limit (%)	68	63	
Average Liquidity Index	0.5	1.0	
Average Undrained Shear Strength (kPa)	190	100	
Average Remolded Shear Strength (kPa)	35	19	

All above summarized soil parameters will be used for the further slope stability analyses and retrogressive landslide risk assessment.

5.0 STABILITY ANALYSIS

5.1 METHOD

As describe earlier, slopes segments found within the gully system south of the proposed development have historically experienced movements and instabilities. Although no active slope movements were noted along the south-facing slope immediately below from the proposed development area, some recent landslide activity was observed in the area.

The City of Ottawa guidelines suggests a Factor of Safety (FOS) of 1.5 and 1.1 under static and pseudo-static (i.e., for the 2,475 years return period earthquake) conditions for slopes associated with developments, respectively. Due to the presence of sensitive clay, and recognized potential for large retrogressive landslide in the Ottawa Region, an empirical retrogressive landslide assessment developed by Ministry of Transportation Quebec (MTQ; Thibault et al. 2008) was also considered in addition to a conventional slope stability analysis.

A conventional limit equilibrium slope stability analysis was carried out using a commercially available slope stability analysis computer software, GeoStudio 2020. The Morgenstern-Price was used for slope stability analyses.

Multiple slope sections (see Figure 5-1) were generated using available LiDAR data to derive the scenario with consideration of subsurface and groundwater conditions. The section A-A is beyond the limits of the Site to the southwest; however, it was considered because some recent movement was identified from the LiDAR change detection analysis at this location. Section E-E' presented a slightly higher slope; however, the slope angle was relatively flatter than other slope sections in the area. As shown in Figure 5-1 below, the remaining slope sections (B-B', C-C' and D-D') are almost identical, and section C-C' was selected for the analysis. The section C-C' was analyzed for short-term (total stress condition), long-term (effective stress condition) and pseudo-static conditions.

Additional slope sections were reviewed to refine the slope set back assessment and hazard zonation (see Figure 5-2). The slope cross-sections at the proposed townhouse block 15 (profile 17) and near the east project limit (profile 25) were analyzed.

5.2 DATA ANALYSIS

The following figures and tables were developed as part of the slope stability analysis. Refer to Section 5.3 for a summary of key findings.

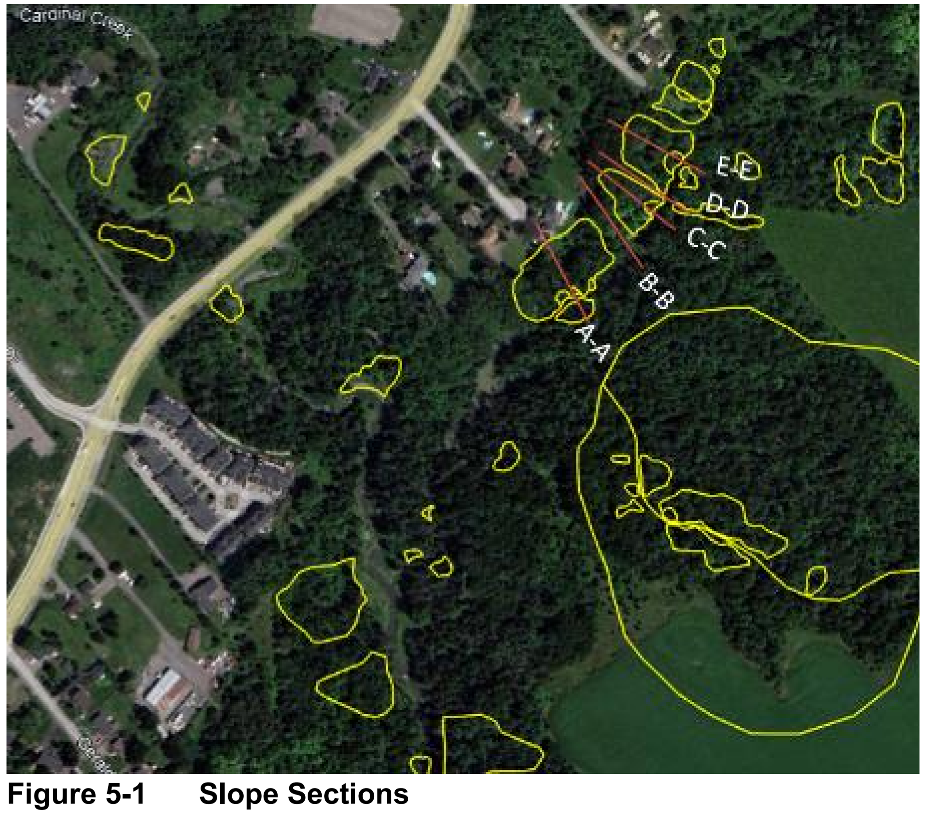
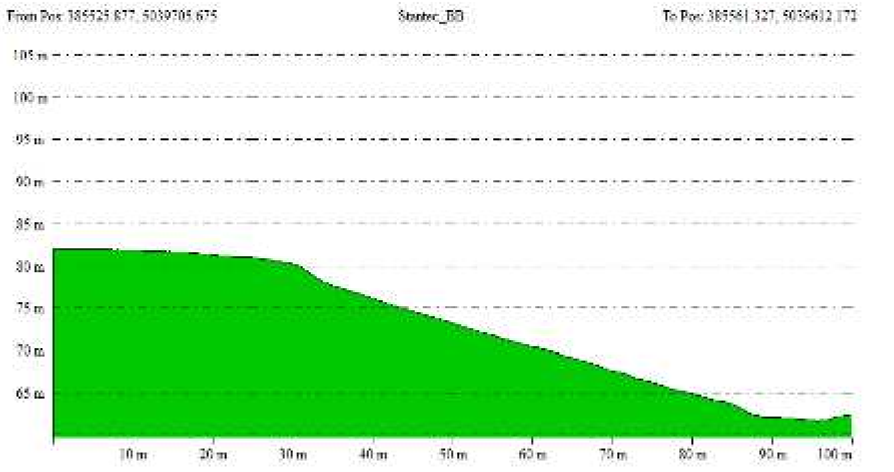
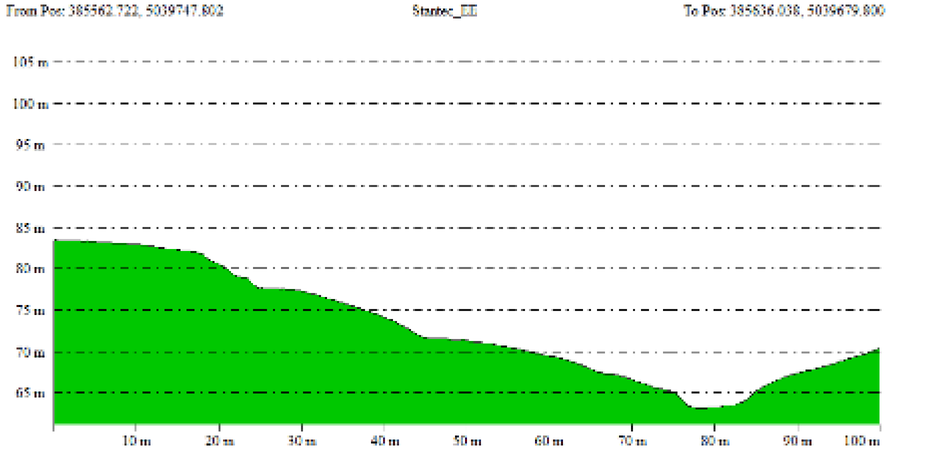


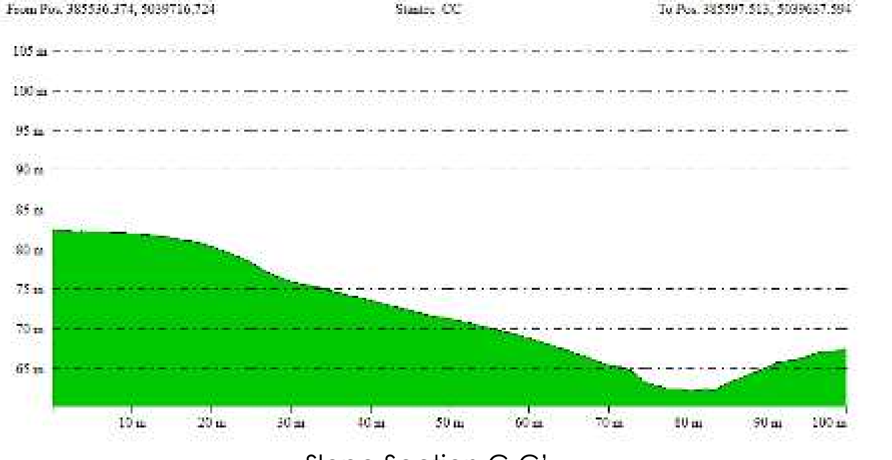
Figure 5-1 Slope Sections



Slope Section B-B'

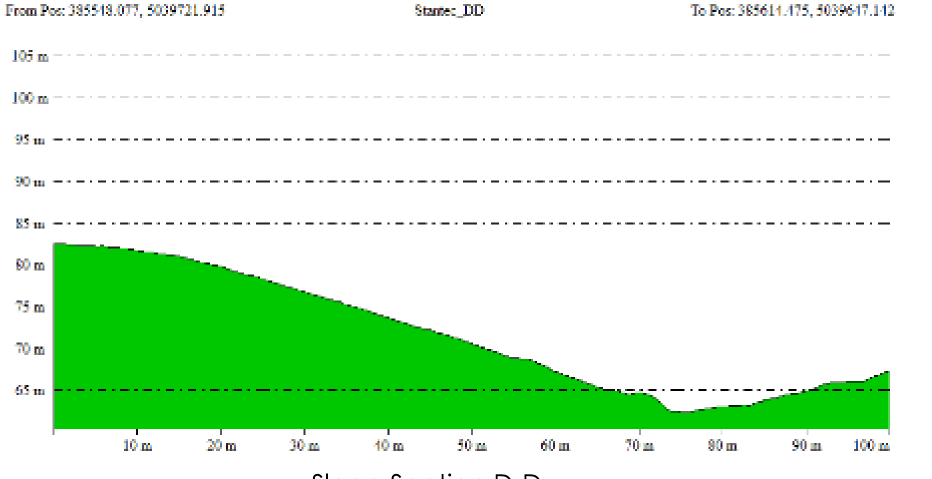


Slope Section E-E'

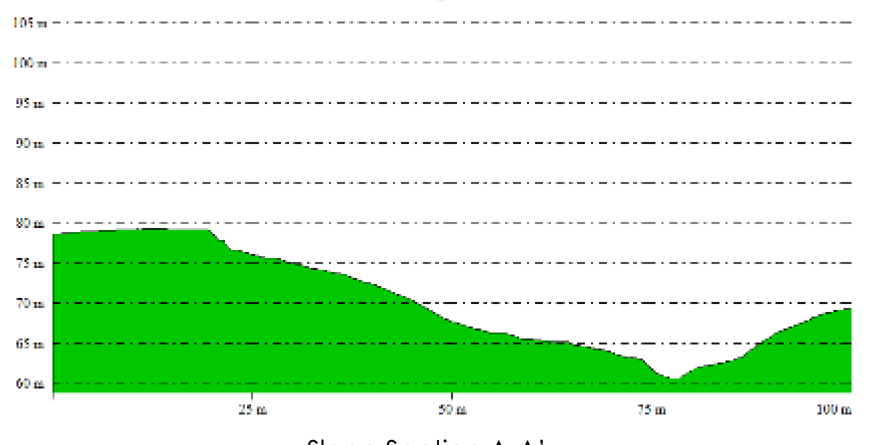


Slope Section C-C'

Overlapping of Slope Sections B-B', C-C' and D-D'



Slope Section D-D



Slope Section A-A'

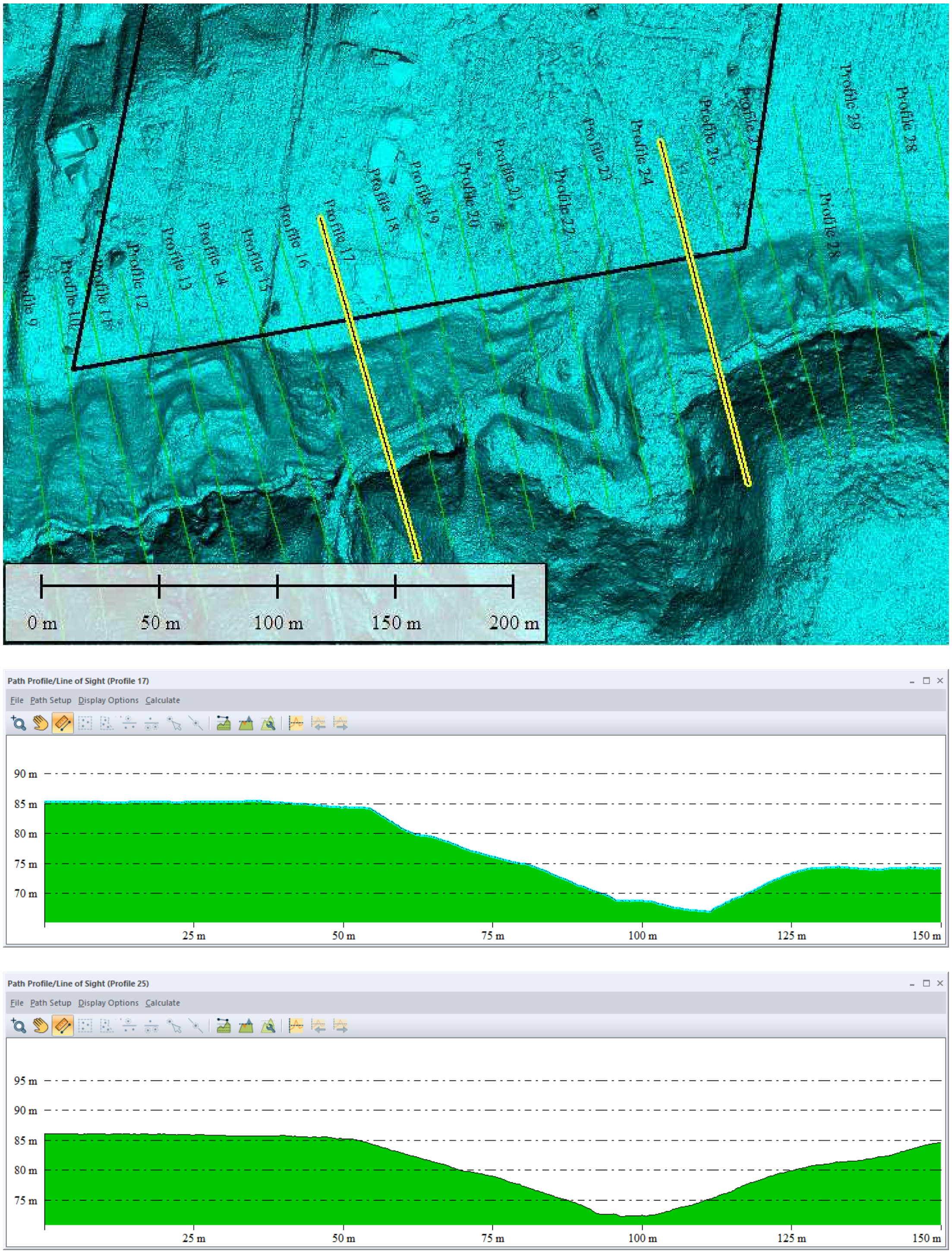


Figure 5-2 Additional Slope Sections Used to Refine the Slope Setback Assessment

The supplementary geotechnical investigation findings are considered for the slope stability analysis. A summary of soil parameters used for the slope stability analysis is summarized in Table 5-1 and Figure 5-3 to Figure 5-11.

Table 5-1 Soil Parameters Used for Slope Stability Analyses

Soil Type	Design Parameters			
	Total Unit Weight (kN/m^3)	Effective Friction Angle (degrees)[1]	Effective Cohesion (KPa)[1]	Undrained Shear Strength (KPa)[2]
Fill	20.0	32	-	-
Clay Crust	18.0	36	10	100
Massive Clay	16.5	36	7.5	70
Till	21	38	-	-

Notes: 1 These friction angle and cohesion are only applicable to drained effective stress (long-term) conditions
2 The shear strength is only applicable to undrained total stress (short term) conditions

It should be noted that undrained shear strength and drained soil were obtained from the available literature (e.g., Slope Stability Guideline For Development Applications in the City of Ottawa, OGS Miscellaneous paper 112 Slope Stability Study of the South Nation River and Portions of the Ottawa River, OGS Miscellaneous paper MP 68 Slope Stability Study of The Regional Municipality of Ottawa-Carleton, Ontario, Canada, Fourth Canadian Geotechnical Colloquium 1981: Strength and Slope Stability in Canadian Soft Clay Deposits by Guy Lefebvre).

As per the City of Ottawa Slope Guideline and observed high groundwater table, fully saturated slopes were considered for the stability analysis.

The 2,475-year return period earthquake PGA (see NBCC 2015 earthquake hazard sheet in Appendix D) was used for the pseudo-static slope stability analysis.

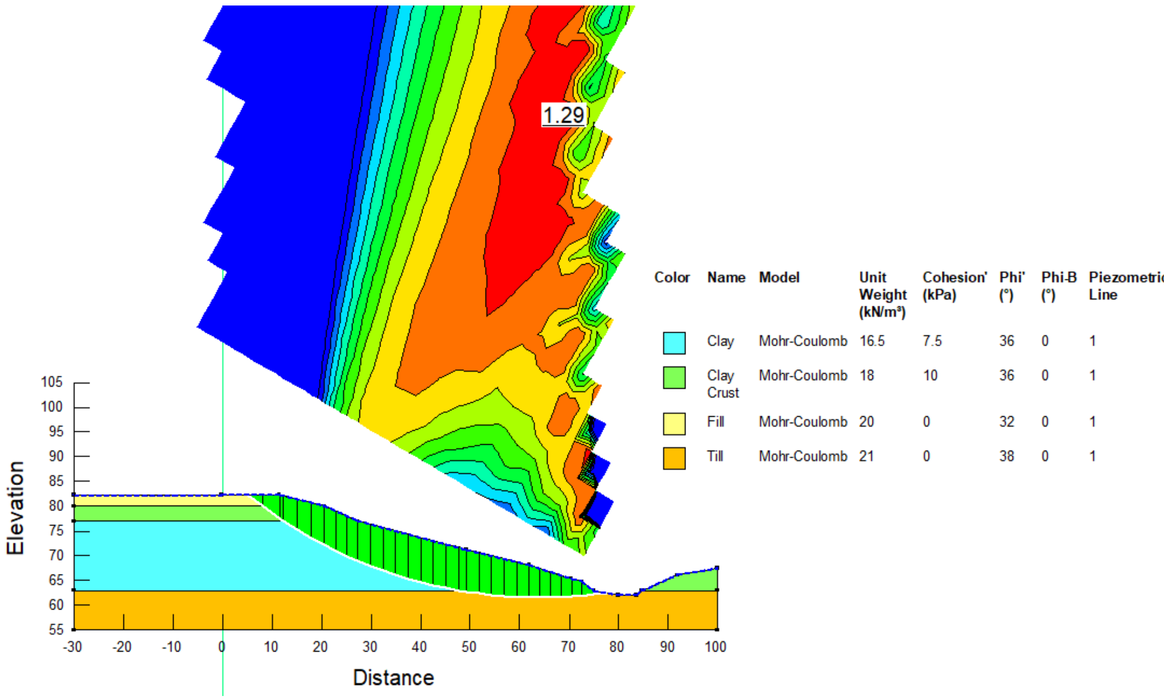


Figure 5-3 Effective Stress (Long-term) Slope Stability Analysis of Section C-C'

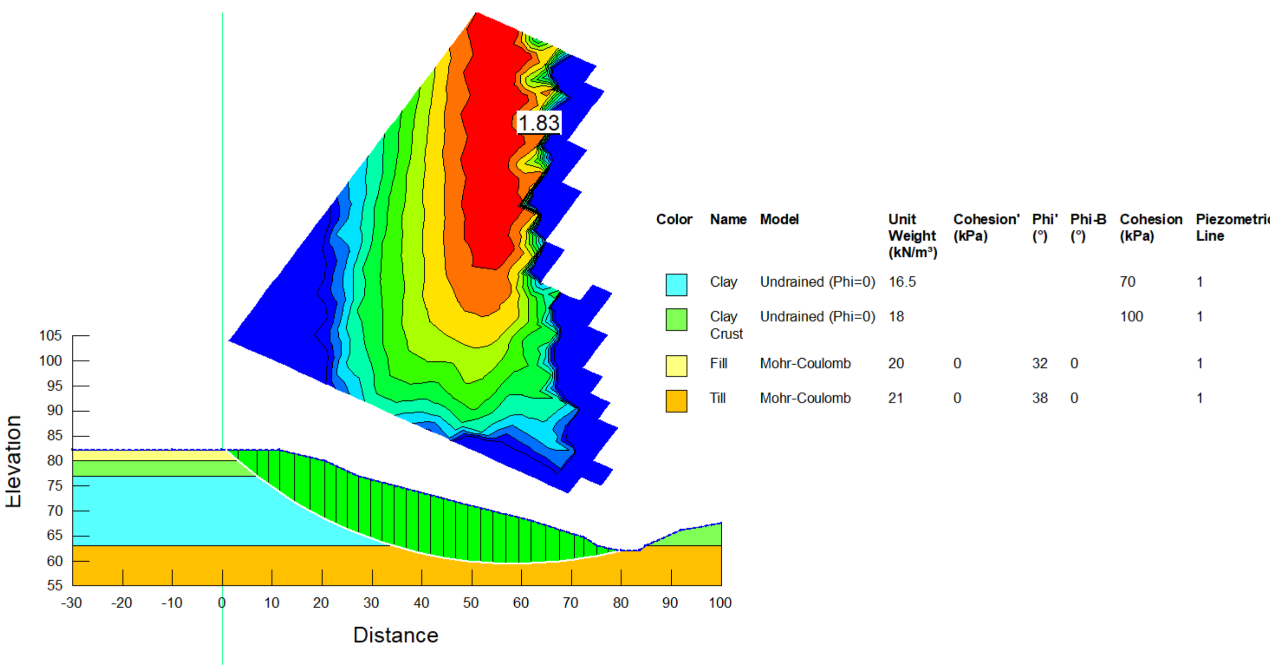


Figure 5-4 Total Stress (Short-term) Slope Stability Analysis of Section C-C'

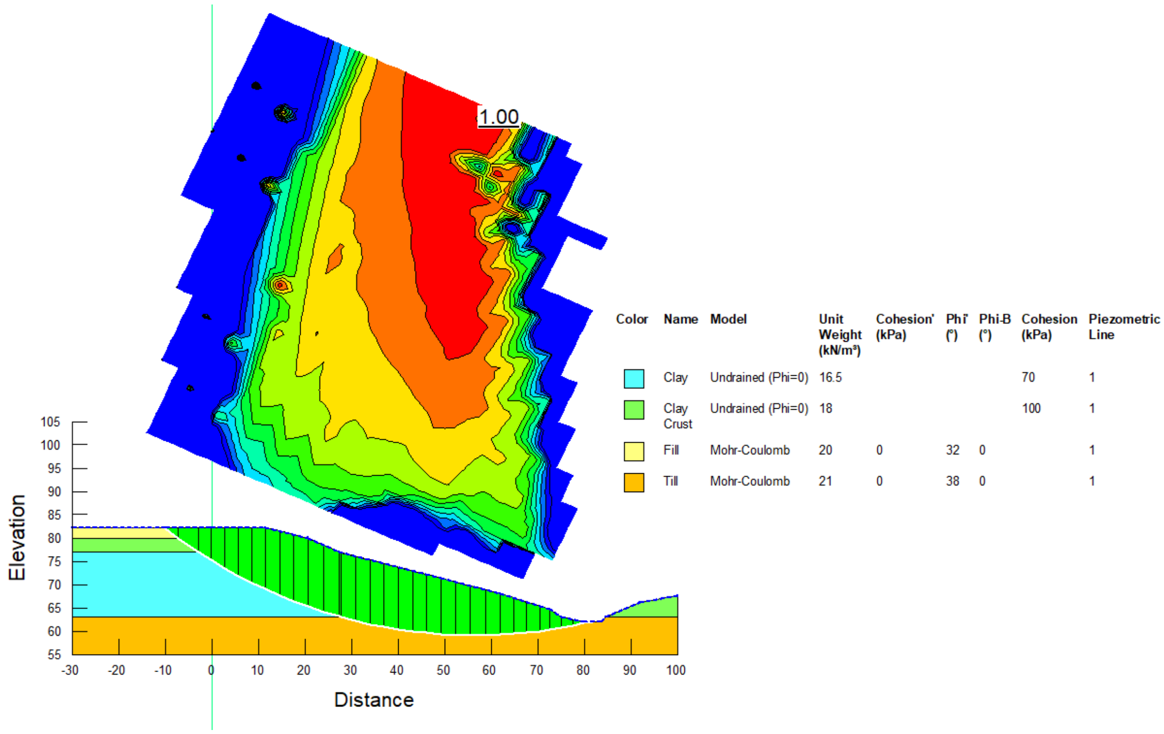


Figure 5-5 Pseudo-Static Slope Stability Analysis of Section C-C'

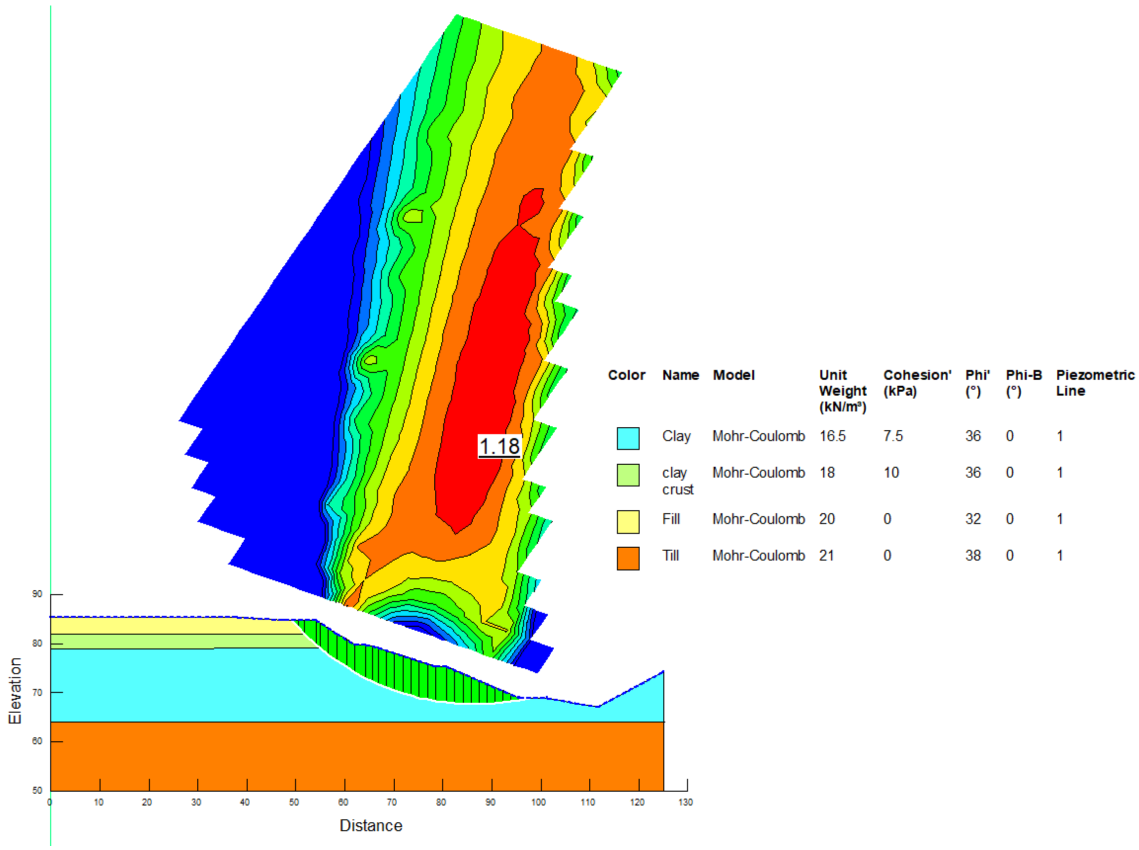


Figure 5-6 Effective Stress (Long-term) Slope Stability Analysis of Slope Profile 17

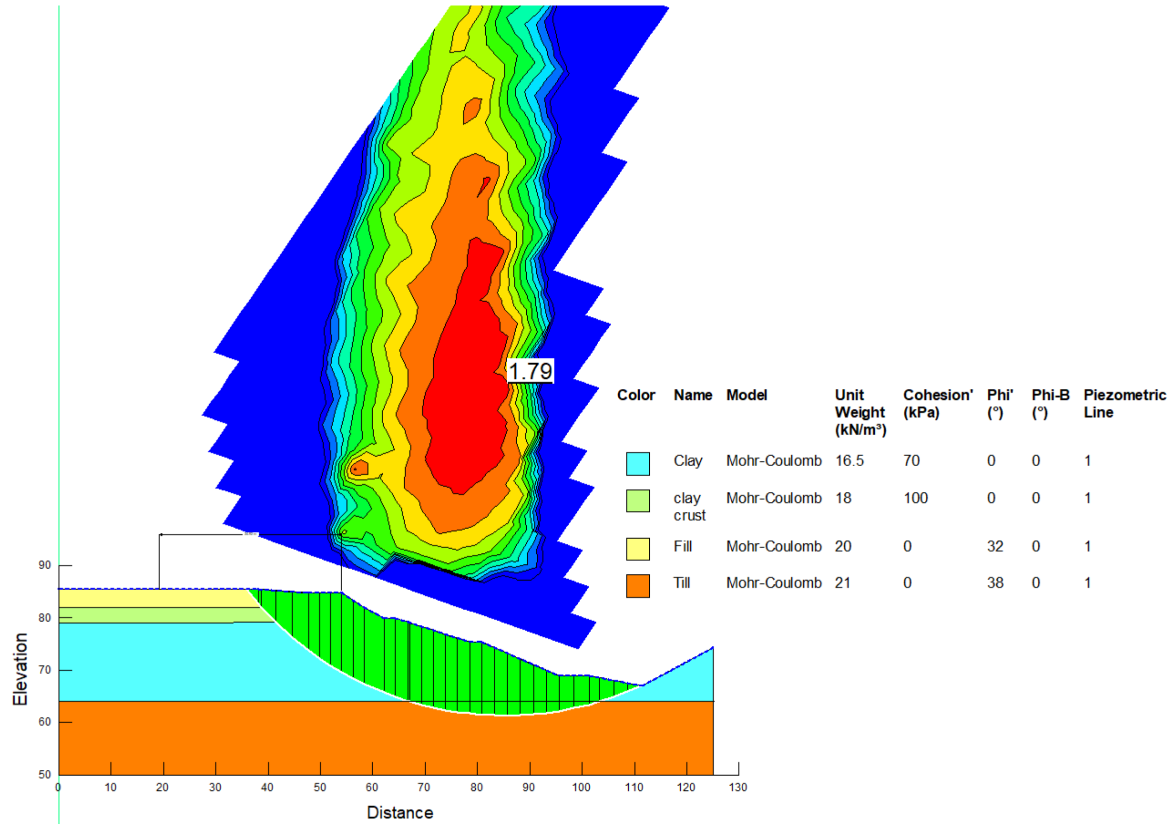


Figure 5-7 Total Stress (Short-term) Slope Stability Analysis of Slope Profile 17

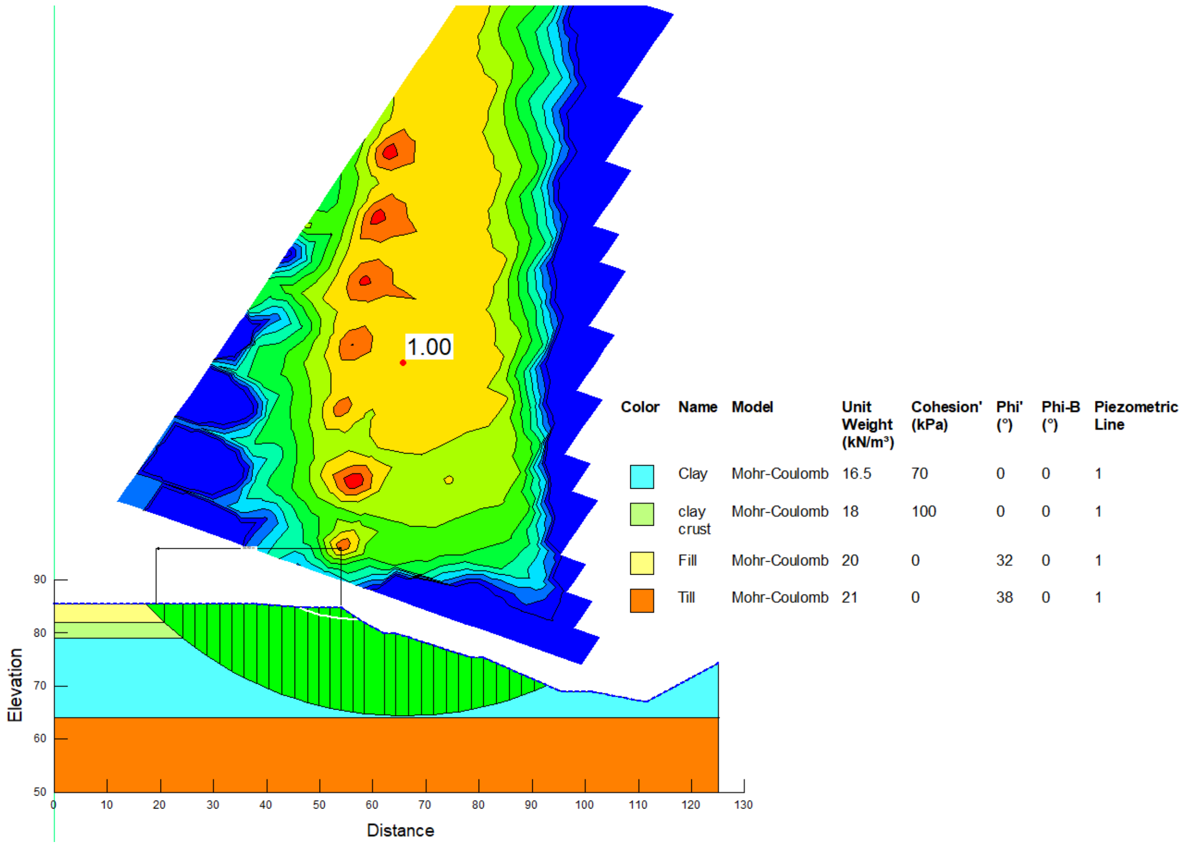


Figure 5-8 Pseudo-Static Slope Stability Analysis of Slope Profile 17

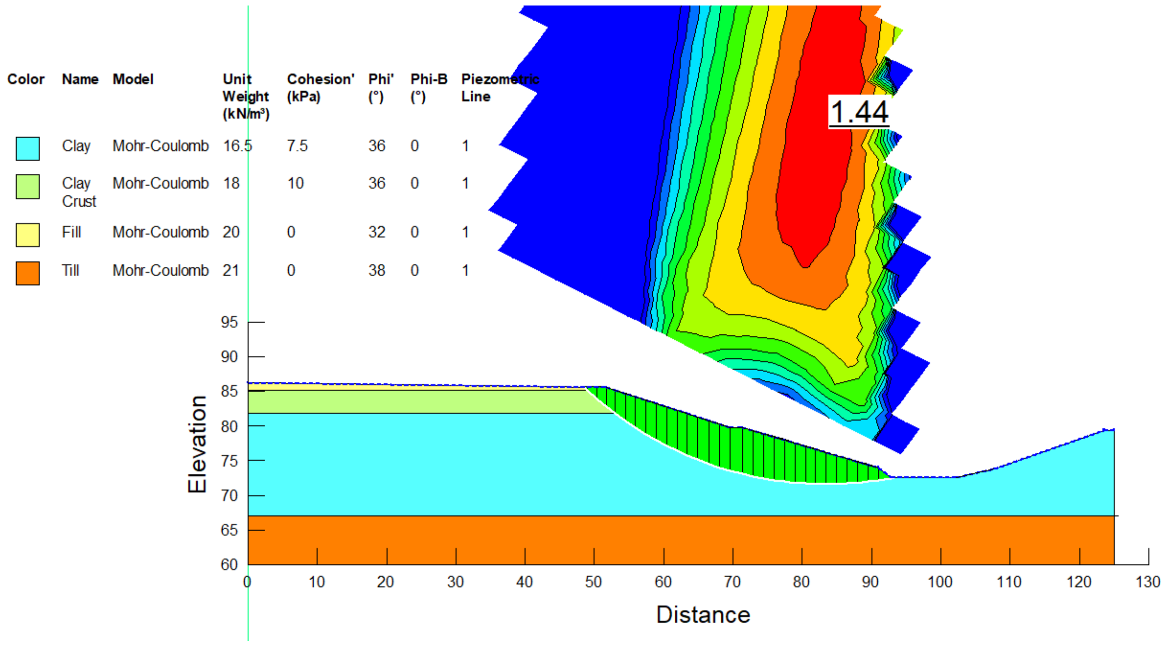


Figure 5-9 Effective Stress (Long-term) Slope Stability Analysis of Slope Profile 25

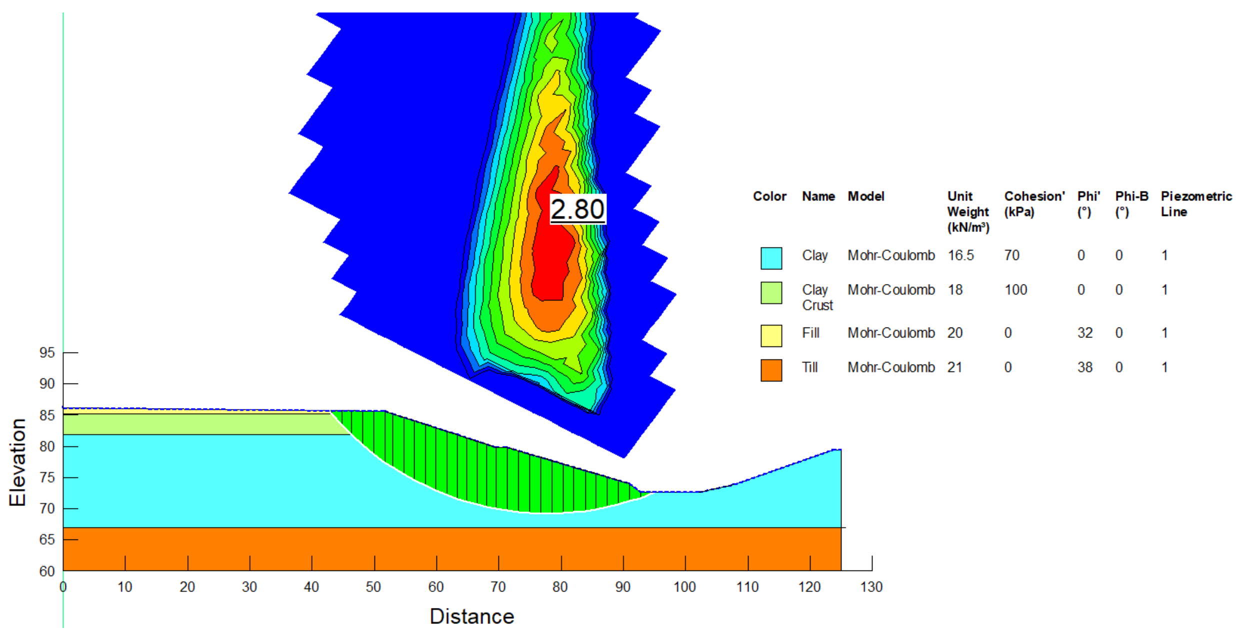


Figure 5-10 Total Stress (Short-term) Slope Stability Analysis of Slope Profile 25

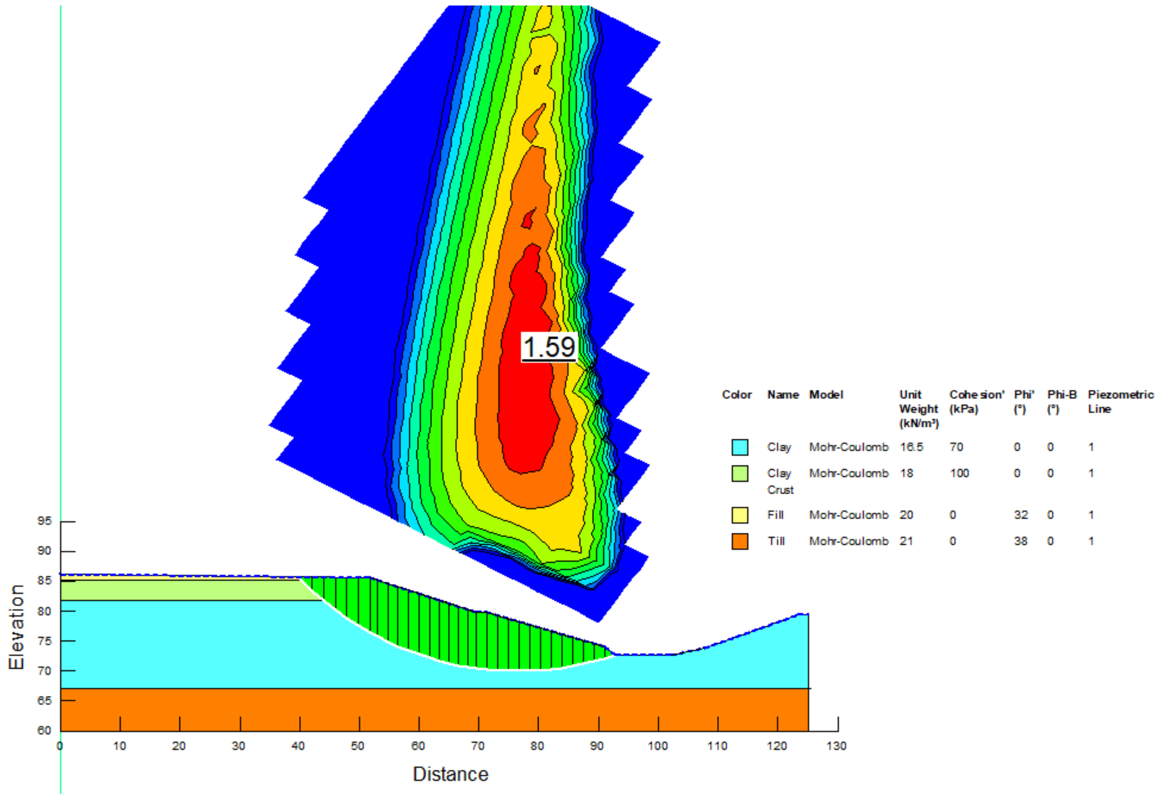


Figure 5-11 Pseudo-Static Slope Stability Analysis of Slope Profile 25

Figure 5-12 presents the unfailed versus failed slopes of 419 survey slopes in the field with calculated slope factor of safety. The factor of safety obtained from our effective stress analysis is within the reasonable range presented in this figure.

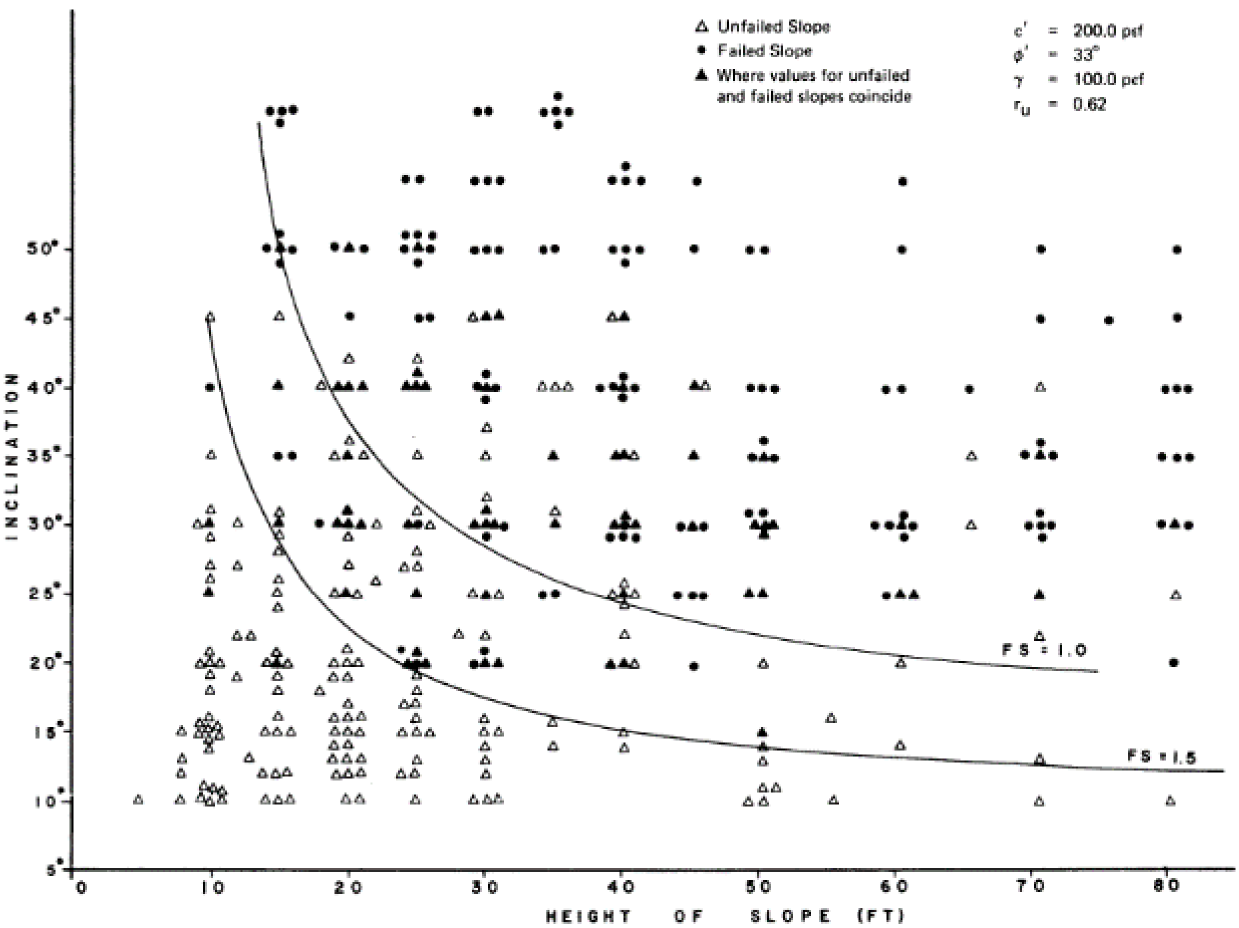


Figure 5-12 Calculated FOS and 419 Survey Slopes in the Field (from Klugan and Chung 1976)

Large-scale retrogressive landslide potential of slopes was also assessed using the MTQ approach (Thibault et al. 2008). Table 5-2 presents the factors and the partial weightings of these factors that were taken considered when comparing the triggering zones to one another and calculating a general weighting coefficient as a function of the degree of influence that each of these factors has on the susceptibility of an initial rotational landslide. For each of the zones that could be affected by a large retrogressive landslide, the slope that exhibited the lowest level of stability was used. Table 5-3 presents the factors and the partial weightings of the factors that were taken into consideration in evaluating the retrogression potential of an initial rotational landslide.

Table 5-2 Initial rotational slide potential assessment based on MTQ approach

Factors	Range	Partial weighting coefficient
Height of slope (K_H)	10 to15	1
	15 to20	1.25
	20 to25	1.5
	25 to 30	1.75
Inclination (K_I)	20 to 25	1
	25 to 30	1.25
	30 to35	1.5
Form of the slope (K_F)	concave	0.75
	rectilinear	1
	convex	1.25
Groundwater condition at base of slope (K_w)	i -	0.5
	i = 1	1
	i +	1.5
	i ++	2
Erosion (K_E)	moderate	1.25
	strong	1.5
	severe	2
Signs of instability (K_S)	yes	1.5
	no	1
Global weighting coefficient for landslides (K_{GSL})		Min = 0.47 Max = 19.7

* The values associated with each of these factors may vary slightly from one region to another.

Notes: (values obtained for the Site are in the red boxes)

Table 5-3 Retrogression potential assessment based on MTQ approach

Factors	Range	Partial weighting coefficient
Removal of debris hindered by obstacles (K_O)	yes	0.8
	no	1
Favorable stratigraphy (K_{St})	yes	0.8
	no	1
Undrained shear strength (K_{su})	0.5 to 1	1
	0.2 to 0.5	1.5
	< 0.2	2
Liquid limit (K_{WL})	60 to 80	1
	40 to 60	1.5
	20 to 40	2
Stability Number (K_N)	3 to 4	0.8
	4 to 5	1
	5 to 6	1.5
	6 and +	2
Global weighting coefficient for large retrogressive landslides (K_{RL})		Min = 0.51 Max = 8

* The values associated with each of these factors may vary slightly from one region to another.
* The values associated with each of these factors may vary slightly from one region to another.

Notes: (values obtained for the Site are in the red boxes)

Possible clay liquefaction and strength degradation under the anticipated earthquake excitation were also considered to check their possible contributions to large scale retrogressive landslide. According to the current earthquake industry practice (Bray et al. 2004), clay with plasticity index higher than 20 is not considered liquefiable and the lowest plastic index obtained from the retrieved clay samples is 30 (average is higher than 40). Based on the extensive research carried out on Champlain Sea clay (Leda Clay) behavior under seismic loading condition, it is generally accepted that Champlain Sea Clay is not susceptible to strength degradation under anticipated Canada East seismic conditions. It should be noted that possible clay strength degradation due to large strain is a separate issue from the strength degradation under seismic loading condition.

5.3 SUMMARY OF KEY FINDINGS

- Based on the EXP updated preliminary geotechnical investigation & supplementary geotechnical investigation findings, existing slope geometry and Stantec’s field reconnaissance observations, it is our opinion that the south-facing slope marking the boundary of the proposed development site is considered to have a low potential for large-scale retrogressive landslide. This conclusion is based on the following factors: Slope morphology – maximum 20 m height, with inclination under than 25 degrees (generally < 20 degrees, with failed slope segment averaging 16 degrees). Slopes are concave (previously failed) and/or rectilinear.
- Erosion and signs of instability– Only minor erosion was identified at the base of the south-facing slope, below from the proposed development area, during the site reconnaissance. Historical landslides were noted during the site reconnaissance; however, no active movement was detected immediately downslope from proposed development area.
- Removal of debris hindered by obstacles – Old landslide debris is present at the base of the south-facing slope and the unnamed creek, due to its small size, was observed to have a low erosion potential. Potential debris removal was still considered for the worst-case scenario
- Favorable stratigraphy – Not interpreted to be present at the site (i.e., clay samples showed overall low sensitivity, with maximum liquidity index close to 1.0; a liquidity index of greater than 1.2 is considered indicative of a clay which could flow out of a landslide scar)
- Undrained shear strength – the measured remolded shear strength of clay was observed to be significantly higher than 1 kPa (remolded shear strength of less than 1 kPa, as measured by the falling cone, is considered indicative of a clay which could flow out of a landslide scar)
- Liquid limit – the measured liquid limits for the clay crust and massive clay are about 40 to 60 and 60 to 80, respectively
- Stability number – a maximum calculated stability number, Ns, of 4.2 was calculate for the slope. Currently, the critical stability number for flow slides is defined as “19 times IP”, where IP is the plasticity index defined as a fraction. A minimum IP value of 36 was measured on the collected samples, which suggest that the critical stability number would be 6.8, which significantly greater than the calculated value noted above.
- Based on the borehole data, the gulley has reached the till layer. The period during slope formation, where a slope is at greatest risk of retrogressive failure, is as the valley floor approaches the underlying till layer. Once the valley floor reaches the till, the till layer is drained, and the groundwater regime or flow-patterns becomes less critical.

Based on the analyses herein, and consistent with the overall investigative approach conducted in Quebec (Thibault et al. 2008), the occurrence of large retrogressive landslides is not expected at the Site. This conclusion reflects a review of the following parameters, which are currently considered as critical:

- Stability Number greater than 19 times IP
- Remolded shear strength, as measured by a fall cone of less than 1 kPa for more than 35% of the slope height
- Liquidity index of greater than 1.2 for more than 35% of the slope height
- The valley formation and groundwater regime

None of the above criteria indicated that the slope could be at risk of a large retrogressive failure.

It is noted that as part of the above evaluation, Stantec did consider the presence of the nearby retrogressive landslides that have occurred in the Cardinal Creek watershed area, specifically two large historical features located approximately 400 m south of the Site (i.e., LID-01 and LID-02). Based on the literature review, these slides most likely occurred when the valley formation was in its intermediate or critical stage, when the valley floor was approaching the base of the clay layer. Regarding the safer or final phase of valley formation, Lefebvre (2017) noted that "once the valley has cut completely through the clay deposit, the river flowing on till or bedrock, the till layer is discharging in the valley with a water head at equilibrium with the water level in the river. This creates a strong underdrainage of the whole slope, a condition highly favorable to slope stability." The presence of these nearby retrogressive landslides confirmed the requirement to carry out the analysis described above.

The results of the investigation confirmed that the valley formation adjacent to the project site is in its final stage and that the soil characteristics are currently or no longer consistent with those associated with quick clays.

6.0 SETBACK DISTANCE

6.1 METHODS

6.1.1 Fahrböschung Angle (Angle of Reach)

In addition to the geotechnical stability assessment, Stantec employed a modified Fahrböschung angle or angle of reach method (Heim, 1932) to determine a setback distance.

The Fahrböschung angle was defined by Heim (1932) as the line connecting the crest of a landslide source to the toe of the deposit, measured along the approximate centerline of motion.

It is described by:

$$TAN(\alpha) = \frac{H}{l}$$

Where α is the Fahrböschung angle, H is the vertical difference between the landslide crest and the base of the landslide toe, and l is the horizontal travel of the landslide between the same two points. The Fahrböschung angle provides an empirically derived stable slope angle for relatively homogenous terrain units (in our case, a single homogenous unit for the entire south-facing slope segment south of the Site).

The Fahrböschung angle was calculated along 30 different cross sections, each separated from the others by a 100 m and established along the south-facing slope in a manner that accounted for topographic variation. A cross plane was then projected through each cross-section using the average calculated Fahrböschung angle of mapped landslides, from the toe to the crest of the slopes, along each cross sections. The locations were the projected plane daylighted on the upslope portion of the cross sections was used to delineate setback limits corresponding to different landslide return period. Refer to Figure 6-1 and Figure 6-2 for diagrams illustrating Fahrböschung angles calculated for the assessment.

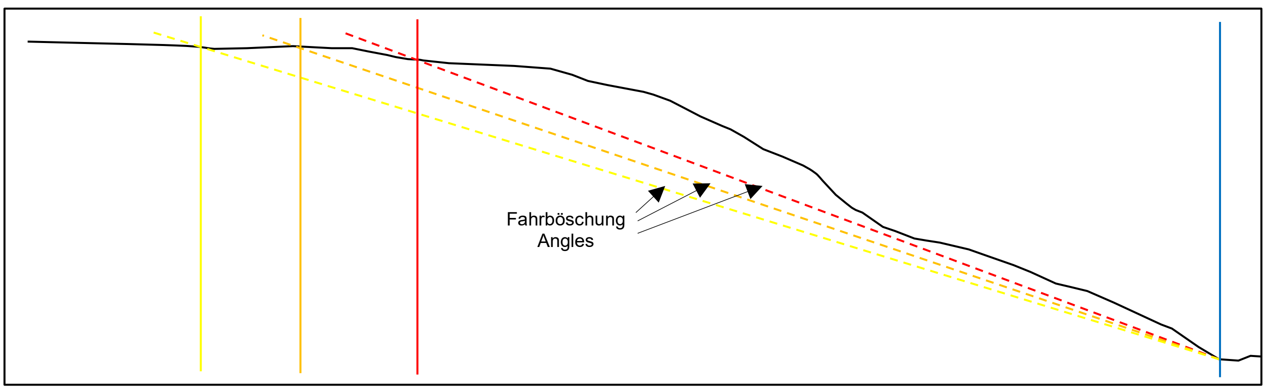


Figure 6-1 Fahrböschung angle of mapped landslides. Solid black line is the existing ground surface, vertical blue line is the toe of the slope, and the red, orange and yellow lines represent different Fahrböschung angles.

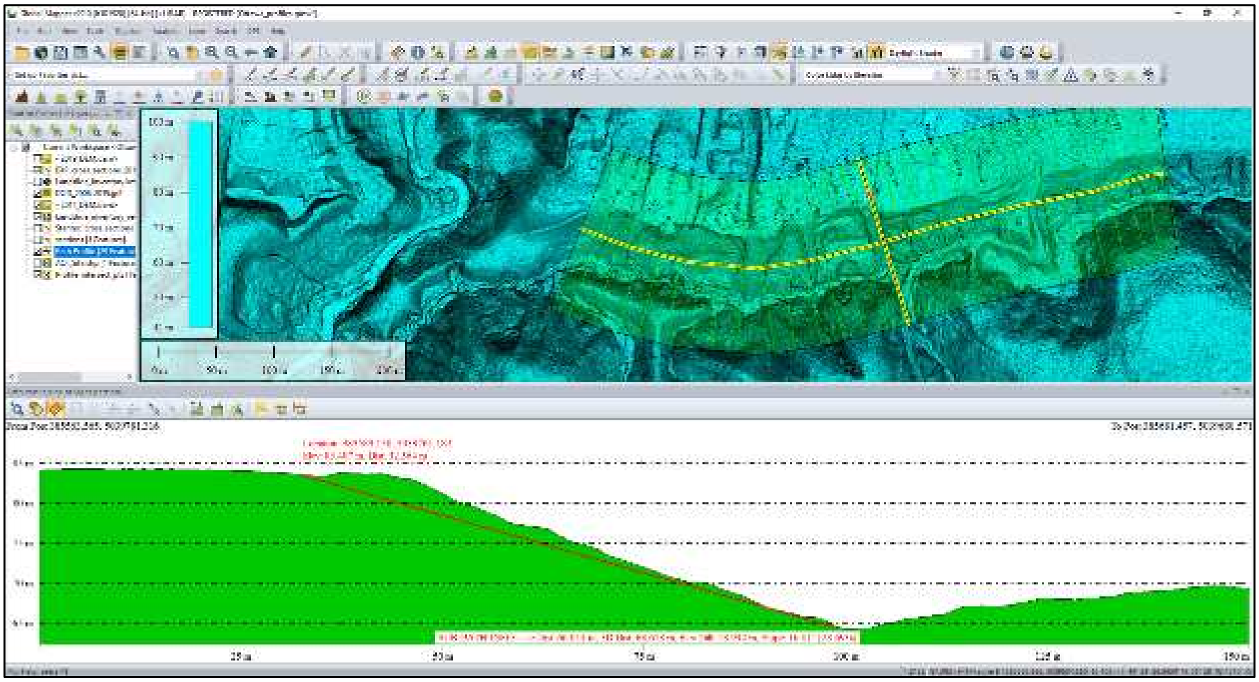


Figure 6-2 Cross Sections and Example of Setback Calculation

6.1.2 Hazard Calculation (PO)

Stantec calculated an annual probability of occurrence (PO) for rotational landslides in flat-over-steep terrain, similar in character to historical slides mapped within the setback zone. A hazard term, $H_{T,S}$, for each landslide was calculated where $H_{T,S}$ is the product of the probability of occurrence of a landslide in time H_T, and space H_s:

$$H_{T,S} = H_T \times H_s$$

H_s was determined by dividing the area occupied by an individual landslide by the total area of the setback zone.

H_T was determined by estimating an age range within which all mapped landslides were inferred to have occurred (based on morphology, vegetative cover, and field evidence), and assuming a stochastic distribution of ages within that range for landslides located on the south-facing slope south of the Site. The process was repeated 10 times to allow for variability between possible distributions, and reduce the overall potential error. $H_{T,S}$ was, therefore, calculated for each of 10 iterations, and the PO for each iteration was calculated as:

$$PO = 1 - \left((1 - H_{T,S1}) \times (1 - H_{T,S2}) \times \ldots (1 - H_{T,S1n}) \right)$$

The final PO for the study was the average of all individual iterations of PO.

6.1.3 Hazard Calculation from Slope Stability Analysis

Silva et al. (2008) proposed a method to estimate PO from FoS analysis. FoS herein was compared to PO using Line II in Silva et al. (2008) and the following best fit line equation:

$$PO = 26352e^{-13.38FoS}$$

Where PO = Probability of Occurrence, e = Euler's number, and FoS = Factor of Safety.

6.2 RESULTS

6.2.1 Fahrböschung

Fahrböschung angles were measured at each of 18 landslides on the south-facing slope adjacent to the proposed development. An angle of 16^{o} was selected based on the observable Fahrböschung and the slope stability analysis in Section 5. Flatter angles were noted in the mapping (Landslide 25 and 63). however, Stantec expects that landslide 25 does not include full headscarp height, and landslide 63 is difficult to differentiate from gully erosion occurring within a larger landslide body. The observed Fahrböschung angle for each landslide is given in Table 6-1.

Table 6-1 Fahrböschung Angle for Mapped Landslides

Landslide ID	H (m)	l (m)	Fahrböschung Angle (°)
03	17.7	59.5	-16.5
08	18.3	61.1	-16.7
11	16.9	56.6	-16.6
20	16.4	43.9	-20.5
23	12.5	36.6	-18.8
25	7.1	28.3	-14.1
26	10.1	33.1	-17.0
42	4.9	14.4	-18.9
46	5.2	16.8	-17.1
47	6.0	15.5	-21.2
52	5.3	18.1	-16.4
55	3.4	10.7	-17.7
58	3.5	8.6	-22.1
60	5.0	16.2	-16.9
61	3.9	9.3	-22.5
63	2.2	10.3	-11.9
64	4.2	6.6	-32.6
65	3.1	7.5	-22.5

6.2.2 Hazard Calculation (PO) for Zone 1

Each landslide hazard (H_S * H_T) and the PO for landslides in Zone 1 (the zone identified by the 16° Fahrböschung) is provided in Table 6-2.

Obtained PO values range from 0.00302 to 0.00964, which appear statistically robust with low variability (a standard deviation across 10 iterations of 0.00225). The annualized PO for a landslide similar to those mapped within the calculated setback zone is estimated at 0.00594 (Table 6-3) or occurring with a return period of 1:168. Put another way, there is approximately a 45% chance a landslide of similar character and magnitude as the ones identified along the south-facing slope will occur in any run of 100 years, less than 6% in any decade, or almost 95% in a run of 500 years.

6.2.3 Hazard Calculation for Zones 2-4

Zones 2 – 4 are bound by two additional Fahrböshcung lines. One at 15-16° (differentiating Zones 2 and 3, 15 ° for the west half where slopes are about 20 m high and 16 ° for the east half where slopes are about 15 m high), and one at 11-13° (differentiating Zones 3 and 4, 11 ° for the west half where slopes are about 20 m high and 13 ° for the east half where slopes are about 15 m high).

The 15-16° angle was selected as the pseudo-static Fahrböshcung based on the analysis in Chapter 5, with a return period of ~1:2,500 years (1:2475) and Factor of Safety 1.50 (corresponding to a return period of ~1:2,500 years)

Given the limits of the geotechnical information currently held at the site, using Silva et al (2008), a FoS of 1.58 delimits a zone beyond which the PO is < 1:10,000 (and Fahrböschung of 11-13° (Figure 6-3)).

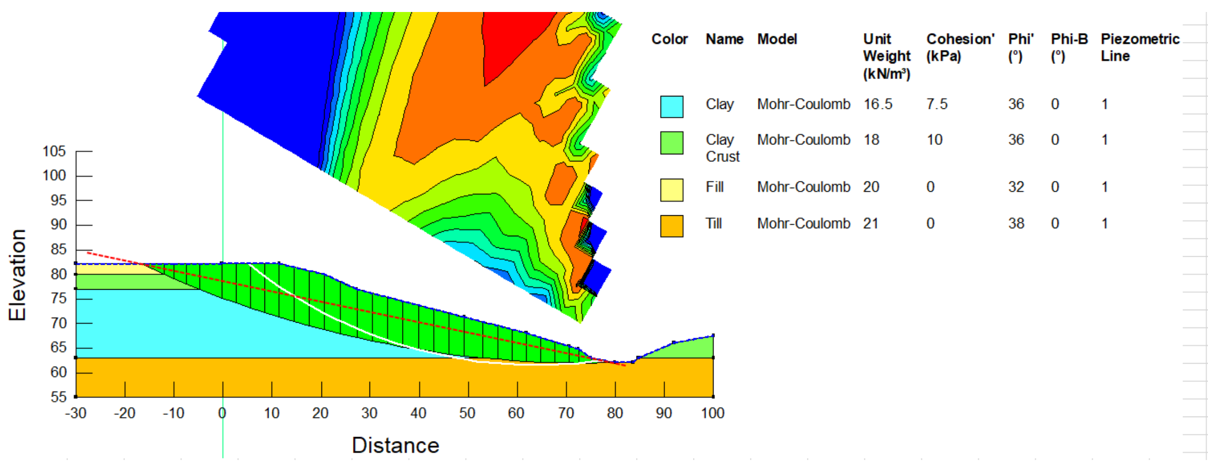


Figure 6-3 Fahrböschung (11 °) of the 1:10,000 return period event (FoS ~1.58)-Section C-C’

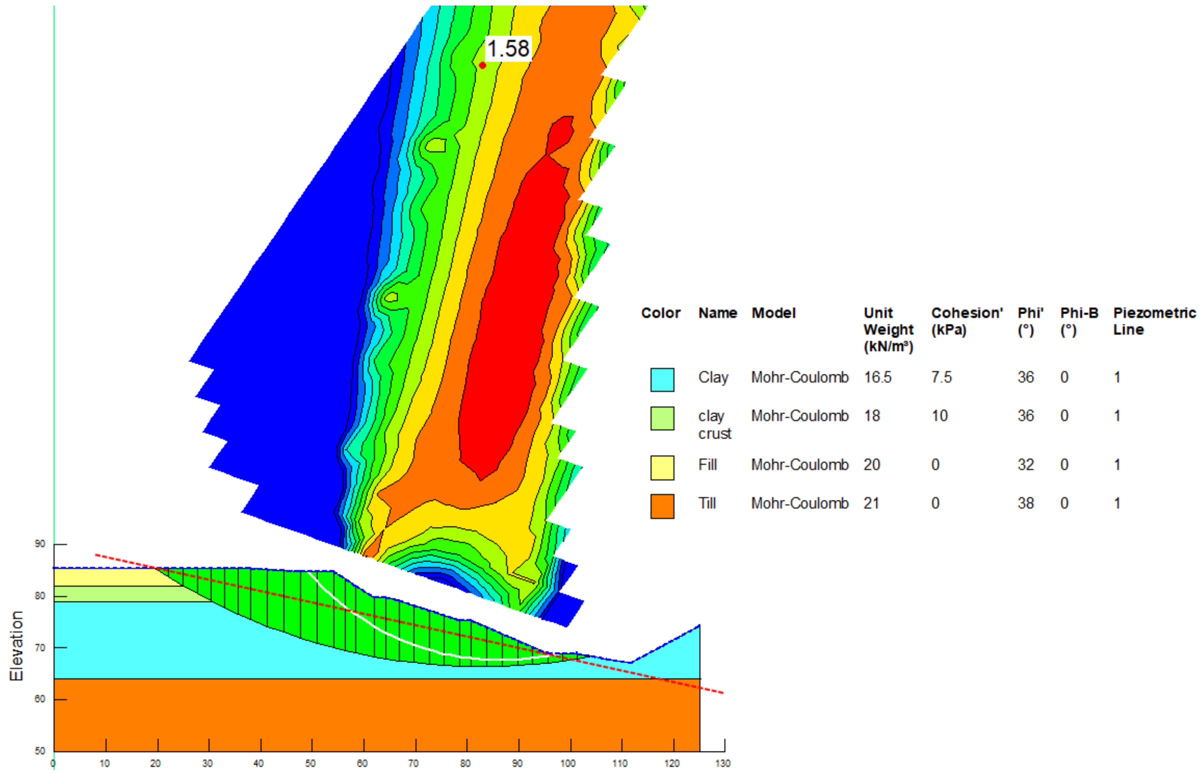


Figure 6-4 Fahrböschung (12 °) of the 1:10,000 return period event (FoS ~1.58)-Slope Profile 17

Due to the hazard zone impact to the development plan, the existing slope crest at planned townhouse blocks 15, 16 and 17 are proposed to be lowered by about 2.5 m (see conceptual macro grading plan in Appendix B). Based on the Fahrböshcung line at 13º shown on the Figure 6-5, with the proposed grade lowering, the development plan may remain unchanged.

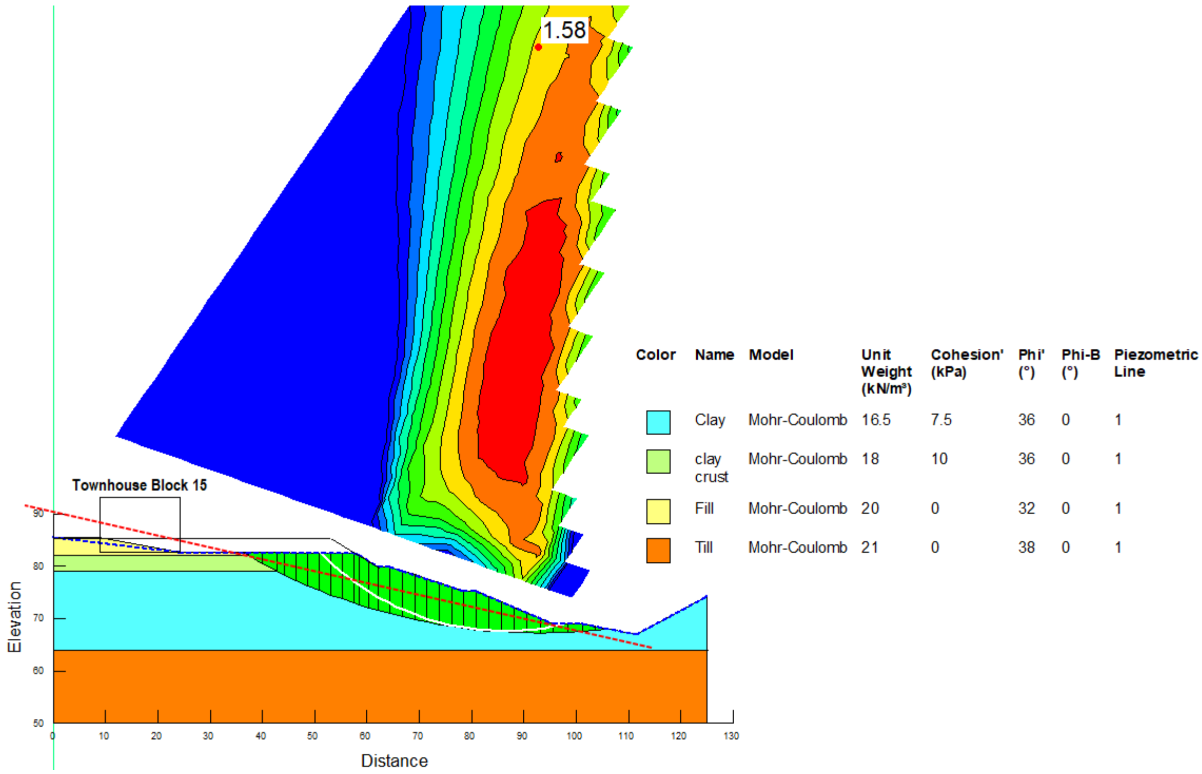


Figure 6-5 Fahrböschung (13 °) of the 1:10,000 return period event (FoS ~1.58)-Slope Profile 17 After 2.5 m Grade Lowering

The large retrogressive failures (spreads) south of the study area (Fahrböshcung of about 5°) were not used in this study based on the results of the stability analysis.

Table 6-2 PO outcome for 10 scenarios with ages between 5 and 200, randomly distributed between landslides.

Landslide ID	Area (ha)	H_S	1	2	3	4	5	6	7	8	9	10
			H_T	H_T	H_T	H_T	H_T	H_T	H_T	H_T	H_T	H_T
3	0,46	0,1357	0,01449	0,02941	0,01429	0,02222	0,03125	0,01042	0,00917	0,00806	0,01136	0,00840
8	0,267	0,0787	0,01136	0,00840	0,00685	0,02273	0,02500	0,00926	0,02857	0,00521	0,00671	0,01515
11	0,171	0,0504	0,00704	0,00526	0,00826	0,00704	0,00654	0,01000	0,00820	0,00645	0,00917	0,00709
20	0,095	0,028	0,00613	0,04545	0,00917	0,03125	0,02439	0,01351	0,04545	0,01205	0,00508	0,00518
23	0,086	0,0254	0,05882	0,01961	0,00699	0,00787	0,00503	0,00667	0,12500	0,00741	0,01176	0,00633
25	0,081	0,0239	0,01818	0,00503	0,02778	0,00556	0,00621	0,01111	0,02564	0,00787	0,00833	0,00917
26	0,075	0,022	0,00730	0,04762	0,05556	0,00901	0,01429	0,00599	0,00521	0,00667	0,02041	0,01205
42	0,021	0,0063	0,01471	0,00592	0,01124	0,00625	0,03226	0,01786	0,01333	0,00690	0,00508	0,01299
46	0,017	0,005	0,00543	0,01053	0,00602	0,01429	0,00833	0,00826	0,01639	0,01333	0,00769	0,01111
47	0,016	0,0047	0,00952	0,00526	0,01786	0,00538	0,00935	0,02273	0,01190	0,00538	0,00585	0,00581
52	0,014	0,0041	0,00610	0,00763	0,00909	0,00758	0,00685	0,00649	0,05882	0,01042	0,02439	0,00877
55	0,01	0,0029	0,00781	0,00685	0,01031	0,03226	0,04762	0,00637	0,00826	0,00847	0,05556	0,00585
58	0,009	0,0027	0,00629	0,04167	0,00787	0,00833	0,01282	0,00602	0,00962	0,00575	0,00546	0,02564
60	0,008	0,0022	0,00524	0,00602	0,01087	0,00709	0,03226	0,01205	0,00787	0,01235	0,00855	0,00794
61	0,007	0,0022	0,00508	0,00971	0,02439	0,00877	0,00562	0,01176	0,00562	0,02326	0,00709	0,00602
63	0,004	0,0012	0,00685	0,02128	0,00730	0,00602	0,00833	0,02000	0,02564	0,01176	0,01010	0,08333
64	0,003	0,001	0,00559	0,00629	0,00532	0,01923	0,00521	0,00505	0,01316	0,00971	0,04000	0,01818
65	0,003	0,0009	0,00649	0,00625	0,00552	0,00641	0,00556	0,00538	0,01613	0,01786	0,00565	0,00556
		PO	0,00573	0,00818	0,00557	0,00690	0,00838	0,00399	0,00964	0,00302	0,00408	0,00392
											Average PO	**0,00594**
											Standard Deviation	**0,00225**

Table 6-3 Exceedance probability for rotational landslides in the flat-over-steep terrain within Zone 1 (see Chapter 6.3).

	Number of Years				
	1	**10**	**50**	**100**	**500**
Exceedance Probability	0.00594	0.05785	0.25768	0.44896	0.94919
Exceedance Probability (%)	0.6	5.8	25.8	44.9	94.9

Note that only landslides present along the south-facing slope within 100 m from the proposed development area were used to support the analysis, which excluded the two large retrogressive landslides located 400 south of the Site. Based on literature available on large retrogressive landslides located in the Ottawa Region (e.g., Brooks 2019, Brooks et al. 2021), these features are understood to have occurred prehistorically, likely triggered by significant paleo-earthquakes that could reach as far 5,500 years ago.

6.3 HAZARD ZONES

The Rideau Valley Conservation Authority required that a quantitative landslide hazard assessment be completed to demonstrate that the landslide hazard, including a large “catastrophic landslide”, has an annual probability less than 1:10,000. Since site construction is not yet underway, Stantec proposes that hazard zonation from FVRD (2020) be adopted (Table 6-4).

Figure 6-6 shows results of the hazards zonation overlain on the proposed development site plan.

Table 6-4 The Fraser Valley Approach (FVRD 2020) adapted to the Old Montreal Road Site Conditions

	Zone 1	Zone 2	Zone 3	Zone 4
Return Period	1:200	1:200 - 1:2,500	1:2,500 - 1:10,000	<1:10,000
Minor Repair (<25%)	5	1	1	1
Major Repair (>25%)	5	1	1	1
Reconstruction	5	2	1	1
Extension	5	3	2	1
New Building	5	4	3	1
Subdivision	5	5	4	1
Rezoning	5	5	4	1

Hazard-Related Responses to Development Approval Applications

1	Approval without conditions relating to hazards.
2	Approval, without siting conditions or protective works conditionsbut with a covenant including “save harmless” conditions.
3	Approval, but with siting requirements to avoid the hazard, or with requirements for protective works to mitigate the hazard.
4	Approval as #3 above, but with a covenant including “save harmless” conditions as well as siting conditions, protective works, or both.
5	Not approvable.

Figure 6-6 Hazard Zonation

7.0 CLOSURE

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of EXP/ Phoenix Homes, who is identified as “the Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions
- Planning, design or construction

We trust that the information contained in this report is adequate for your present purposes. If you have any questions about the contents of the report or if we can be of any other assistance, please contact us at your convenience.

8.0 REFERENCES

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APPENDIX A
Statement of General Conditions

PROFESSIONAL SERVICES TERMS AND CONDITIONS

The following Terms and Conditions are attached to and form part of the Proposal for Professional Services to be performed by STANTEC and together, when the CLIENT authorizes STANTEC to proceed with the services, constitute the AGREEMENT.

DESCRIPTION OF WORK: STANTEC shall render the services described in the Proposal (hereinafter called the "SERVICES") to the CLIENT.

TERMS AND CONDITIONS: No terms, conditions, understandings, or agreements purporting to modify or vary these Terms and Conditions shall be binding unless hereafter made in writing and signed by the CLIENT and STANTEC. In the event of any conflict between the Proposal and these Terms and Conditions, these Terms and Conditions shall take precedence. This AGREEMENT supercedes all previous agreements, arrangements or understandings between the parties whether written or oral in connection with or incidental to the PROJECT

COMPENSATION: Payment is due to STANTEC upon receipt of invoice. Failure to make any payment when due is a material breach of this AGREEMENT and will entitle STANTEC, at its option, to suspend or terminate this AGREEMENT and the provision of the SERVICES. Interest will accrue on accounts overdue by 30 days at the lesser of 1.5 percent per month (18 percent per annum) or the maximum legal rate of interest. Unless otherwise noted, the fees in this agreement do not include any value added, sales, or other taxes that may be applied by Government on fees for services. Such taxes will be added to all invoices as required.

NOTICES: Each party shall designate a representative who is authorized to act on behalf of that party. All notices, consents, and approvals required to be given hereunder shall be in writing and shall be given to the representatives of each party.

TERMINATION: Either party may terminate the AGREEMENT without cause upon thirty (30) days notice in writing. If either party breaches the AGREEMENT and fails to remedy such breach within seven (7) days of notice to do so by the non-defaulting party, the non-defaulting party may immediately terminate the Agreement. Non-payment by the CLIENT of STANTEC's invoices within 30 days of STANTEC rendering same is agreed to constitute a material breach and, upon written notice as prescribed above, the duties, obligations and responsibilities of STANTEC are terminated. On termination by either party, the CLIENT shall forthwith pay STANTEC all fees and charges for the SERVICES provided to the effective date of termination.

ENVIRONMENTAL: Except as specifically described in this AGREEMENT, STANTEC's field investigation, laboratory testing and engineering recommendations will not address or evaluate pollution of soil or pollution of groundwater.

Where the SERVICES include storm water pollution prevention (SWPP), sedimentation or erosion control plans, specifications, procedures or related construction observation or administrative field functions, CLIENT acknowledges that such SERVICES proposed or performed by STANTEC are not guaranteed to provide complete SWPP, sedimentation or erosion control, capture all run off or siltation, that any physical works are to be constructed and maintained by the CLIENT's contractor or others and that STANTEC has no control over the ultimate effectiveness of any such works or procedures. Except to the extent that there were errors or omissions in the SERVICES provided by STANTEC, CLIENT agrees to indemnify and hold STANTEC harmless from and against all claims, costs, liabilities or damages whatsoever arising from any storm water pollution, erosion, sedimentation, or discharge of silt or other deleterious substances into any waterway, wetland or woodland and any resulting charges, fines, legal action, cleanup or related costs.

PROFESSIONAL RESPONSIBILITY: In performing the SERVICES, STANTEC will provide and exercise the standard of care, skill and diligence required by customarily accepted professional practices normally provided in the performance of the SERVICES at the time and the location in which the SERVICES were performed.

LIMITATION OF LIABILITY: The CLIENT releases STANTEC from any liability and agrees to defend, indemnify and hold STANTEC harmless from any and all claims, damages, losses, and/or expenses, direct and indirect, or consequential damages, including but not limited to attorney's fees and charges and court and arbitration costs, arising out of, or claimed to arise out of, the performance of the SERVICES, excepting liability arising from the sole negligence of STANTEC. It is further agreed that the total amount of all claims the CLIENT may have against STANTEC under this AGREEMENT, including but not limited to claims for negligence, negligent misrepresentation and/or breach of contract, shall be strictly limited to the lesser of professional fees paid to STANTEC for the SERVICES or $500,000. No claim may be brought against STANTEC more than two (2) years after the cause of action arose. As the CLIENT's sole and exclusive remedy under this AGREEMENT any claim, demand or suit shall be directed and/or asserted only against STANTEC and not against any of STANTEC's employees, officers or directors.

STANTEC's liability with respect to any claims arising out of this AGREEMENT shall be absolutely limited to direct damages arising out of the SERVICES and STANTEC shall bear no liability whatsoever for any consequential loss, injury or damage incurred by the CLIENT, including but not limited to claims for loss of use, loss of profits and/or loss of markets.

INDEMNITY FOR MOLD CLAIMS: It is understood by the parties that existing or constructed buildings may contain mold substances that can present health hazards and result in bodily injury, property damage and/or necessary remedial measures. If, during performance of the SERVICES, STANTEC knowingly encounters any such substances, STANTEC shall notify the CLIENT and, without liability for consequential or any other damages, suspend performance of services until the CLIENT retains a qualified specialist to abate and/or remove the mold substances. The CLIENT agrees to release and waive all claims, including consequential damages, against STANTEC, its subconsultants and their officers, directors and employees arising from or in any way connected with the existence of mold on or about the project site whether during or after completion of the SERVICES. The CLIENT further agrees to indemnify and hold STANTEC harmless from and against all claims, costs, liabilities and damages, including reasonable attorneys' fees and costs, arising in any way from the existence of mold on the project site whether during or after completion of the SERVICES, except for those claims, liabilities, costs or damages caused by the sole gross negligence and/or knowing or willful misconduct of STANTEC. STANTEC and the CLIENT waive all rights against each other for mold damages to the extent that such damages sustained by either party are covered by insurance.

DOCUMENTS: All of the documents prepared by or on behalf STANTEC in connection with the PROJECT are instruments of service for the execution of the PROJECT. STANTEC retains the property and copyright in these documents, whether the PROJECT is executed or not. These documents may not be used for any other purpose without the prior written consent of STANTEC. In the event STANTEC's documents are subsequently reused or modified in any material respect without the prior consent of STANTEC, the CLIENT agrees to defend, hold harmless and indemnify STANTEC from any claims advanced on account of said reuse or modification.

Any document produced by STANTEC in relation to the Services is intended for the sole use of Client. The documents may not be relied upon by any other party without the express written consent of STANTEC, which may be withheld at STANTEC's discretion. Any such consent will provide no greater rights to the third party than those held by the Client under the contract, and will only be authorized pursuant to the conditions of STANTEC's standard form reliance letter.

STANTEC cannot guarantee the authenticity, integrity or completeness of data files supplied in electronic format ("Electronic Files"). CLIENT shall release, indemnify and hold STANTEC, its officers, employees, consultants and agents harmless from any claims or damages arising from the use of Electronic Files. Electronic files will not contain stamps or seals, remain the property of STANTEC, are not to be used for any purpose other than that for which they were transmitted, and are not to be retransmitted to a third party without STANTEC's written consent.

FIELD SERVICES: STANTEC shall not be responsible for construction means, methods, techniques, sequences or procedures, or for safety precautions and programs in connection with work on the PROJECT, and shall not be responsible for any contractor's failure to carry out the work in accordance with the contract documents. STANTEC shall not be responsible for the acts or omissions of any contractor, subcontractor, any of their agents or employees, or any other persons performing any of the work in connection with the PROJECT.

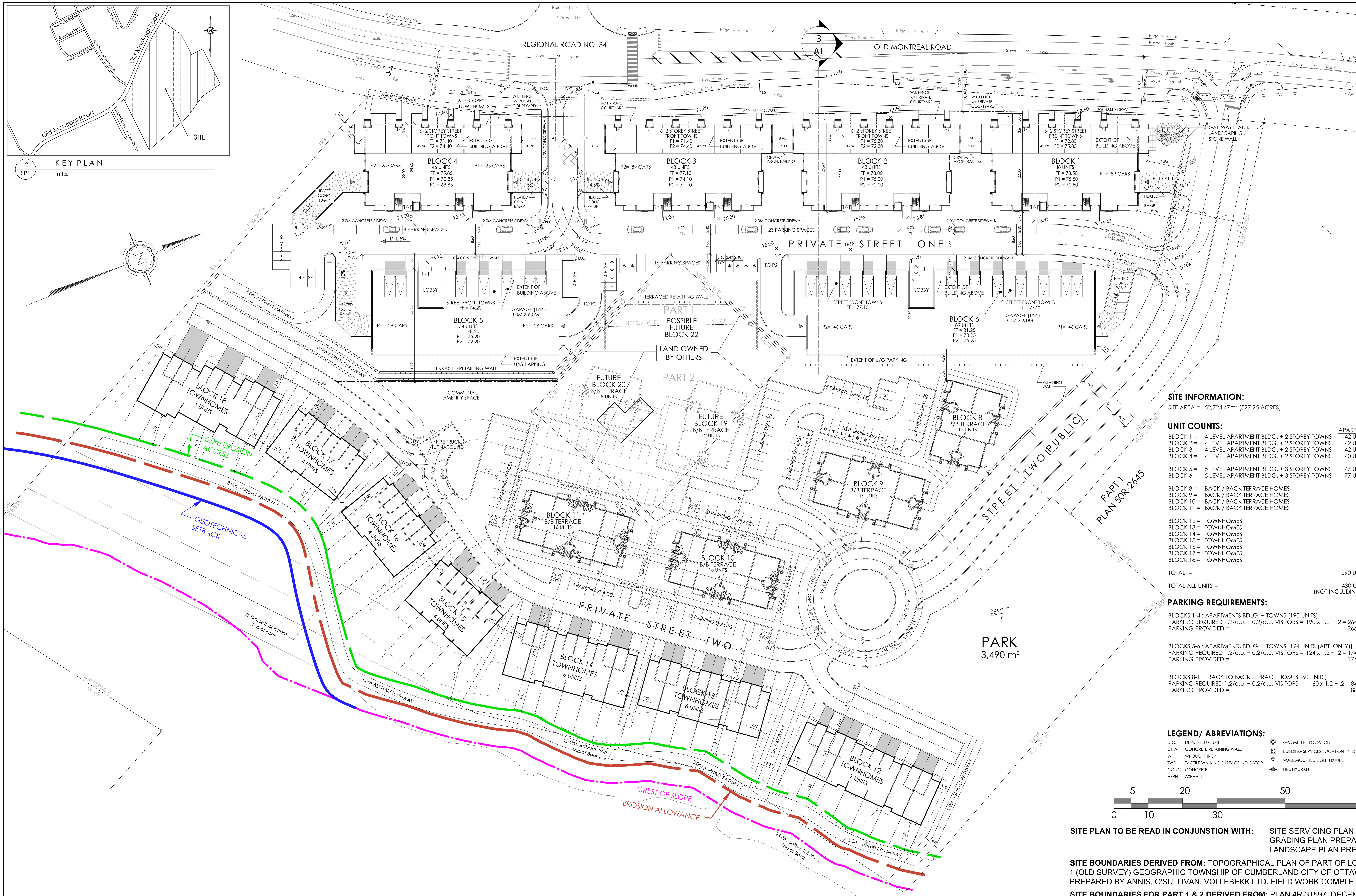
GOVERNING LAW/COMPLIANCE WITH LAWS: The AGREEMENT shall be governed, construed and enforced in accordance with the laws of the jurisdiction in which the majority of the SERVICES are performed. STANTEC shall observe and comply with all applicable laws, continue to provide equal employment opportunity to all qualified persons, and to recruit, hire, train, promote and compensate persons in all jobs without regard to race, color, religion, sex, age, disability or national origin or any other basis prohibited by applicable laws.

DISPUTE RESOLUTION: If requested in writing by either the CLIENT or STANTEC, the CLIENT and STANTEC shall attempt to resolve any dispute between them arising out of or in connection with this AGREEMENT by entering into structured non-binding negotiations with the assistance of a mediator on a without prejudice basis. The mediator shall be appointed by agreement of the parties. If a dispute cannot be settled within a period of thirty (30) calendar days with the mediator, if mutually agreed, the dispute shall be referred to arbitration pursuant to laws of the jurisdiction in which the majority of the SERVICES are performed or elsewhere by mutual agreement.

ASSIGNMENT: The CLIENT and STANTEC shall not, without the prior written consent of the other party, assign the benefit or in any way transfer the obligations under these Terms and Conditions or any part hereof.

SEVERABILITY: If any term, condition or covenant of the AGREEMENT is held by a court of competent jurisdiction to be invalid, void, or unenforceable, the remaining provisions of the AGREEMENT shall be binding on the CLIENT and STANTEC.

APPENDIX B
Site Plan



SITE INFORMATION:

SITE AREA = 52,724.47m² (527.25 ACRES)

UNIT COUNTS:

		APARTMENTS	URBAN TOWNS	TOWNS	B/B TERRACE HOMES
BLOCK 1 =	4 LEVEL APARTMENT BLDG. + 2 STOREY TOWNS	42 UNITS	6 UNITS		
BLOCK 2 =	4 LEVEL APARTMENT BLDG. + 2 STOREY TOWNS	42 UNITS	6 UNITS		
BLOCK 3 =	4 LEVEL APARTMENT BLDG. + 2 STOREY TOWNS	42 UNITS	6 UNITS		
BLOCK 4 =	4 LEVEL APARTMENT BLDG. + 2 STOREY TOWNS	40 UNITS	6 UNITS (w/ P2 PARKING)		
BLOCK 5 =	5 LEVEL APARTMENT BLDG. + 3 STOREY TOWNS	47 UNITS	7 UNITS		
BLOCK 6 =	5 LEVEL APARTMENT BLDG. + 3 STOREY TOWNS	77 UNITS	12 UNITS (w/ GARAGES)		
BLOCK 8 =	BACK / BACK TERRACE HOMES				12 UNITS
BLOCK 9 =	BACK / BACK TERRACE HOMES				16 UNITS
BLOCK 10 =	BACK / BACK TERRACE HOMES				16 UNITS
BLOCK 11 =	BACK / BACK TERRACE HOMES				16 UNITS
BLOCK 12 =	TOWNHOMES			7 UNITS	
BLOCK 13 =	TOWNHOMES			6 UNITS	
BLOCK 14 =	TOWNHOMES			6 UNITS	
BLOCK 15 =	TOWNHOMES			4 UNITS	
BLOCK 16 =	TOWNHOMES			4 UNITS	
BLOCK 17 =	TOWNHOMES			4 UNITS	
BLOCK 18 =	TOWNHOMES			6 UNITS	
TOTAL =		290 UNITS	43 UNITS	37 UNITS	60 UNITS

TOTAL ALL UNITS = 430 UNITS
(NOT INCLUDING FUTURE UNITS)

PARKING REQUIREMENTS:

BLOCKS 1-4 : APARTMENTS BDLG. + TOWNS [190 UNITS]
PARKING REQUIRED 1.2/d.u. + 0.2/d.u. VISITORS = 190 x 1.2 + .2 = 266 SPACES
PARKING PROVIDED = 266 SPACES (228 U/G, 38 SURFACE)

BLOCKS 5-6 : APARTMENTS BDLG. + TOWNS [124 UNITS (APT. ONLY)]
PARKING REQUIRED 1.2/d.u. + 0.2/d.u. VISITORS = 124 x 1.2 + .2 = 174 SPACES
PARKING PROVIDED = 174 SPACES (148 U/G, 26 SURFACE)

BLOCKS 8-11 : BACK TO BACK TERRACE HOMES (60 UNITS)
PARKING REQUIRED 1.2/d.u. + 0.2/d.u. VISITORS = 60 x 1.2 + .2 = 84 SPACES
PARKING PROVIDED = 88 SPACES (SURFACE)

LEGEND/ ABREVIATIONS:

D.C. DEPRESSED CURB
CRW CONCRETE RETAINING WALL
W.I. WROUGHT IRON
TWSI TACTILE WALKING SURFACE INDICATOR
CONC. CONCRETE
ASPH. ASPHALT
GAS METERS LOCATION
BUILDING SERVICES LOCATION (IN LOWER LEVEL)
WALL MOUNTED LIGHT FIXTURE
FIRE HYDRANT
L.S. LIGHT STANDARD
TWSI
SIAMESE CONNECTIONS
TRANSFORMER

5 20 50 100m
0 10 30 75

SITE PLAN TO BE READ IN CONJUNSTION WITH: SITE SERVICING PLAN PREPARED BY IBI GROUP
GRADING PLAN PREPARED BY IBI GROUP
LANDSCAPE PLAN PREPARED BY ____________

SITE BOUNDARIES DERIVED FROM: TOPOGRAPHICAL PLAN OF PART OF LOTS 27 AND 28 CONCESSION 1 (OLD SURVEY) GEOGRAPHIC TOWNSHIP OF CUMBERLAND CITY OF OTTAWA PREPARED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD. FIELD WORK COMPLETED ON FEBRUARY 8, 2017.

SITE BOUNDARIES FOR PART 1 & 2 DERIVED FROM: PLAN 4R-31597, DECEMBER 12, 2018.

M. David Blakely Architect Inc.
2200 Prince of Wales Dr. Suite 101 Ottawa, Ontario
Phone (613) 226-8811 Fax (613) 226-7942 k2E 6Z9

GENERAL NOTES:
1. THE CONTRACTOR IS RESPONSIBLE FOR CHECKING AND VERIFYING ALL DIMENSIONS, ANY DISCREPANCY MUST BE REPORTED TO M. DAVID BLAKELY ARCHITECT INC.
2. ALL WORK AND MATERIALS TO BE IN COMPLAINCE WITH ALL CODES, REGULATIONS, & BY-LAWS
3. ADDITIONAL DRAWINGS MAY BE ISSUED FOR CLARIFICATION TO ASSIST THE PROPER EXECUTION OF WORK. SUCH DRAWINGS WILL HAVE THE SAME MEANING AND INTENT AS IF THEY WERE INCLUDED WITH THE PLANS IN CONTRACT DOCUMENTS
4. DO NOT SCALE DRAWINGS.
5. THIS DRAWING SHALL NOT BE USED OR COPIED WITHOUT THE AUTHORIZATION OF THE ARCHITECT
6. THIS DRAWING SHALL NOT BE USED FOR PERMIT OR CONSTRUCTION UNLESS THE DRAWING BEARS THE ARCHITECT'S SEAL AND SIGNATURE

CONSTRUCTION NORTH

SEAL

REVISIONS

No.	DATE	DESCRIPTION	INIT.
12.	11/09/19	REVISED MODEL TYPES / LAYOUT	SM
11.	28/01/19	REVISED MODEL TYPES / LAYOUT	SM
10.	08/01/19	CONTOUR LINES ADDED	SM
9.	13/06/18	REVISED BLOCKS 1-4	SM
8.	05/04/18	ADDED SCALE TO SECTION	SM
7.	04/01/18	REVISED BLOCK LOCATIONS	SM
6.	28/09/17	REVISED SITE BOUNDARIES	SM
5.	20/09/17	REVISED SITE LAYOUT	SM
4.	05/07/17	REVISED UNIT TYPES	SM
3.	13/04/17	REVISED SITE LAYOUT	SM
2.	21/12/16	REVISED 36 UNIT BLOCK LAYOUT	SM
1.	30/11/16	FOR REVIEW	SM

REVISIONS

No.	DATE	DESCRIPTION	INIT.
24.			
23.			
22.			
21.			
20.			
19.	08/06/21	BLK. 8 CONVERTED TO TERR. HOMES	MB
18.	10/02/21	SIDEWALK @ PVT. STREET 2 DELETED	MB
17.	05/02/21	OLD. MTL. RD. GEOMETRY UPDATED	MB
16.	29/01/21	BOUNDARIES UPDATED/ PARTS 1&2	MB
15.	03/11/20	GEOTECHNICAL ADDED	MB
14.	13/07/20	BLKS. 5,6,& 8 REVISED/ GRADES REVISED	MB
13.	04/12/19	REVISED BLDG. FOOTPRINTS / LAYOUT	MB

A - DETAIL NUMBER
B - SHEET NUMBER (DETAIL REQUIRED)
C - SHEET NUMBER (DETAIL LOCATION)

PROJECT: PROPOSED SUBDIVISION OLD MONTREAL ROAD OTTAWA, ONTARIO.

DRAWING TITLE: SITE PLAN

CLIENT: PHOENIX HOMES 18A Bentley Ave Ottawa, ON K2E 6T8

DATE: NOV., 2016
SCALE: 1:500
SHEET No. REV No.: SP-1
DRAWN BY: SBM
CHECKED: MDB

IBI

Scale	Project Title	Drawing Title	Sheet No.
N.T.S.	DCR / PHOENIX 1208 OLD MONTREAL ROAD	CONCEPTUAL MACRO GRADING PLAN ----	FIG 5.1

APPENDIX C
Borehole Record

Log of Borehole BH-14

Project No: OTT-00234493-A0

Figure No. 15A

Project: Geotechnical Investigation. Proposed Residential Subdivision

Location: 1154, 1172, 1176, 1180, and 1208 Old Montreal Road, Ottawa, ON

Date Drilled: 08-26-21

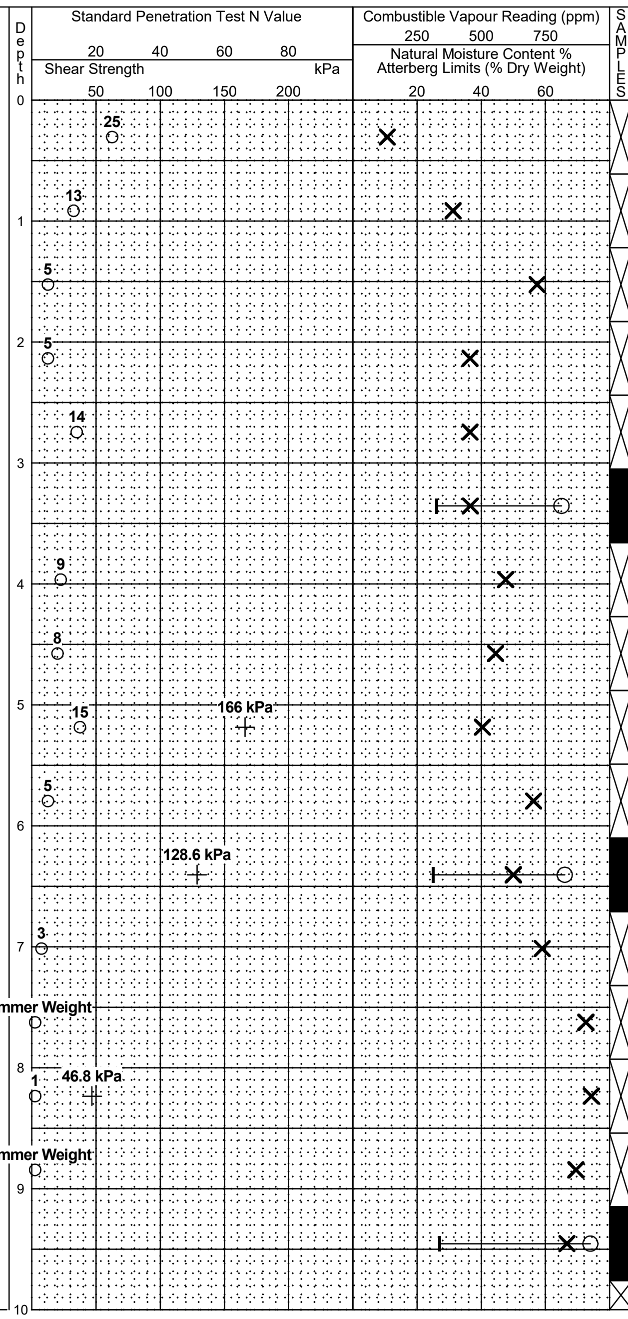
Drill Type: 'Rubber Trackmount CME 55

Datum: Geodetic

Logged by: ________ Checked by: ________

Split Spoon Sample	⊠	Combustible Vapour Reading	□
Auger Sample	◫	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
			84.556
		SAND AND GRAVEL FILL ~250mm some organics, brown, dry, compact	84.3
		SILTY SAND AND GRAVEL some clay, brown, red mottling, moist, (compact to loose)	82.7
		SILTY CLAY (DESSICATED CRUST) Greyish Brown to grey, moist, high plasticity, (hard)	79.1
		SILTY CLAY Grey, moist to wet, high plasticity, (very stiff to stiff)	



SAMPLES	Natural Unit Wt. kN/m³
SS1	19.1
SS2	18.9
SS3	19.6
SS4	18.4
SS5	18.6
ST1	
SS6	18.4
SS7	
SS8	
SS9	
ST2	
SS10	
SS11	
SS12	
SS13	
ST3	

Continued Next Page

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Nested 19 mm diameter piezometers were installed upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00234493-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Sept 9, 2021	3.2	(BH-14A)
Sept 9, 2021	15.1	(BH-14B)
Sept 21, 2021	2.3	(BH-14A)
Sept 21, 2021	4.8	(BH-14B)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-14

Project No: OTT-00234493-A0

Project: Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 15A

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value / Shear Strength (kPa)	Combustible Vapour Reading (ppm) / Natural Moisture Content % / Atterberg Limits (% Dry Weight)	SAMPLES	Natural Unit Wt. kN/m³
		SILTY CLAY Grey, moist to wet, high plasticity, (very stiff to stiff) *(continued)*	74.55	10	Hammer Weight; 65.5 kPa			SS14
					Hammer Weight			SS15
				11	1; 60.8 kPa			SS16
					Hammer Weight			SS17
				12	65.5 kPa			ST4
				13	1; 80.7 kPa			SS18
					Hammer Weight			SS19
				14	2; 93.5 kPa			SS20
				15	Hammer Weight			SS21
					Hammer Weight			SS22
				16	3; 86.5 kPa			SS23
					Hammer Weight			SS24
				17	3; 91.2 kPa			SS25
				18	Hammer Weight; 93.5 kPa			SS26
					Hammer Weight			SS27
				19	2; 91.2 kPa			SS28
					Hammer Weight			SS29
				20	1			SS30
				21	Hammer Weight			SS31
		GRAVELLY SANDY TILL some silt, some clay, grey, wet, compact to very dense	63.3		16			SS32
				22				

Continued Next Page

NOTES:

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4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00234493-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Sept 9, 2021	3.2	(BH-14A)
Sept 9, 2021	15.1	(BH-14B)
Sept 21, 2021	2.3	(BH-14A)
Sept 21, 2021	4.8	(BH-14B)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-14

Project No: OTT-00234493-A0

Project: Geotechnical Investigation. Proposed Residential Subdivision

Figure No. 15A

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value (20, 40, 60, 80) / Shear Strength kPa (50, 100, 150, 200)	Combustible Vapour Reading (ppm) (250, 500, 750) / Natural Moisture Content % / Atterberg Limits (% Dry Weight) (20, 40, 60)	SAMPLES	Natural Unit Wt. kN/m^3
			62.556	22				
		GRAVELLY SANDY TILL some silt, some clay, grey, wet, compact to very dense *(continued)*			>100			SS33
					>100			SS34
			61.4	23				
		Borehole Terminated at 23 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Nested 19 mm diameter piezometers were installed upon completion.
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4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-00234493-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Sept 9, 2021	3.2	(BH-14A)
Sept 9, 2021	15.1	(BH-14B)
Sept 21, 2021	2.3	(BH-14A)
Sept 21, 2021	4.8	(BH-14B)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

APPENDIX D
NBCC 2015 Earthquake Hazard Sheet

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: 45.495N 75.464W **User File Reference:** Old Montreal Road 2021-10-05 12:19 UT

Requested by: GR, Stantec

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)	0.520	0.286	0.170	0.049
Sa (0.1)	0.602	0.342	0.210	0.067
Sa (0.2)	0.495	0.286	0.179	0.060
Sa (0.3)	0.372	0.216	0.137	0.047
Sa (0.5)	0.261	0.151	0.095	0.033
Sa (1.0)	0.127	0.074	0.047	0.016
Sa (2.0)	0.059	0.034	0.021	0.006
Sa (5.0)	0.016	0.008	0.005	0.001
Sa (10.0)	0.006	0.003	0.002	0.001
PGA (g)	0.318	0.185	0.114	0.036
PGV (m/s)	0.216	0.121	0.074	0.023

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