Humanics Universal Inc.

Geotechnical Investigation and Slope Stability Assessment

Type of Document
Final (5th Revision)

Project Name
Proposed Humanics Sanctuary and Sculpture Park
3400 Old Montreal Road
Part of Lot 7, Concession 1, Old Survey
Ottawa (Formerly Township of Cumberland), Ontario

Project Number
OTT-00229886-A0

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Date Submitted
September 26, 2023 (supersede May 1 4th revision report)

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Attention: Dr. Ranjit Perera

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Executive Summary

A geotechnical investigation was undertaken at the site of the proposed Humanics Sanctuary and Sculpture Park to be located on the south side of 3400 Old Montreal Road in the City of Ottawa, Ontario. This work was authorized by Dr. Ranjit Perera on November 15, 2015.

The purpose of 3rd revision was to update Figure No. 2 to reflect the new limit of the environmental set back line established by Bernie Muncaster's in his letter on April 14, 2022, which adjust the line at the east portion of the south slope. All other items on this report remained unchanged. Bernie Muncatser's letter is attached as Appendix C of this report. The purpose of this current 4th revision was to update the report to address the March 21, 2023, comments received by the City of Ottawa and to reflect the new site plan proposed by the client and design team.

The proposed development would comprise of a hostel, residential building, sanctuary building, education centre, and designated areas across the parkland for display of various sculptures, which will be constructed over the years and in phases. The proposed buildings will be one- to three-storeys high with one basement level.

The geotechnical investigation comprised of drilling four boreholes to 18.9 m to 19.8 m depth and excavating 10 test pits to 3.4 m to 3.7 m depth. The boreholes and test pits revealed that beneath 150 mm to 355 mm of topsoil, silty sand extends to 0.9 m to 1.5 m depth. The sand is underlain by clay, which extends to the entire depth investigated in the test pits and boreholes, i.e., 3.4 m to 19.8 m depth. The clay is firm to very stiff with some localized soft clay layers (undrained shear strength of 19 kPa to 180 kPa). The clay at the site is over-consolidated by 130 kPa to 135 kPa based on the consolidation tests undertaken on the Humanics subdivision site located south of the subject property.

It is recommended that the maximum grade raise at the site should be limited to 2 m to prevent over-stressing of the clay.

The investigation has revealed that the geotechnical conditions at the site are suitable for construction of the one- to three-storey buildings on spread and strip footing foundations. It is recommended that the footings should be founded above the groundwater table (i.e., at a maximum depth of 2.2 m below the existing ground surface) and designed for Serviceability Limit State (SLS) bearing pressure of 95 kPa and factored geotechnical resistance at Ultimate Limit State (ULS) of 140 kPa.

The lowest level floor slabs of the structures may be constructed as slabs-on-grade. Perimeter as well as underfloor drains should be provided for the structures with basements.

Excavations at the site will be undertaken to a maximum depth of 2.2 m below the existing ground surface and will be above the groundwater table. The excavations may be undertaken as open cut provided they are cut back at 45 degrees. Minor seepage of surface and subsurface water into the excavations should be anticipated. However, it should be possible to collect this water in perimeter ditches and remove by pumping from sumps. The backfill against the subsurface walls should be free draining granular materials conforming to OPSS 1090 for Granular B, Type II. It should be compacted to 95 percent of standard Proctor Maximum Dry Density (SPMDD).

The pavement structure of the gravel parking lot(s) may consist of 200 mm of Granular A base underlain by 550 mm of Granular B, Type II sub-base. Unpaved access roads may consist of 250 mm of Granular A base underlain by 550 mm of Granular B, Type II sub-base.

General Use (GU) Portland cement may be used in the subsurface structures at the site.

The site has been classified as Class D for seismic site response.

Trees should not be planted in close proximity of the structures to prevent settlements due to shrinkage of the clay as a result of water extraction by tree roots. Planting of trees at the site should conform to the City of Ottawa, 2017 Guidelines on Tree Planting in Sensitive Marine Clay Soils.

Slope stability analysis was undertaken to compute the slope inclination, which will have a minimum factor of safety of 1.5. This information was used to compute geotechnical set back. To this was added the erosion allowance and erosion access allowance to arrive at limit of hazardous lands. These limits have been plotted on the Site Plan, Figure 2.

It is noted that a pathway was constructed approximately 10 years ago part way up the slope by cutting into the slope on the east and west side of the creek. This has resulted in steepening and caving of the upper slope and erosion of the downhill slope at random locations along the pathway. These areas should be remediated as discussed in Section 16 of this report.

Table of Contents

Page

List of Tables

Page

List of Figures

List of Appendices

1 Introduction

A geotechnical investigation was undertaken at the site of the proposed Humanics Sanctuary and Sculpture Park to be located on the south side of 3400 Old Montreal Road in the City of Ottawa, Ontario. The site forms part of Lot 7, Concession 1 in the former Township of Cumberland, now City of Ottawa, Plan 4R-2542 (Figure 1). This work was authorized by Dr. Ranjit Perera on November 15, 2015.

It is proposed to construct a hotel, residential building, sanctuary building, education centre and designated areas across the parkland for display of various sculptures to be constructed over the years and in phases. The proposed buildings will be one- to three-storeys high with one basement level. The proposed development will be supported by private well and septic tile bed and storm sewers.

The purpose of 3rd revision issued in November 2022 was to update Figure No. 2 to reflect the new limit of the environmental set back line established by Bernie Muncaster's in his letter on April 14, 2022, which adjust the line at the east portion of the south slope. All other items on this report remained unchanged. Bernie Muncatser's letter is attached as Appendix C of this report. The purpose of this current 4th revision was to update the report to address the comments received in March 21, 2023 by the City of Ottawa/RVCA and to reflect the new site plan proposed by the client and design team.

The investigation was undertaken to:

a) Establish geotechnical and groundwater profile at the site;

b) Comment on maximum grade raise feasible at the site;

c) Make recommendations regarding the most suitable type of foundations, founding depth and allowable bearing pressure of founding soil;

d) Determine anticipated settlements;

e) Classify the site for seismic site response in accordance with the requirements of Ontario Building Code, 2012.

f) Comment on excavation conditions;

g) Discuss backfilling requirements and suitability of on-site soils for backfilling purposes;

h) Recommend pavement structure thickness for access roads and parking areas;

i) Comment on subsurface concrete requirements; and

j) Assess the stability of the slopes to the creek that crosses the site and establish the limit of hazardous lands.

The comments and recommendations given in this report are based on the assumption that the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2 Procedure

The geotechnical investigation was completed between November 30 and December 3, 2015. It comprised the drilling of four boreholes (Boreholes 1 to 4) to depths ranging between 18.9 m to 19.8 m. The locations of the boreholes are shown on Site Plan, Figure 2.

The fieldwork was undertaken with a track-mounted drill rig equipped with continuous flight hollow stem augers and with a backhoe. It was supervised on a full-time basis by a representative of EXP. A dynamic cone penetration test was then performed in the bottom of each borehole to refusal at 30.2 m to 34.8 m depth.

Standard penetration tests were performed in all the boreholes at 0.75 m to 3.0 m depth intervals and soil samples retrieved by split barrel sampler. The undrained shear strength of the clay was established by in-situ field-vane shear tests.

Water levels were measured in the open boreholes on completion of drilling. In addition, long-term groundwater monitoring installations consisting of 13 mm diameter PVC (polyvinyl chloride) pipes were placed in Borehole Nos. 1 and 4. The installation configuration is documented on the respective borehole logs. All the boreholes were backfilled upon completion of the fieldwork. The locations of the boreholes were established by a representative of EXP using GPS technology and therefore are approximate. The elevations of the boreholes were determined based on the contours shown on the Lidar Survey Plan.

All the soil samples were visually examined in the field for textural classification, logged, preserved in plastic bags and identified. On completion of the fieldwork, all the soil samples were transported to the EXP laboratory in the City of Ottawa, Ontario.

All the soil samples were visually examined in the laboratory by a geotechnical engineer and borehole logs prepared. The engineer also assigned the laboratory testing which consisted of performing natural moisture content, unit weight, grain-size analysis, Atterberg Limit, pH and sulphate content tests on selected soil samples.

In addition to the above investigation, a total of 12 test pits were excavated as part of a hydrogeological study by EXP in November 2015. The approximate locations of these test pits are also shown on Figure 2. The test pit logs have been presented on Table I.

The slopes at the site were visually examined during the fieldwork and subsequently in spring of 2022 to determine erosion areas as a result of construction of pathways at the site.

3 Site and Soil Description

The site of the proposed development is located on the south side of Old Montreal Road at 3400 Old Montreal Road in the City of Ottawa, Ontario (Figure 1). The site forms Part of Lot 7, Concession 1 in the former Township of Cumberland, now the City of Ottawa.

A ravine is located in the central part of the property and runs in a northwest to southeast direction. It then swings to the north and drains into the Ottawa River. The slopes to the creek are 9.5 m to 13 m high at an inclination of 2.7H:1V to 3.2H:1V. The slopes are covered with vegetation.

A detailed description of the geotechnical conditions encountered in the four boreholes drilled and 12 test pits excavated at the site is given on the borehole logs, Figures 3 to 6 inclusive and on the test pit logs, Table I. The borehole and test pit logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted. Boreholes were drilled and test pits excavated to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions.

Table I: Test Pit Logs

Test Pit No.	Depth (m)	Soil Description	Grain-size Analysis (%)		
			Clay	Silt	Sand
1	0 – 0.15	Topsoil			
	0.15 – 1.2	Sand	0	0	100
	1.2 – 3.7	Clay			
		Test pit terminated at 3.7 m depth. Water seepage into test pit at 2.6 m depth. Backfilled.			
2	0 – 0.2	Topsoil			
	0.2 – 0.9	Silt	0	98	2
	0.0 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 3.0 m depth. Backfilled.			
3	0 – 0.22	Topsoil			
	0.22 – 1.2	Sand			
	1.2 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Water seepage into test pit at 2.7 m depth. Backfilled.			
4	0 – 0.3	Topsoil			
	0.3 – 1.4	Sand			
	1.4 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 2.7 m depth. Backfilled.			

Table I: Test Pit Logs

Test Pit No.	Depth (m)	Soil Description	Grain-size Analysis (%)		
			Clay	Silt	Sand
5	0 – 0.3	Topsoil			
	0.3 – 0.9	Sand			
	0.9 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Water seepage into test pit at 2.7 m depth. Backfilled.			
6	0 – 0.3	Topsoil			
	0.3 – 0.9	Sand			
	0.9 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 2.7 m depth. Backfilled.			
7	0 – 0.3	Topsoil			
	0.3 – 0.9	Sand			
	0.9 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 2.7 m depth. Backfilled.			
8	0 – 0.25	Topsoil			
	0.25 – 1.2	Sand			
	1.2 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 2.7 m depth. Backfilled.			
9	0 – 0.25	Topsoil			
	0.25 – 1.2	Sand	0	0	100
	1.2 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 2.7 m depth. Backfilled.			
10	0 – 0.36	Topsoil			
	0.36 – 1.2	Sand			
	1.2 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 2.4 m depth. Backfilled.			
11	0 – 0.2	Topsoil			
	0.2 – 1.2	Sand			
	1.2 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 2.4 m depth. Backfilled.			
12	0 – 0.2	Topsoil			
	0.2 – 1.2	Sand			
	1.2 – 3.4	Clay			
		Test pit terminated at 3.4 m depth. Wet at 2.4 m depth. Backfilled.			

A review of Table I and Figures 3 to 6 reveals the following soil stratigraphy with depth in descending order.

3.1 Topsoil

A 150 mm to 355 mm of topsoil was encountered at the locations of Borehole No. 4 and at the locations of all the test pits.

3.2 Fill

Approximately 0.6 m of fill was encountered in Borehole 1. The fill comprised of crushed run limestone to sand and gravel. The fill is loose (‘N’ value = 7).

3.3 Silt and Sand

The surficial soil in Boreholes 2, 3, and 4 and beneath the topsoil in all the test pits, silt and fine to medium sand was encountered, which extends to a depth of 0.9 m to 1.5 m. The silt and sand are very loose to loose (‘N’ values of 2 to 9). Its moisture content varies from 10 to 48 percent. Three grain-size analyses performed on silt and sand samples retrieved from the test pits yielded a soil composition of 0 to 100 percent silt and 0 to 100 percent sand (Figures 7 to 9 inclusive).

3.4 Clay

The fill in Borehole 1 and the silt and/or the sand stratum in all the other boreholes and test pits are underlain by clay, which extends to the entire depths investigated in all the test pits and boreholes, i.e., 3.4 m to 19.8 m depth. The clay is very stiff to soft as indicated by its undrained shear strength, which varies from 180 kPa to 19 kPa. It is noted that shear strength of 19 kPa recorded in Borehole No. 3 at 4.5 m depth is localized and likely the result of disturbance from augering. The sensitivity of the clay varied from 1.7 to 10.2, which indicates clay of low sensitivity to extra sensitive clay based on the references contained in the Canadian Foundation Engineering Manual, 4th Edition, 2006. It is noted that extra sensitive clay was encountered in Borehole 4 only. However, no construction is to be undertaken in the vicinity of Borehole 4. It is noted that construction in the sensitive clay in the Ottawa area is quite common and is successfully undertaken without causing liquefaction of the clay. Therefore, in our opinion, risk of undertaking construction on the subject site is low. This type of clay is formed from the disposition of fine-grained sediments in marine environment with low oxygen levels. The natural moisture content of the clay varies from 30 to 96 percent. It has a unit weight of 16.1 to 17.8 kN/m^3.

Three grain-size analyses performed on the clay samples yielded a composition of 71 to 73 percent clay, 25 to 29 percent silt and 0 to 2 percent sand (Figures 10 to 12).

Atterberg Limit tests performed on the clay samples indicated its plastic limit as 25 to 26 percent and liquid limit of 40 to 56 percent. The results of the Atterberg Limit tests have been plotted on Figure 13. The test results indicate that the on-site silty clay is inorganic and of medium to high plasticity.

3.5 Liquefaction Potential of On-Site Clay

The liquefaction potential of the clay during a seismic event was assessed by three methods, i.e., Modified Chinese Criteria (Seed and Idriss, 1982)[1], Andrews and Martin Criteria (2000) [2], and Polito Criteria (Polito, 2001)[3]. The results of these analyses have been plotted on Figures 14a, 14b and 14c. The Thresholds to Liquefaction (After Jennings, 1980) and Plasticity Based Liquefaction Criteria (Polito C, 2001)[4] are given below.

A review of Figures 14a, 14b and 14c indicates that the analysis by three different criteria conclude that the on-site clay is not susceptible to liquefaction during a seismic event.

Thresholds to Liquefaction

Table 1:Thresholds to Liquefaction (After Jennings, 1980)

Condition	Threshold
Mean grain size (mm)	$0.02 < D_{50} < 1.0$
Clay particle content (percent)	10 <
Uniformity coefficient	10 <
Relative density (percent)	75 <
Void Ratio	> 0.80
Plasticity index (percent)	< 10
Depth to water table (m)	< 5
Depth to sand layer (m)	< 20

Table 2: Modifications to Plasticity Based Liquefaction Criteria

Author	Proposed Criteria/Modifications
Seed et al. (1983)	Percent finer than 0.005 mm < 15% Liquid Limit, LL < 35% Water content at least 90% of LL
Finn et al. (1994) modifying Seed et al. (1983)	Decrease fines content by 5% Decrease liquid limit by 2% Increase water content by 2%
Koester (1994) modifying Seed et al. (1983)	Decrease fines content by 5% Increase liquid limit by 1% Decrease water content by 2%

Reference: Polito C. (2001). Plasticity Based Liquefaction Criteria. Missouri University of Science and Technology. 2001 - Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics.

Uniformity coefficient $C_u = d_{60} / d_{10}$
Porosity $n = 0.255 * (1 + 0.83^{Cu})$
Void ratio $e = 1 / (1 - n)$

Thresholds to Liquefaction

Condition		Threshold	Liquefiable	OTT-00229886-A0
Mean grain size	(mm)	(0.02, 1.0)	no	< 0.001
Clay content (< 5 µm)	(%)	< 15	no	> 72
Uniformity coefficient		< 10		N.A.
Relative density	(%)	< 75	yes	0
Void Ratio		> 0.80		#VALUE!
Liquid limit	(%)	< 35	no	46
Plasticity index	(%)	< 10	no	21
Water content	(%)	> 42	yes	72
Depth to water table	(m)	< 5	no	5
Depth to sand layer	(m)	< 20		N.A.

Key parameter - %clay, PI & relative density

The geotechnical properties of the soil strata established have been summarized on Table II.

[1] Seed, H.B. and Idriss, I.M. (1982). Ground Motions and Soil Liquefaction during Earthquakes. Earthquake Engineering Research Institute, Berkeley, University of California
[2] Andrews, D.C.A. and Martin, G.R. (2000). Criteria for liquefaction of silty soils. Proc. 12th World Conference on Earthquake Engineering, Auckland, New Zealand. Paper 0312
[3] Polito C. (2001). Plasticity Based Liquefaction Criteria. Missouri University of Science and Technology. 2001 - Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
[4] Polito C. (2001). Plasticity Based Liquefaction Criteria. Missouri University of Science and Technology. 2001 - Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

Table II: Summary of Geotechnical Properties of Soils

Type of Soil	Moisture Content	γ (kN/m^3)	Undrained Shear Strength (kPa)	Atterberg Limit Test Results			Grain-size Analyses (%)				N Values
				Natural Moisture Content (%)	Plastic Limit (%)	Liquid Limit (%)	Clay	Silt	Sand	Gravel	
Fill	16										7
Silty Sand	10-48						--	0 - 100	0 - 100	--	2 - 9
Clay	30-96	17.8-16.1	180 – 19	67 – 74	25 – 26	40 – 56	71 – 73	25 – 29	0 – 2	--	HW* - 12

** HW denotes Hammer Weight*

3.6 Consolidation Characteristics of Clay

Two one-dimensional oedometer tests were performed on clay samples from two boreholes drilled as part of the Humanics' subdivision located south of the site and reported as part of EXP Report OTT-00019355-A0 dated February 2, 2016. The test results have been included in Appendix A and have been tabulated on Table III below.

Table III: Results of Consolidation Tests

Borehole No.	Sample Depth	Effective Overburden Pressure p_o' (kPa)	Effective Consolidation Pressure p_c'(kPa)	Compression Index (C_c)	Re-Compression Index (C_r)	Over-consolidation Pressure (kPa)
2	3.0 – 3.6	45	175	0.957	0.086	130
1	4.6 – 5.2	55	190	1.014	0.0864	135

A review of Table III indicates that the sample from Borehole No. 2 is pre-consolidated to 175 kPa, whereas its effective overburden pressure is 45 kPa. Therefore, this sample is over consolidated by 130 kPa (pre-consolidation pressure minus effective overburden pressure). The sample from Borehole No. 1 is pre-consolidated to 190 kPa. Its effective overburden pressure is 55 kPa. Therefore, this sample is over consolidated by 135 kPa. The compression index (C_c) of the clay varies from 0.957 to 1.014. Its re-compression index (C_r) is 0.086.

3.7 Bedrock

A dynamic cone penetration test performed in the bottom of each borehole met refusal at 30.2 m to 34.8 m depth. This refusal is likely to have been met on the bedrock. Available geological information indicates that the bedrock in the areas is shale of Rockcliffe Formation.

3.8 Groundwater Observations

Water level observations were made in the boreholes during drilling and after the completion of drilling in standpipes installed in Borehole Nos. 1 to 4. The groundwater observations made to date are given on Table IV.

Table IV: Groundwater Observations in Boreholes

Borehole No.	Date Drilled	Observation Date	Groundwater Depth (m)	Elevation of Groundwater Table (m)
1	November 30, 2015	December 14, 2015	5.3	55.3
		April 20, 2016	2.2	58.4
2	December 3 & 04, 2015	-	-	-
		April 20, 2016	4.3	58.0
3	December 3, 2015	-	-	-
		April 20, 2016	3.7	58.8
4	November 31, 2015	December 14, 2015	5.0 (not stabilized)	54.6
		April 20, 2016	Seal broken	

A review of Table IV indicates that the groundwater table at the site was measured at depths of 2.2 m to 4.3 m (Elevations 58.0 m to 58.8) approximately fifteen and one hundred thirty-six days following the completion of the drilling. Water level measurement was taken by lowering a co-axial cable attached to an ohm meter into the standpipe installed in the borehole. When the tip of the co-axial cable touches the water level, the circuit is completed and the ohm meter registers a deflection. Had the water in the borehole been frozen, it would not have been possible to lower the co-axial cable into the borehole beyond this point. However, when taking the water level measurement, it was possible to lower the co-axial cable beyond the depth at which deflection was observed, thereby indicating that the groundwater was not frozen. Therefore, in our opinion, the water level reading recorded represents the high groundwater table which would be expected during spring.

Water levels were determined in the boreholes at the times and under the conditions stated in the scope of services. Note that fluctuations in the level of groundwater may occur due to a seasonal variation such as precipitation, snowmelt, rainfall activities, and other factors not evident at the time of measurement and therefore may be at a higher level during wet weather periods.

4 Site Re-grading

It is noted that the site contains a deep deposit of clay (in the order of 30 m to 35 m) and that the clay at the site is prone to consolidation settlements if fill is placed on the site. This may result in settlements and cracking of any structures founded in the clay if the clay is overstressed. To evaluate if the grade at the site can be raised, two one-dimensional oedometer tests undertaken on the clay samples collected from the boreholes drilled as part of the Humanics' subdivision located south of the site and reported as part of EXP Report OTT-00019355-A0 dated February 2, 2016, were used. The test results summarized in Table III indicate that clay is over-consolidated by 130 kPa to 135 kPa. It is therefore considered that the additional load that can be applied on the clay for the settlements to be within normally tolerated limits of 25 mm total and 19 mm differential is in the order of 100 kPa (assumed 80 percent of over-consolidation pressure).

It has been stated in Section 5 of the report that footings of the proposed residences should be set above the groundwater table. As such, lowering of the groundwater table at the site will not result due to proposed development. Therefore, an allowance for groundwater lowering is not required. Allowing for the Serviceability Limit State (SLS) bearing pressure recommended in Section 5, it is considered that the grade at the site may be raised by up to 2.0 m.

The site grading plan prepared by EXP under Project Number OTT-00229886-A0 Drawing SGP dated January 25, 2017, was reviewed. It indicates that the proposed grade raises are less than permitted in the geotechnical investigation report for this project and therefore meet the requirements of the geotechnical investigation.

5 Foundation Considerations

The investigation has revealed that the geotechnical conditions at the site are suitable for construction of the proposed one- to two-storey structures with one level of basement on spread and strip footing foundations. As required by the City of Ottawa, it is recommended that the footings of the proposed structures should be set above the groundwater table, i.e., at a maximum depth of 2.2 m below the existing ground surface. Footings founded on the clay below any loose silty sand at a maximum depth of 2.2 m below the existing ground surface may be designed for Serviceability Limit State (SLS) bearing pressure of 95 kPa and factored geotechnical resistance at Ultimate Limit State (ULS) of 140 kPa. To compute the SLS bearing pressure of the soil, the undrained shear strength was reduced by 15 percent in accordance with Section 6.6.3.2 (7) of the Canadian Foundation Engineering Manual.

The recommended bearing capacities have been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes, when foundation construction is underway. The interpretation between boreholes, and the recommendations of this report must therefore be checked through field monitoring provided by an experienced geotechnical engineer to validate the information for use during the construction stage.

A minimum of 1.5 m of earth cover should be provided to all the exterior footings of heated structures to protect them from damage due to frost penetration. Where earth cover is less than 1.5 m, an equivalent combination of earth fill and rigid polystyrene insulation (i.e. Styrofoam HI-40) should be provided. Footings of unheated structure should be provided with a cover of 2.1 m if snow would not be cleared from their vicinity. If the snow would be cleared from the vicinity of the footings, they should be provided with 2.4 m of earth cover.

All the footing beds should be examined by a geotechnical engineer/geotechnician to ensure that the founding soil is capable of supporting the design bearing pressure and that the footings beds have been prepared satisfactorily.

Settlements of the structures founded on strip and spread footings design according to the above recommendations are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential movements.

6 Floor Slab and Drainage Requirements

The lowest level floors of the proposed structures may be constructed as slabs-on-grade provided they are set on beds of well-compacted 19 mm clear stone at least 200 mm thick placed on the natural soil or on well-compacted fill. The clear stone would prevent the capillary rise of moisture from the sub-soil to the floor slab. Adequate saw cuts should be provided in the floor slabs to control cracking. Any underfloor fill required should conform to OPSS 1010 for Granular B, Type II and should be placed in 300 mm lift thickness and each lift compacted to at least 98 percent of the standard Proctor maximum dry density (SPMDD).

Perimeter as well as underfloor drains should be provided for structures with basements (Figure 14). The perimeter and sub-surface drains for the structure will be connected to storm sewers. All subsurface walls should be properly damp-proofed. The exterior grade should be sloped away from the structures at an inclination of 1 to 2 percent to prevent the ingress of surface runoff.

7 Lateral Earth Pressure Against Subsurface Walls

The subsurface walls should be backfilled with free draining material, such as OPSS 1010 for Granular B, Type II and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the walls. The walls will be subjected to lateral static and dynamic (seismic) earth forces.

For design purposes, the lateral static earth thrust against the subsurface walls may be computed from the following equation:

$$P = K_0 H (q + \tfrac{1}{2} \gamma H)$$

where

- P = lateral earth thrust acting on the subsurface wall; kN/m
- K_0 = lateral earth pressure coefficient for 'at rest' condition for Granular B Type II backfill material = 0.5
- γ = unit weight of free draining granular backfill; Granular B = 22 kN/m^3
- H = Height of backfill adjacent to foundation wall, m
- q = surcharge load, kPa

The lateral seismic thrust may be computed from the equation given below:

$$\Delta P_E = 0.3 \gamma H^2$$

where

- ΔP_E = resultant thrust due to seismic activity; kN/m
- γ = unit weight of free draining granular backfill; Granular B Type II = 22 kN/m^3
- H = height of backfill behind wall, (m)

The ΔPE value does not take into account the surcharge load. The resultant load should be assumed to act at 0.6 H from the bottom of the wall.

8 Excavations

Excavations for construction of spread and strip footings are expected to extend to a maximum depth of 2.2 m below the existing ground surfaces and may be deeper if engineered fill is placed to raise the grade. These excavations are expected to terminate in the clay. They are expected to be above the groundwater table.

Excavations above the groundwater table in the engineered grade raise fill, silty sand and clay are expected to be stable when cut back at 45 degrees.

Seepage of surface and subsurface water into the excavations is anticipated. However, it should be possible to collect any water entering the excavations in perimeter ditches and to remove it by pumping from sumps. Although this investigation has estimated the groundwater levels at the time of the field work, and commented on dewatering and general construction problems, conditions may be present which are difficult to establish from standard boring techniques and which may affect the type and nature of dewatering procedures used by the contractor in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile, thin layers of soil with large or small permeabilities compared with the soil mass, etc. Only carefully controlled tests using pumped wells and observation wells will yield the quantitative data on groundwater volumes and pressures that are necessary to engineer construction dewatering systems adequately.

Many geologic materials deteriorate rapidly upon exposure to meteorological elements. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from moisture, desiccation, and frost action throughout the course of construction.

The clay at the site is susceptible to disturbance due to the movement of construction equipment, and personnel on its surface. It is therefore recommended that the excavation at the site should be undertaken by equipment, which does not travel on the excavated surface, e.g., a gradall, or mechanical shovel. It is anticipated that temporary granular roads may be required to gain access to the site. A 50-mm mud slab is also recommended to be placed on top of the founding clay immediately following approval.

It is anticipated that Permit to Take Water will not be required since the excavations will not extend below the groundwater table.

9 Backfilling Requirements and Suitability of On-Site Soils for Backfilling Purposes

The backfill against subsurface walls should consist of free draining material preferably conforming to OPSS 1010 for Granular B, Type II. It should be compacted to 95 percent of the SPMDD.

The backfill in footing trenches and service trenches should be compactible i.e. free of organics and debris and with natural moisture content, which is within 2 percent of the optimum moisture content. It should be compacted to 95 percent of the SPMDD.

The material to be excavated during construction of the footings and installation of services is silty sand and clay. These materials are generally too wet for adequate compaction and should be only used for general grading purposes. Any fill imported for backfill footing trenches, service trenches, etc. should preferably conform to OPSS 1010 for Granular B, Type II. It should be placed in 300 mm lift thickness and compacted to 95 percent of the SPMDD. In this case, clay dykes should be provided at upstream end of the manholes to prevent drainage of the granular backfill and lowering of the groundwater table. The clay dykes should be constructed in accordance with OPSD 802.095 requirements.

10 Access Roads

Pavement structure thicknesses required for unpaved parking areas and access roads were computed. The pavement structure is shown on Table V. The thicknesses are based upon an estimate of the subgrade soil properties determined from visual examination, textural classification and grain-size analyses of the soil samples and functional design life of 25 to 30 years. The proposed functional design life represents the number of years to the first rehabilitation, assuming regular maintenance is carried out. Gravel surfaced facilities are expected to require regular maintenance.

Table V: Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Parking Lot(s)	Access Road(s)
		Unpaved	Unpaved
OPSS 1010 Granular 'A' Base (crushed limestone)	100% SPMDD*	200 mm	250
OPSS 1010 Granular 'B', Type II Sub-base	100% SPMDD*	550 mm	550
Subgrade	Engineered fill/approved fill as per specifications		

Notes:
SPMDD denotes standard Proctor maximum dry density, ASTM, D-698-12e2
The upper 300 mm of the subgrade fill must be compacted to 98% SPMDD.

An existing access roadway is located on the site. However, its structure is not known. It is understood that this road is to be upgraded so that the recommended roadway structure is met.

Construction procedures for the pavement structure are discussed below.

The entire road should be excavated to the subgrade level. The subgrade should be crowned with a centre to edge slope of at least 2 percent. It should then be proof rolled with a heavy roller. Any soft areas that become evident should be sub-excavated and replaced with approved native fill or free draining granular material. All subgrade fill should be placed in maximum 300 mm lifts and compacted to 95 percent of SPMDD. In-place density tests should be performed at regular intervals to ensure that the specified degree of compaction is being achieved.

It is stressed that the overall satisfactory performance of the recommended pavement structure is contingent upon the provisions of good drainage. It is therefore recommended that roadside ditches should be provided on both sides of the pavement to drain the subgrade. Granular infiltration trenches may be located beneath the ditches to reduce run-off volumes and promote groundwater infiltration. The infiltration trenches will drain to a positive outlet.

Additional comments on construction of the roadways are as follows:

1.) The most severe loading conditions on pavement subgrades may occur during construction. Consequently, special provisions such as restricted lanes, may be required, especially if construction is carried out during wet weather conditions, i.e. during spring and fall seasons

2.) The finished pavement surface should be free of depressions and should be sloped to provide effective surface drainage towards catch basins, the drainage ditches and roadside subdrain. Surface water should not be allowed to pond adjacent to the outside edges of paved areas.

3.) The granular materials used for pavement construction should conform to OPSS 1010 for Granular A and Granular B, Type II and should be compacted to 100 percent of the SPMDD.

It is recommended that a geotechnical engineer be retained to review the final pavement structure design at construction to ensure that it is consistent with the recommendations of this report.

11 Subsurface Concrete Requirements

Subsurface concrete will be used to construct basements of the residences. Chemical tests limited to pH and sulphate tests were performed on two selected soil samples. The results are given on Table VI.

Table VI: Chemical Test Results

Borehole No.	Depth	pH	Sulphate (%)
3	1.5 – 2.1	6.88	0.0012
4	2.1 – 2.7	6.44	0.0009

The test results indicate the clay contains a sulphate content of less than 0.1 percent. This concentration of sulphates in the soil would have a negligible potential of sulphate attack on subsurface concrete. It is therefore considered that normal Portland cement may be used in the basement walls of the residences. The concrete for the site should be designed in accordance with the requirements of CSA A23.1-14.

12 Seismic Site Classification and Liquefaction Potential of On-Site Soils

The surficial soil at the site is silty sand, which is underlain by silty clay. The silty sand is located above the groundwater table and as such is not liquefiable during a seismic event.

The investigation also revealed that the predominant soil at the site is clay, which extends to 30.2 m to 34.8 m depth. The clay is expected to be underlain by bedrock at this level. The average shear strength of the clay was computed as 73 kPa which indicates that the clay is stiff.

The liquefaction potential of the clay on the site was assessed by plotting the results of laboratory tests on Bray et al (2004), Andrews and Martin (2000) and Bray and Sancio (2006), plots for assessment of liquefaction susceptibility. A review of Figures 14 to 14B indicates that with the exception of the Bray et al criteria, both the other plots indicate that the clay is non-liquefiable. It is noted that Bray et al (2004) has been superseded by Bray and Sancio (2006) criteria. Therefore, it is considered that the clay at the site is not susceptible to liquefaction during a seismic event.

The site has been classified as Class D for seismic site response in accordance with Table 4.1.8.4 (A) of the Ontario Building Code, 2012.

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13 Tree Planting

The clay in the Ottawa area is prone to shrinkage on drying. This process is largely not reversible. Therefore, settlement and cracking of the structures can result if trees are planted too close to the residences. During dry seasons, the tree roots draw moisture from the clay thereby resulting in the clay drying and shrinking. Therefore, planting of the trees at the site must comply to the City of Ottawa “The Tree Planting in Sensitive Marine Clay Soils” 2017 guidelines. It states the following.

For street trees in the road right-of-way where SMC soils have been identified, the tree to foundation setbacks may be reduced to 4.5 m for small (mature tree height up to 7.5 m) and medium size trees (mature tree height 7.5 m to 14 m) provided all of the following six conditions are met:

1. The modified plasticity index of the soil between the underside of footing (USF) and a depth of 3.5 m generally does not exceed 40%. This corresponds to soils with low/medium potential for soil volume change. Clay soils that exceed the 40% plasticity index are considered to have high potential for soil volume change. For these worst-case soils, the setbacks and tree planting restrictions remain unchanged from the 2005 Clay Soils Policy (tree setback must equal the mature height of the tree – i.e., 7.5 m setback for small trees).

2. The USF is 2.1 m or greater below the lowest finished grade. Note: this footing level must be satisfied for footings within 10 m of the tree, as measured from the centre of the tree trunk, and verified by means of the Grading Plan as indicated in the Procedural Changes below.

3. A **small** size tree must be provided with a minimum of **25 m^3** of available soil volume, as determined by a Landscape Architect. A **medium** size tree must be provided with a minimum of **30 m^3** of available soil volume, as determined by a Landscape Architect. The developer will ensure the soil is generally uncompacted when backfilling in street tree planting locations.

   Note: the soil volume calculation must be based on a depth of 1.5 m below finished grade (e.g., 5 m length x 4 m width at surface x 1.5 m depth = 30 m^3). It may include lands in the right-of-way and on private property but must subtract the volume of shallow utility trenches (i.e., volume of shallow utility trenches cannot count toward minimum soil volume).

4. The tree species must be small to medium size, as confirmed by a Landscape Architect in the Landscape Plan.

5. The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15 m) to provide ductility as described in the Geotechnical Report.

6. Grading surrounding the tree must promote draining to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

14 Slope Stability

14.1 Slope Stability Analysis

The stability of the existing slopes was analyzed by using Morgenstern-Price Method, GeoStudio /Geo-slope office, Version 8.13 computerized system. The purpose of the analysis was to assess the stability of the existing slopes and to determine the required set back of the proposed structures from the crest of the slopes. A total of seven cross-sections were analyzed. These cross-sections have been shown as Sections A-A, B-B, C-C, D-D, E-E, F-F, and G-G on Figure 2.

These cross-sections were obtained from the 2015 Lidar Survey of the site.

The natural slope inclinations at the cross-sections analyzed were determined, and the results have been presented on Table VIII.

Table VIII: Natural Slope Inclination of Cross-sections Analyzed

Section	Crest of Slope (m)	Toe of Slope (m)	Height of Slope (m)	Overall Slope Inclination
A-A	62.0	49.0	13.0	2.7H:1V
B-B	60.0	49.0	11	3.2H:1V
C-C	62.0	50.5	11.5	3.1H:1V
D-D	60.0	49.0	11.0	2.9H:1V
E-E	61.0	49.0	12.0	3.3H:1V
F-F	59.0	49.0	10.0	2.9H:1V
G-G	58.3	50.0	8.25	7.6H:1V

The slopes were analyzed for the following conditions:

1.) Effective stress analysis.

2.) Total stress analysis; and

3.) Total stress analysis with seismic loading;

The following assumptions were made:

1.) The crest of the existing slopes varies from Elevation 58.25 m to 62.0 m whereas the toe of the slopes is at Elevation 49.0 m to 50.5 m (Table VIII).

2.) The soil stratigraphy for the various cross-sections is shown on Figure Nos. 15 to 36 inclusive. The soil stratigraphy was established from the boreholes drilled at the site.

3.) The unit weight of the various soils was established from laboratory tests. The undrained shear strength of the clay was established by performing in-situ field vane tests. The shear strength parameters were selected based on literature search. Previous work undertaken on the Ottawa Area Clays by Mitchell (1970), Sangrey & Paul (1971), Eden, Fletcher and Mitchell (1971), Eden &

Jarret 1971, Mitchell & Eden 1972 was reviewed. In addition, a certain number of back analysis of the natural slopes in Champlain clay have been presented in the literature: Crawford & Eden (1967), Eden & Mitchell (1970), Lefebvre & La Rochelle (1974), Lo & Lee (1974), Silvestri (1980), Lefebvre, G. (1981), Tavenas & Leroueil (1981) and Lefebvre, G., Pare, J.J. and Dascal, O. (1987) were reviewed. The review indicated that values of the effective cohesion (c') and effective angle of internal friction (φ') of the clay obtained by the various researchers varied from 5.3 kPa to 11.5 kPa and 31 degrees to 35 degrees respectively. The effective cohesion (c') and effective angle of internal friction (φ') values applicable in this case depend on the stress conditions in the slopes. Based on the review of the literature and site conditions, and using somewhat conservative approach, an effective cohesion of 9.8 kPa and effective angle of internal friction of 36 degrees was used in the analysis.

The average undrained shear strength used in the analysis was computed from the field-vane test results, which varied from 180 kPa to 46 kPa. The undrained shear strength was reduced by 15 percent in accordance with Section 6.6.3.2 (7) of the Canadian Foundation Engineering Manual.

4.) The slopes were assumed to be fully submerged i.e. the groundwater table in the slope coincides with the existing ground surface.

5.) Building loads were not taken into consideration in the analyses since the structures would be located away from the slopes.

The computed factors of safety of all the cross-sections analyzed for effective stress analysis were less than 1.5 except for sections E-E and G-G, which were stable. Therefore, the required slope inclination was computed for each section analysed to achieve a minimum factor of safety of 1.5 using reiterative slope stability analysis as follows:

Section	**Required Slope Inclination for FS = 1.5**
A-A	3.7H:1V
B-B	3.3H:1V
C-C	3.4H:1V
D-D	3.3H:1V
G-G	3.4H:1V
Average	**3.42H:1V**

The reiterative slope stability analysis, which yielded factor of safety of less than 1.5, have not been included in the report.

The results of the analyses are given on Figures 16 to 36 inclusive and have also been tabulated on Table IX.

Table IX: Results of Slope Stability Analysis

Section	Condition Analysed	Factor of Safety	Geotechnical Set Back from Toe of Slope	Figure No.
A-A	Effective stress analysis and set back determination	1.50	48.0	16
	Total stress analysis	1.75	-	17
	Total stress analysis with seismic loading	1.12	-	18
B-B	Effective stress analysis and set back determination	1.51	37.0	19
	Total stress analysis	2.13	-	20
	Total stress analysis with seismic loading	2.12	-	21
C-C	Effective stress analysis and set back determination	1.52	39.0	22
	Total stress analysis	1.97	-	23
	Total stress analysis with seismic loading	1.17	-	24
D-D	Effective stress analysis and setback determination	1.50	36.0	25
	Total stress analysis	2.14	-	26
	Total stress analysis with seismic loading	1.34	-	27
E-E	Effective stress analysis and set back determination	1.67	40.0	28
	Total stress analysis	2.06		29
	Total stress analysis with seismic loading	1.26		30
F-F	Effective stress analysis and set back determination	1.50	34.0	31
	Total stress analysis	2.23		32
	Total stress analysis with seismic loading	1.38		33
G-G	Effective stress analysis and setback determination	2.90	63.0	34
	Total stress analysis	3.82		35
	Total stress analysis with seismic loading	1.70		36

Current practice of the City of Ottawa requires a minimum acceptable factor of safety of 1.5 for static loading conditions. The minimum acceptable factor of safety for seismic loading conditions is 1.1 (Mitchell 1983).

It is noted that the ravines located adjacent to the north property boundary and in the southeast part of the site are covered with vegetation and trees. The vegetation and trees provide stability to the slopes and should not be disturbed in anyway. It is understood that the owner is willing to give an undertaking to preserve the vegetation on these slopes and to enhance the vegetative cover using local species.

During construction, the following precautions should be taken so that the stability of the slopes is not adversely affected:

1.) Care should be exercised during construction to ensure that the existing slopes are not steepened by placement of fill close to the crest of the slope since this would reduce the stability of the slope.
2.) Excavations should not be undertaken at the toe of the slopes since this would adversely affect the stability of the slopes.
3.) Natural drainage paths should not be blocked by placement of fill on the slope. If fill must be placed on the slope, adequate drainage should be provided to prevent buildup of pore pressures in the soil.
4.) Vegetation should not be removed from the faces of the slopes to prevent erosion. Additional vegetation should be planted on the slopes wherever necessary.

14.2 Behaviour of Slopes During Earthquakes

Lafebvre, G. (1981)[5] has stated that if the clay is not liquefiable, liquefaction during dynamic loading of earthquake will not be a concern. It has been previously demonstrated that the clay at the site is not susceptible to liquefaction. Therefore, it is concluded that the stability of the slopes at the site will not be adversely affected during a seismic event.

14.3 Flow Slides

Mitchell and Markall (1974)[6] have developed a method based on undrained shear strength to estimate the likelihood of flow slides. The undrained shear strength of the clay was measured using field shear vane. Based on analyzing the data for more than 40 sites, they established that flow slides will only occur in soils with total overburden pressure more than six times the undrained shear strength of the soil, i.e.:

$$\frac{\gamma H}{Su} > 6$$

Where γ is the bulk density of soil,

H is the height of the slope, and

Su is the undrained shear strength of the soil.

The undrained shear strength used in the above expression was 85 percent of the measured shear strength to mitigate future cyclic loading. The ratio of total overburden pressure to reduced undrained shear strength of the on-site clay was computed as 4.64. Therefore, it is concluded that the clay at the site is not prone to flow slides.

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[5] Lefebvre, G. (1981), "*Fourth Canadian Geotechnical Colloquium: Strength and Slope Stability in Canadian Soft Clay Deposits*", Can. Geot. J., Vol 18, pgs. 8420-422.6

[6] Mitchell, R..J. and Markell, A.R. 1974, *"Flow Slides in Sensitive Soils", Can Geot. J11, pgs. 423-454.*

15 Limit of Hazardous Lands

It is noted that to establish the limit of hazardous lands, in addition to the geotechnical set back, two other factors must be taken into consideration. These are toe erosion allowance and erosion access allowance. The magnitude of the toe erosion allowance depends on the soil types, the state of erosion along the creek/riverbank and upon the width of the channel. The Ministry of Natural Resources procedures permit either the installation of erosion protection or alternatively to consider a toe erosion allowance.

The north and south slopes of the creek located on the site were examined by geotechnical engineers from EXP to determine if creek banks are eroding. The examination of the slopes did reveal some evidence of erosion of both the creek banks as shown on photos in Appendix B. Boreholes drilled have indicated that the natural soil in the vicinity of the creek bottom is stiff clay. Based on this information, it is considered that a toe erosion allowance of 8 m should be provided.

In addition to the toe erosion allowance, an erosion access allowance of 6 m is normally required.

Based on the cross-sections analyzed, the limit of hazardous land was computed by the addition of required geotechnical set back, the erosion allowance and erosion access allowance. The exception to this is between Section F-F and the concrete culvert located close to Section G-G (see Figure 2) and between two culverts along Section G-G where the erosion protection will be provided and the limit of hazardous lands was obtained by the addition of required geotechnical set back and access allowance.

The required limit of hazardous lands has been tabulated on Table X at cross-section locations and has been plotted on Site Plan, Figure 2. The crest of the slope was assumed at the location where the ground surface flattens to inclination of 10H:1V. The limit of hazardous lands should be staked out in the field by a registered Ontario Land Surveyor as shown in Figure 2. No development should take place within the hazardous lands limits.

Table X: Limit of Hazardous Lands at Cross-Section Locations

Section	Geotechnical Setback from the Toe (m)	Erosion Set Back (m)	Available Erosion Allowance in Valley Floor (m)	Erosion Access Allowance (m)	Total Setback from creek or toe of the slope (Limit of Hazardous Lands) (m)
A-A	48	8	0	6	62 from creek
B-B	37	8	8	6	51 from creek
C-C	39	8	17	6	62 from creek
D-D	37	8	0	6	51 from creek
E-E	40	8	0	6	54 from creek
F-F	34	8	12	6	52 from creek
G-G	67	0	0	6	73 from creek

16 Pathway

Pathways have been constructed part way up the slopes by excavating into the slopes. It is understood that these pathways were constructed approximately 10 years ago and that very little to no maintenance was required during this period. A visual examination of these slopes was undertaken. It revealed that construction of the pathway has resulted in localized steepening of the slope uphill of the pathway. This has led to cave-in and/or erosion of the soil at some locations resulting in undermining of the toe of the upper slope. This was observed at Locations 1 to 6, 8, 10, 12, 13, 15, 16, 18, 19, 21, 24 and 25 as shown on Figure 37 and in photos of the same number except at Location 16 where a photo was not taken.

Some erosion and/or sloughing of the slope downhill of the pathway was also observed at Locations 7, 9, 11, 14, 17, 20, 22 and 23 as shown in photos of the corresponding number.

It is considered that cave-in and undermining of the toe of the upper slope is likely due to steepening of the slope and removal of vegetation from the face of the slope. The cave-in and undermining of the upper slope is expected to continue if not remediated and may result in eventual failure of the slope. Potential remedies may consist of armour stone retaining walls, gabion baskets, coir logs, etc.

For repair purposes, the upper slopes at the location of Photos 1, 6, 8, 10, 12, 13, 15, 16, 18, 19, 21 and 24 have been divided into three categories as follows:

b.) Significant slopes – These are slopes with a height of greater than 1 m. These slopes should be stabilized by constructing retaining walls as per City of Ottawa requirements as soon as possible.

c.) Somewhat significant – These are slopes with a height of more than 0.5 m to 1.0 m. These slopes should be monitored and repaired as necessary.

d.) Less significant slopes – These are slopes with a height of 0.5 m or less. These slopes will likely remain stable and are expected to require little to no repairs or maintenance.

It is recommended that the above slopes should be stabilized by construction of armour stone retaining walls, gabion baskets, coir logs, etc.

It is understood that at some locations, the slopes were cut into to borrow material for construction of the pathways. The owner has indicated that these areas will be rehabilitated by construction of retaining walls, etc.

The owner has already constructed retaining walls at several locations. These include the pond area (Photo 26), across from the pond and the pathway (Photo 27), the Buddhist section (Photos 28) and the Christian Meditation area located in the northeast portion of the site where the slope has been terraced with armour stone retaining walls (Photo 29). Similarly, a retaining wall has been constructed at the entrance to the sanctuary (Photo 30). The construction of these retaining walls would increase the stability of the slopes and prevent slope failures in the areas.

The potential erosion areas located downhill of the pathway (areas represented by Photos 7, 9, 11, 14, 17, 20, 22 and 23) should be stabilized by removing all the loose soil that has sloughed from the face of the slope and dressing the slope with rip rap, bushes with deep rooted growth, live stakes, etc.

It is noted that the creek banks are also eroding in some areas. Most significant areas are located along the meanders in the creek. It is considered that these areas may be repaired by construction of armour stone retaining walls, placement of rip rap, etc. A permit would be required from the Rideau Valley Conservation Authority (RVCA) prior to undertaking any work.

Detailed design of the retaining walls, rip rap, etc., is beyond the terms of reference in this investigation. It is recommended that a detailed plan for the remedial walls should be prepared, approval obtained from the RVCA and implemented.

17 General Comments

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for the design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

The information contained in this report is not intended to reflect on environmental aspects of the soils. Should specific information be required, including for example, the presence of pollutants, contaminants or other hazards in the soil, additional testing may be required.

We trust that the information contained in this report will be satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Figures

Explanation of Terms Used on Borehole Logs

Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Societv for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by **exp** Services Inc. also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

0.002 | 0.006 | 0.02 | 0.06 | 0.2 | 0.6 | 2.0 | 6.0 | 20 | 60 | 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Appendix A: One-dimensional Oedometer Test Results from Report OTT-00019355-A0

Appendix B – Pathways & Erosion Photos

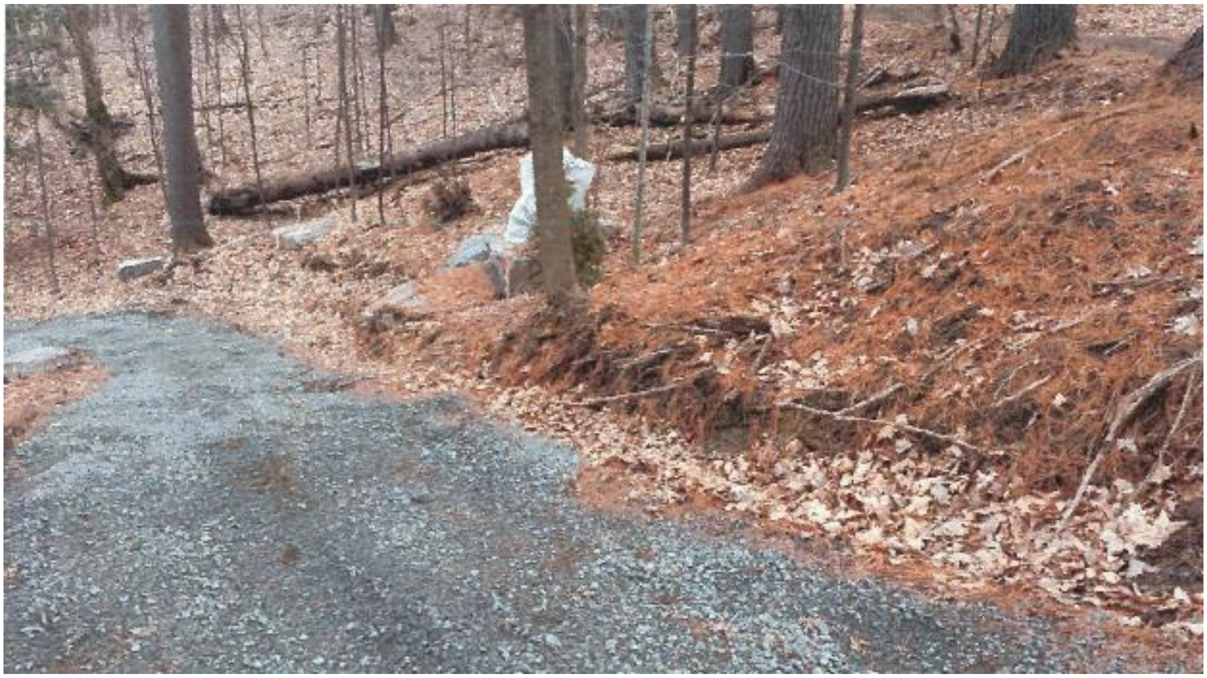

Photo No. 1 – Erosion of cut uphill of the pathway.
Cut approximately 18 m long x 0.3 m high

Photo No. 2 – Erosion of cut uphill of pathway. Cut approximately 9 m long x 0.4 m high.

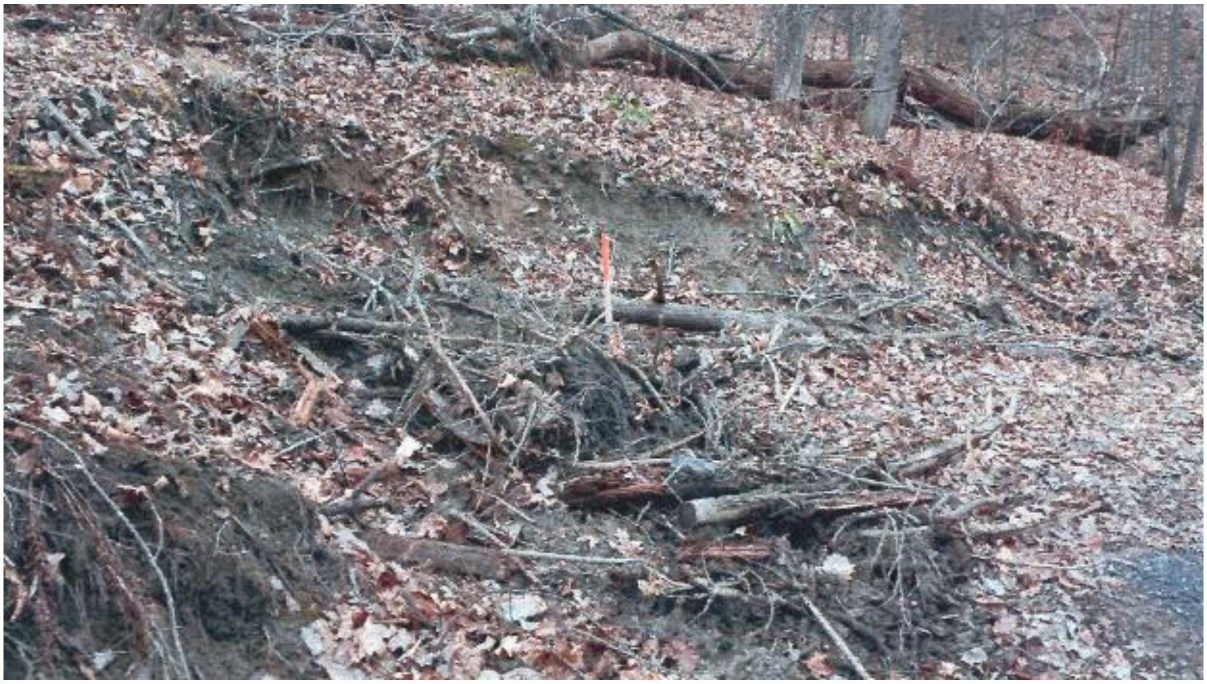

Photo No. 3 – Erosion above pathway approximately 5 m long x 0.4 to 1.0 m high

Photo No. 4 – Erosion above pathway. Approximately 3 m deep and 0.6 m to 1.3 m high.

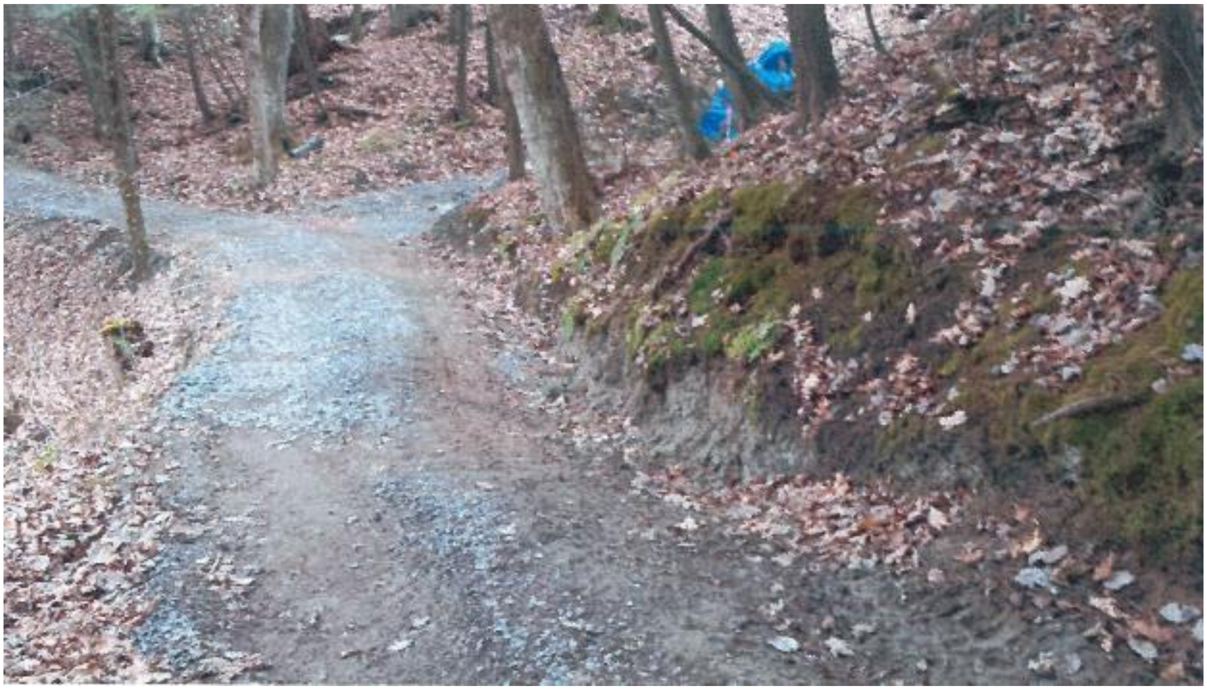

Photo No. 5 – Erosion of cut uphill of pathway. Cut approximately 12 m long x 0.9 m high.

Photo No. 6 – Erosion of cut uphill of pathway.
Cut approximately 16 m long x 0.5 m to 1 m high.

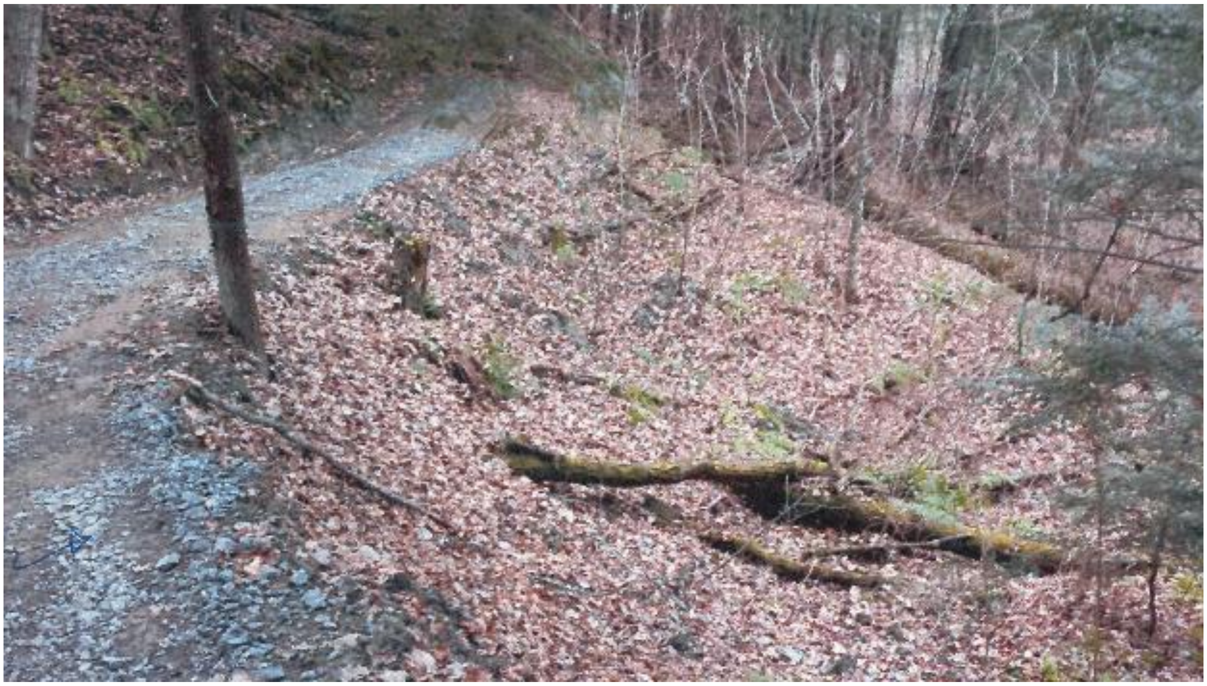

Photo No. 7 – Erosion of slope downhill of pathway.
Area approximately 30 m long x 1 m to 3 m high.

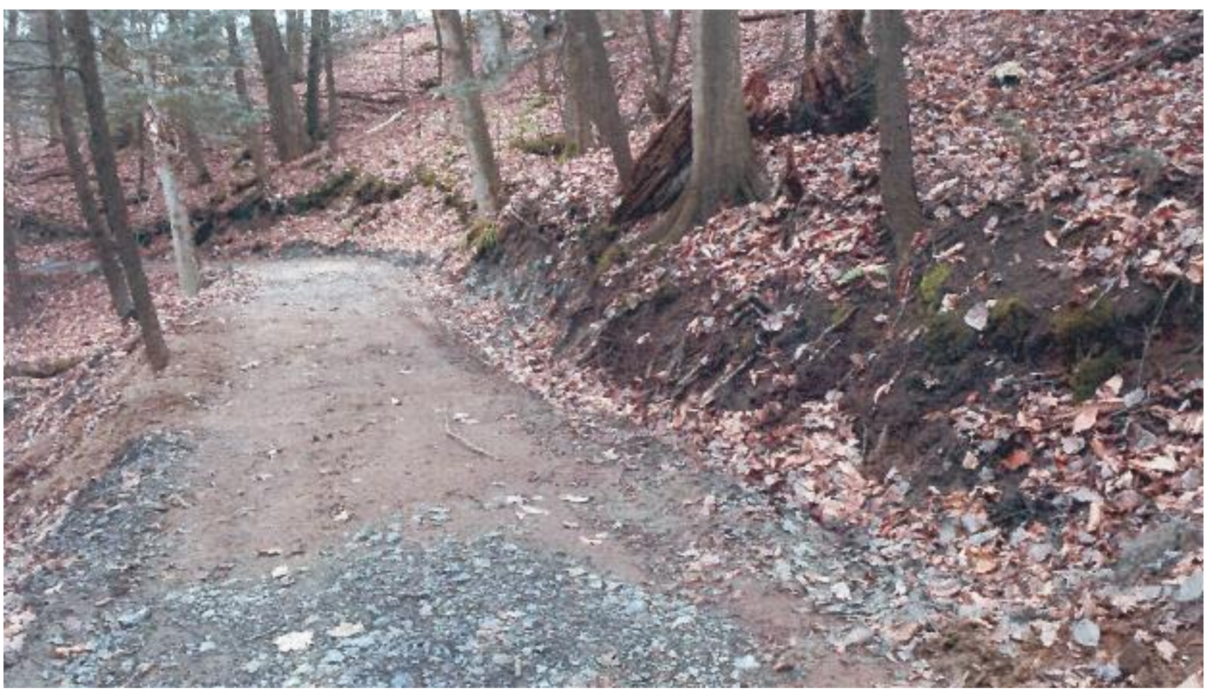

Photo No. 8 – Erosion of cut uphill of pathway.
Cut approximately 42 m long x 0.5 m to 1.3 m high.

Photo No. 9 – Erosion of slope downhill of pathway.
Area approximately 30 m long by 2 m to 3 m high.

Photo No. 10 – Erosion of cut uphill of pathway.
Cut approximately 20 m long x 0.4 m to 1.2 m high.

Photo No. 11 – Erosion of slope downhill of pathway.
Approximately 18 m long x 1 m to 2 m high.

Photo No. 12 – Erosion of slope uphill of pathway.

Photo No. 13 – Erosion of slope uphill of pathway.
Area approximately 5 m long and 1 m to 1.5 m high.

Photo No. 14 – Erosion of slope downhill of pathway.
Note placement of logs to prevent erosion.

**Photo No. 15 – Erosion of cut uphill of pathway.
Area approximately 7 m long and 0.8 m to 1.6 m high.**

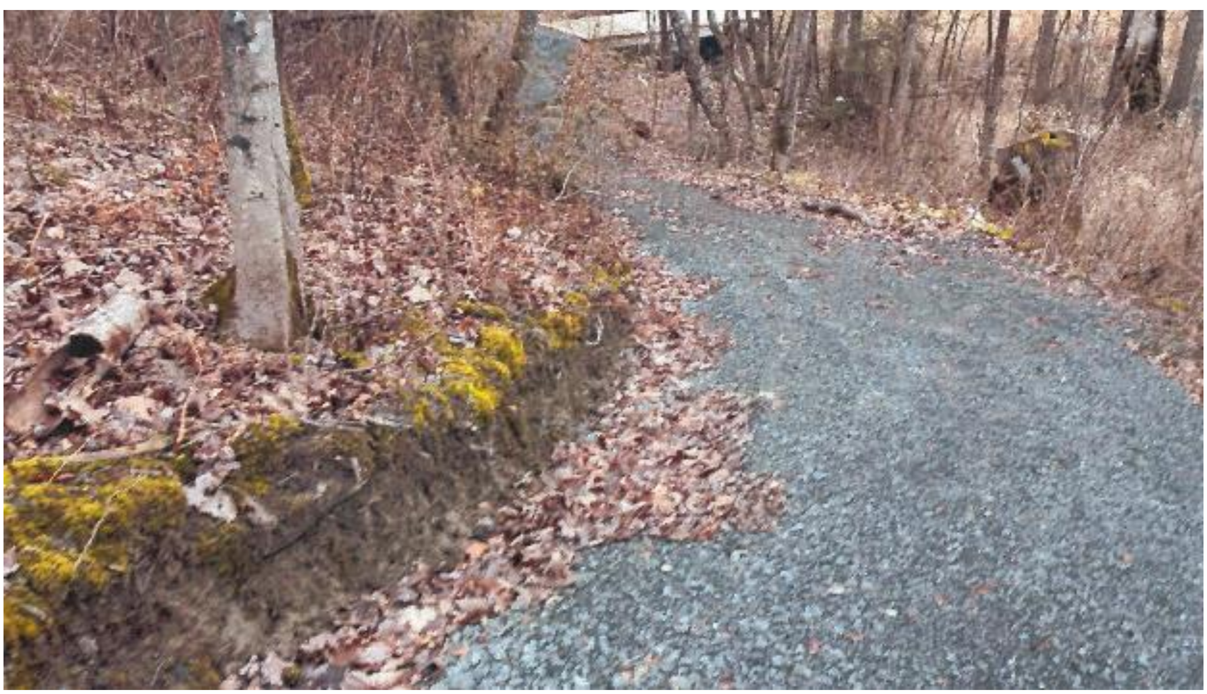

**Photo No. 17 – Erosion of cut downhill of pathway.
Cut approximately 30 m long x 0.3 m high.**

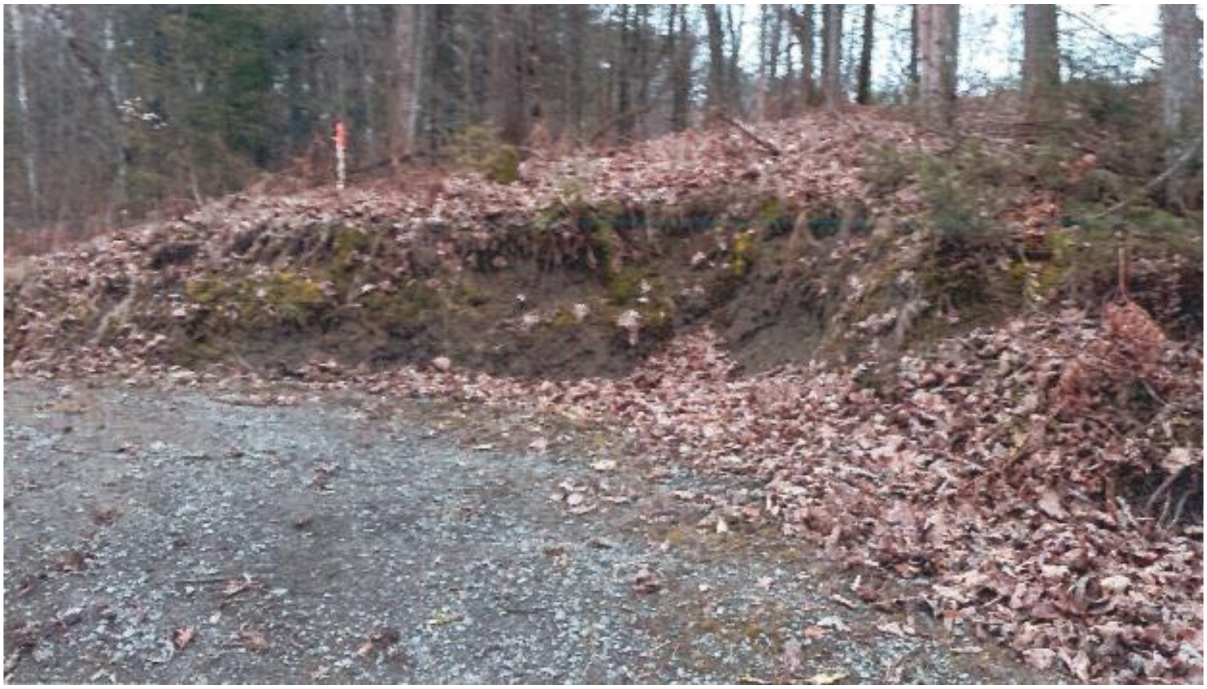

Photo No. 18 – Erosion of cut uphill of pathway. Cut approximately 3 m long x 1.2 m high.

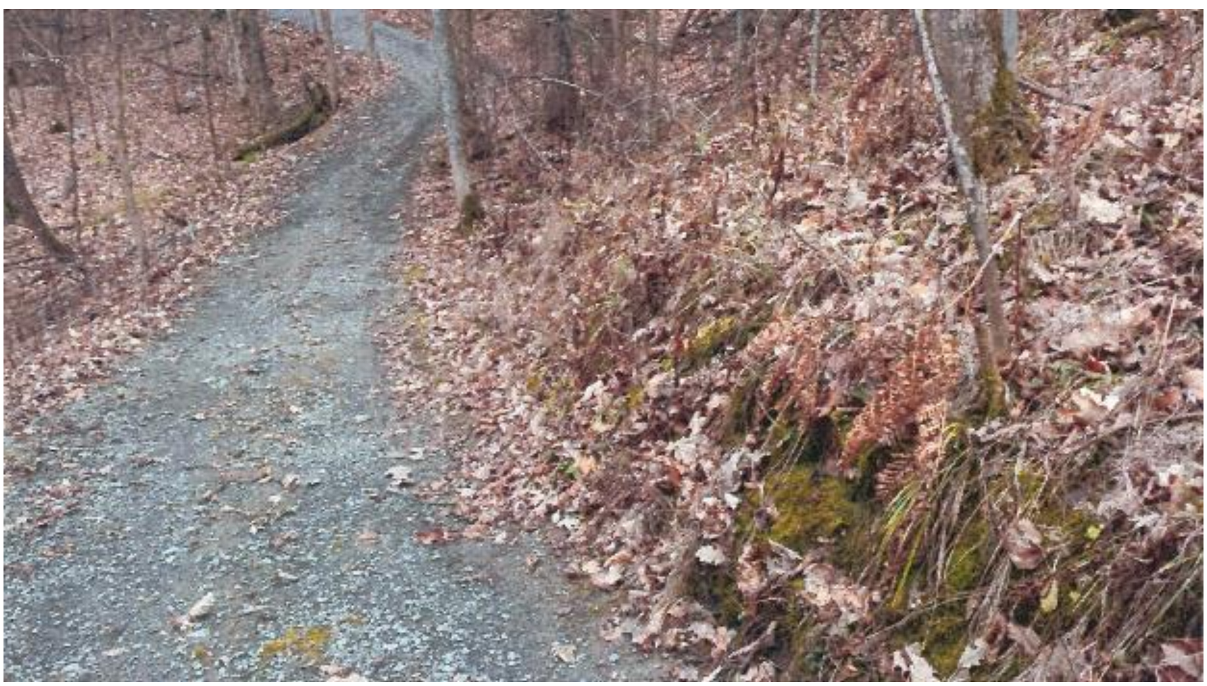

Photo No. 19 – Erosion of cut above pathway. Cut approximately 7 m long x 0.8 m high.

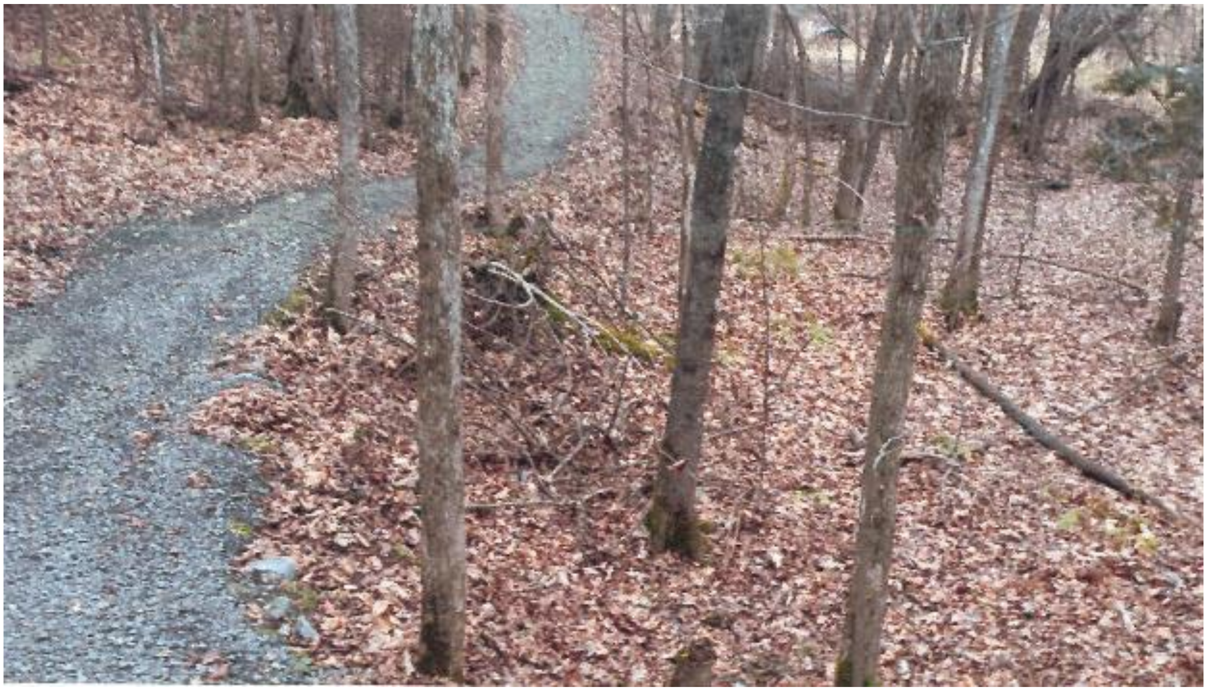

Photo No. 20 – Deep cut downhill of pathway.
Cut approximately 12 m long x 0.5 m to 1 m high.

Photo No. 21 – Erosion of cut above pathway.
Cut approximately 45 m long x 0.3 m to 0.8 m high.

Photo No. 22 – Erosion below pathway. Cut approximately 30 m long x 0.5 to 2 m high.

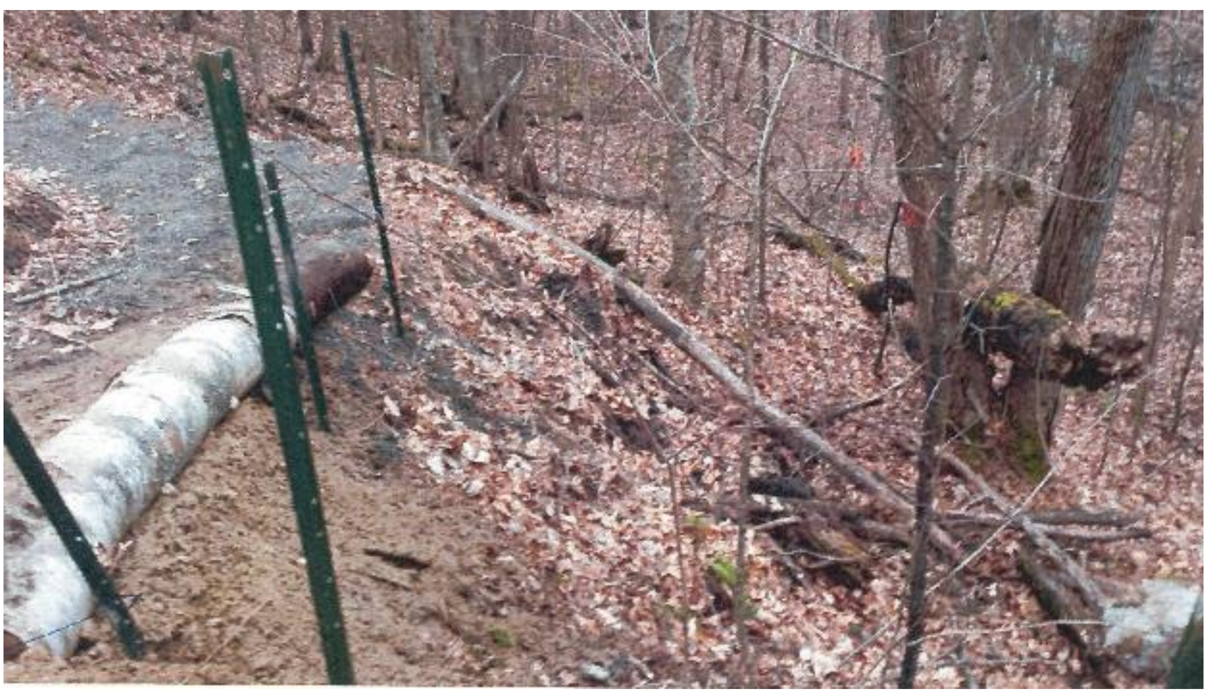

**Photo No. 23 – Erosion of slope downhill of pathway.
Area approximately 12 m long x 1.5 m to 2 m high.**

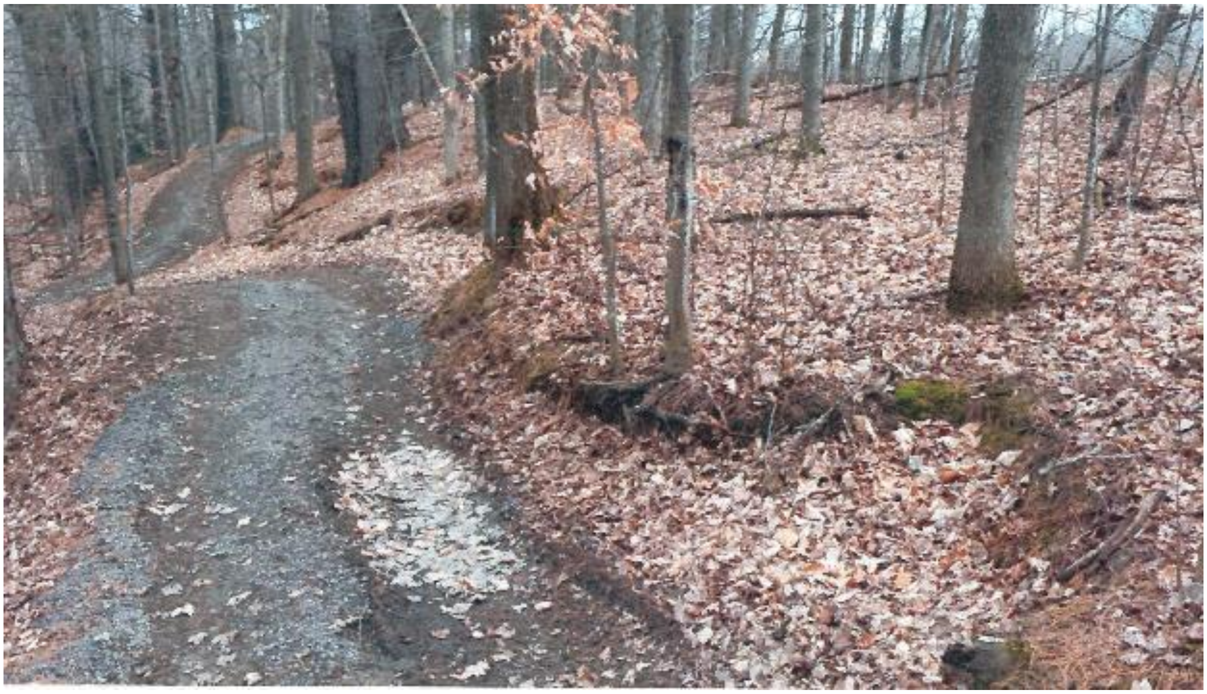

Photo No. 24 – Erosion of cut above pathway. Cut approximately 20 m long x 0.4 m high.

Photo No. 25 – Erosion of cut above pathway. Cut approximately 4 m long x 0.6 m high.

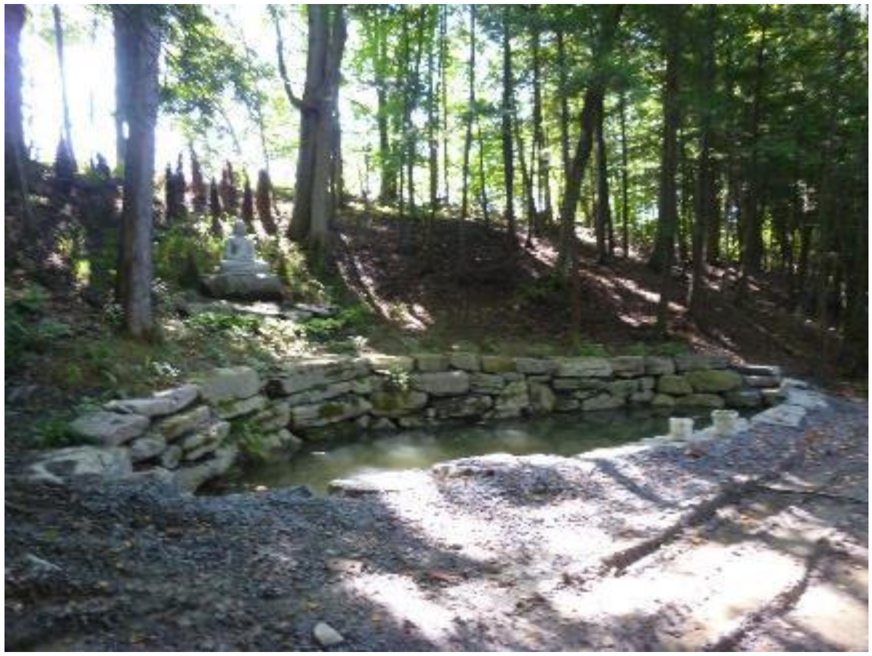

Photo No. 26 –Retaining wall at pond location.

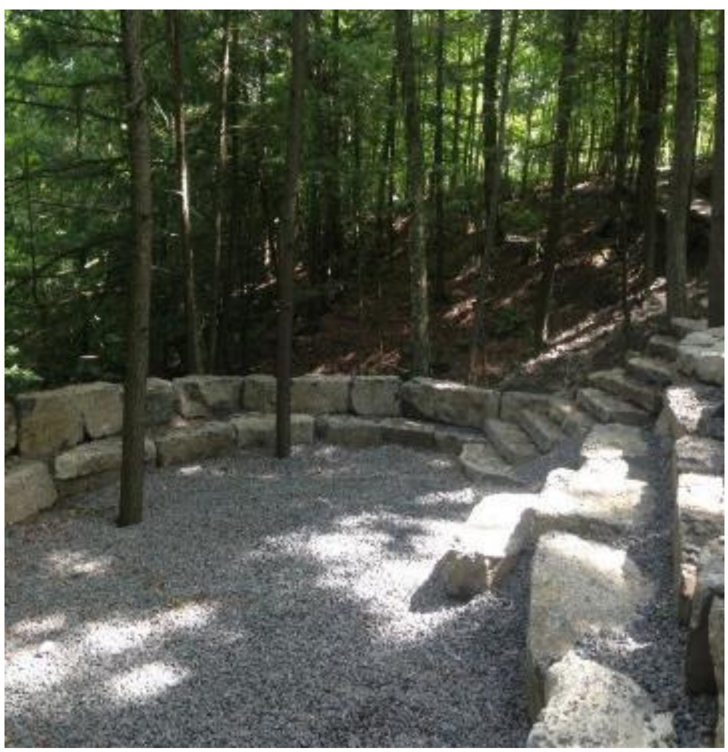

Photo No. 27 –Retaining walls across from the pond and the pathway.

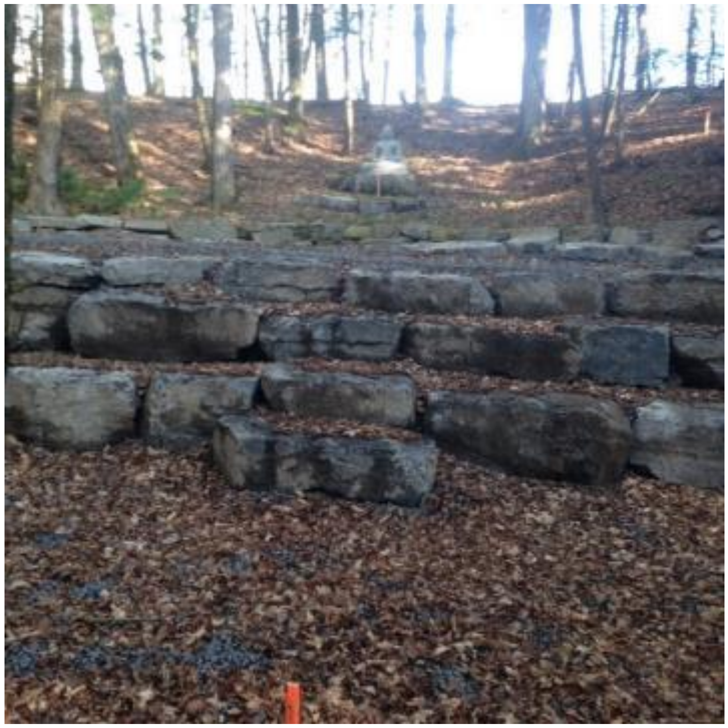

Photo No. 28 – Terraced retaining wall at the Buddhist Meditation section.

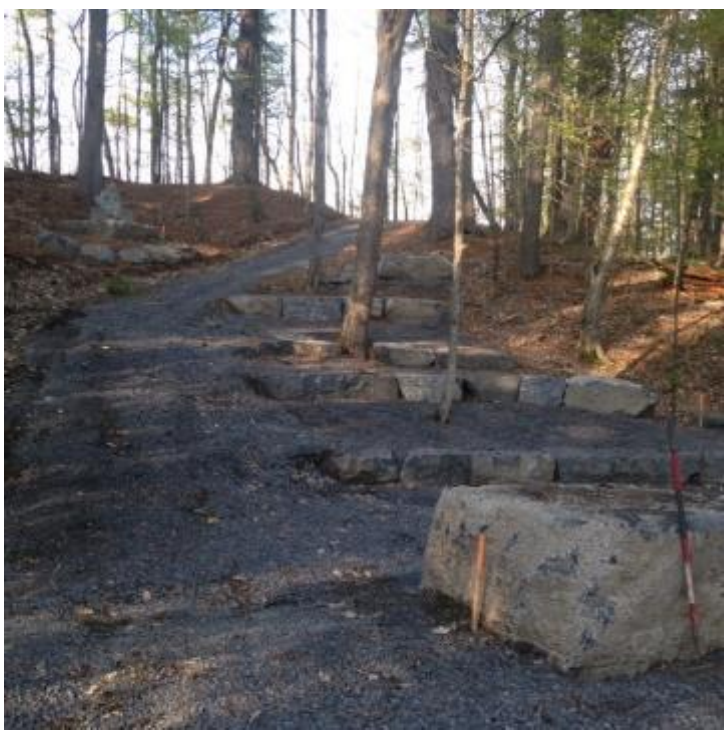

Photo No. 29 – Terraced retaining wall at the Christian Meditation area.

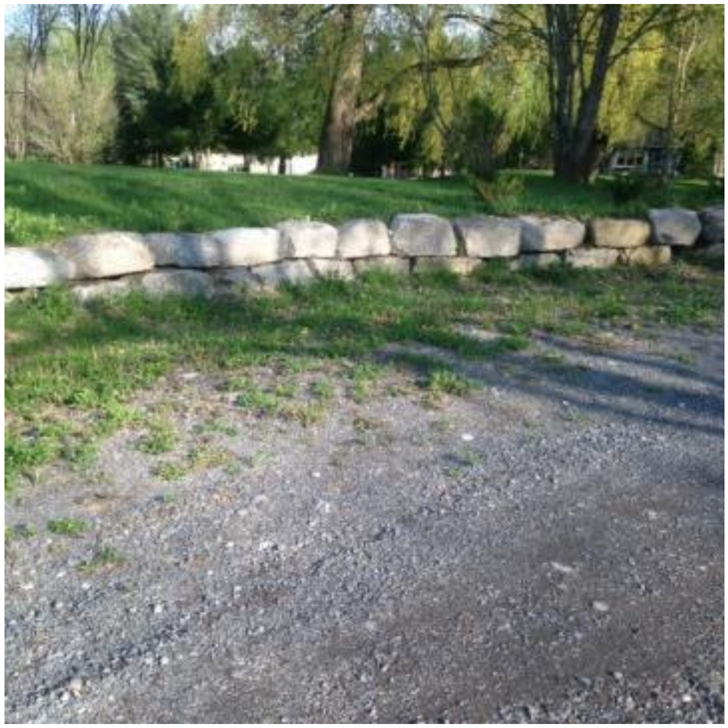

Photo No. 30 – Retaining wall at the entrance to the sanctuary.

Appendix C: Bernie Muncatser Letter , April 14, 2022

List of Distribution

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