AllRock
Consulting Ltd

Geotechnical Investigation Report

2184 & 2194 Oneida Crescent, Mississauga Ontario

Prepared For:

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Geotechnical Investigation Report

Proposed Residential Detached Dwellings

Project No.: 25431

April 6, 2026

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1. INTRODUCTION

This report presents the results of a geotechnical investigation and slope stability analysis carried out for the proposed residential detached dwellings located at 2184 & 2194 Oneida Crescent in Ontario.

The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of boreholes and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

2. BACKGROUND

2.1 Project Description

The proposed development is to include nine (9) residential lots ranging from 648m² to 3,428m² [1]. The land is currently developed with two (2) existing dwellings located at 2194 and 2184 Oneida Crescent respectively, with associated parking and landscaped areas. The project area is also bordered by the Credit River Valley and The Mississauga Golf & Country Club to the south and west. AllRock has been retained to conduct a geotechnical investigation and provide recommendations, including preliminary grading, foundation design, and slope stability analysis.

2.2 Documents Reviewed

AllRock notes no previous investigations have been provided for review. The following documentation was reviewed as apart of preparation for the geotechnical investigation and developing the recommendations herein:

[1] Development Concept Plan - Prepared by GSA Glen Scharr & Associated Inc - Dated 2025-05-28
[2] Submission Requirement Checklist- Prepared by The City of Mississauga - Dated 2025-01-22
[3] Topographic Survey - Prepared by Tarasick McMillan Kubicki Limited - Dated 2024-12-19

3. SUBSURFACE INVESTIGATION

3.1 Geotechnical Investigation

The field work for this investigation was carried out on the 4th to 5th of November 2025. At that time, five (5) boreholes, numbered BH25-1 to BH25-5, were advanced to depths ranging from 2.29 to 3.63 meters below existing grade.

The borehole locations were selected and positioned on-site by AllRock in consultation with the Client’s design team; locations were determined based on the proposed concept plan [1]. The field work was observed throughout by a member of our engineering staff who directed the drilling operations and logged the samples.

Following completion of the boreholes, the soil samples were returned to our laboratory for examination by a geotechnical / materials engineer. Selected samples were submitted for moisture content and grain size distribution testing.

The approximate locations of the boreholes are shown on the Borehole Location Plan **Error! Reference source not found.** results of the boreholes are provided on the Record of Boreholes Sheets in Appendix A.

3.2 Methodology

Materials and soil description have been made with reference to the following documents:

- Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) – ASTM D2487-06
- Standard Practice for the Description and Identification of Soils (Visual-Manual Procedure) – ASTM D2488-06

It should be noted that the split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be represented in the grain size distributions contained herein. Any cobbles or boulders that may be present within the subsurface would be noted based on frequency of encounters as observed through visual observation of shaking / grinding of augers or within the soil cuttings.

3.3 Site Survey

Survey data for the subject development site was collected by an AllRock onsite technician using a Trimble Catalyst DA2 Receiver + Trimble TDC600. The results are summarized in table 3-1 below:

Table 3-1 Survey Results

Borehole No.	UTM	Easting	Northing	Elevation (mASL)
BH25-1	17N	611090.8	4823369.7	103.6
BH25-2	17N	611131.5	4823347.2	102.4
BH25-3	17N	611135.1	4823434.6	103.9
BH25-4	17N	6111433.4	4823387.6	103.1
BH24-5	17N	611170.3	4823265.0	78.7

4. SUBSURFACE CONDITIONS

4.1 General

The soil and groundwater conditions identified in the boreholes are given on the Record of Borehole sheets in Appendix A. The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of exploration, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the borehole locations may vary from the conditions encountered in the boreholes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and AllRock does not guarantee descriptions as exact but infers accuracy to the extent that is common in current geotechnical practice.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. It is noted that groundwater conditions can vary seasonally or as a result of construction activities in the area.

4.2 Regional Geology

Based on publicly available Ontario Geological Survey (OGS) mapping, the site is mapped within Glaciolacustrine deposits consisting of gravelly sand and gravel dated to the Pleistocene era. However, it is noted that quaternary and physiographic mapping is approximate and that as such, conditions at the Site may differ.

The publicly available bedrock mapping of the area shows the site is mapped within the Georgian Bay Group consisting of limestone and shale.

4.3 Site Lithology

The following section presents an overview of the subsurface conditions encountered in the borehole investigation. In general, soils at the site consisted of natural fill, overlaying native silty sand and glacial till.

4.3.1 Topsoil

A layer of topsoil was encountered at every borehole location. The topsoil was described as light brown with organic material and was found to have a thickness ranging between approximately 100 to 200 millimeters. An assessment of the organic material content or other topsoil quality indicator tests was beyond the scope of this investigation.

4.3.2 Fill

A layer of Fill was encountered at all borehole locations and extended to a depth of ranging from 0.8 meters to 2.3 meters below ground surface. It was described as a brown, moist sand, with some silt and gravel. In general, the fill was found to extend to 0.8 mbgs along the crest / tableland of the slope, with the exception of BH25-2, where the fill extended to 2.3 mbgs and was likely the result of regrading at the site during the construction of the existing dwellings.

4.3.3 Silty Sand

A silty sand layer was encountered below the fill at boreholes BH25-1, BH25-3, BH25-4 and extended to a depth of ranging from 1.5 meters to 2.3 meters below ground surface. It was described as a brownish grey, dry, compact, with some gravel and clay.

Standard penetration tests carried out in the native silty sand gave N values ranging from 8 to 39 blows per 0.3 metres of penetration, which reflects a loose to dense relative density. The SPT test results show an increase in blows with depth reflecting an increase in relative density.

4.3.4 Glacial Till

A glacial till was encountered below the silty sand at all borehole locations and extended to a depth of ranging from 2.3 meters to the termination depth of 3.7 meters below ground surface. It is noted that the borehole that reached termination depth was BH25-5. This borehole was advanced at the bottom of the slope Southeast of the project area and is not included within the excavation area.

Standard penetration tests carried out in the glacial till gave N values ranging from 8 to 50 blows per 0.3 metres of penetration, which reflects a loose to very dense relative consistency. Similar to the overlaying silty sand, SPT values advanced in the glacial till increased with depth, indicating an increase in relative density

4.3.5 Soil Laboratory Sampling

Table 4.1 – Summary of Grain Size Distribution Testing

<table>
<tr><th>Location</th><th>Sample Number</th><th>Sample Depth (m)</th><th>Test Type</th><th>Gravel (%)</th><th>Sand (%)</th><th>Silt (%)</th><th>Clay (%)</th><th>Moisture Content (%)</th></tr>
<tr><td>BH25-1</td><td>SS4</td><td>2.3 – 3.1</td><td>Hydro</td><td>16</td><td>8</td><td>60</td><td>16</td><td>15.1</td></tr>
<tr><td>BH25-3</td><td>SS4</td><td>2.3 – 3.1</td><td>Hydro</td><td>12</td><td>3</td><td>70</td><td>15</td><td>15.3</td></tr>
<tr><td>BH25-2</td><td>SS3</td><td>1.5 – 2.3</td><td>Grain Size</td><td>21.3</td><td>69.6</td><td colspan="2">9.1</td><td>14.3</td></tr>
<tr><td>BH25-4</td><td>SS2</td><td>0.76 – 1.5</td><td>Grain Size</td><td>32.5</td><td>56.5</td><td colspan="2">11.0</td><td>17.3</td></tr>
<tr><td>BH25-2</td><td>SS1</td><td>0.0 – 0.76</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>14.8</td></tr>
<tr><td>BH25-2</td><td>SS2</td><td>0.76 – 1.5</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>10.0</td></tr>
<tr><td>BH25-2</td><td>SS4</td><td>2.3 – 3.1</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>15.5</td></tr>
<tr><td>BH25-4</td><td>SS1A</td><td>0.0 – 0.76</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>24.3</td></tr>
<tr><td>BH25-4</td><td>SS1B</td><td>0.0 – 0.76</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>19.0</td></tr>
<tr><td>BH25-4</td><td>SS3</td><td>1.5 – 2.3</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>11.9</td></tr>
<tr><td>BH25-5</td><td>SS1</td><td>0.0 – 0.76</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>13.1</td></tr>
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<tr><td>BH25-5</td><td>SS3</td><td>1.5 – 2.3</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>22.3</td></tr>
<tr><td>BH25-5</td><td>SS4</td><td>2.3 – 3.1</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>21.3</td></tr>
<tr><td>BH25-5</td><td>SS5</td><td>3.1 – 3.9</td><td>Moisture</td><td>/</td><td>/</td><td colspan="2">/</td><td>7.2</td></tr>
</table>

Moisture content analysis conducted on selected samples yielded a moisture content of 7.2 to 24.3 percent moisture content. Lab testing results of moisture content analysis are provided in Appendix B.

4.3.6 Inferred Bedrock / Refusal

Auger refusal on presumed bedrock was encountered at boreholes excluded BH25-05. Based off the bedrock geology mapping of the area and SPT samples, it is believed the bedrock is shale. Below is a table summarizing the borehole refusal elevations.

Table 4-1 Refusal Elevations

MW Name	Refusal Depth (mbgl)	Refusal Elevation (mbgl)
BH25-1	2.57	101.0
BH25-2	2.59	99.8
BH25-3	3.07	100.8
BH25-4	2.29	100.8
BH25-5	Not Encountered	NA

4.3.7 Groundwater Levels

Water levels were taken the installed well in BH25-2 by a AllRock Technician on November 14, 2025. The well was dry at the time measurement was taken.

It should be noted that groundwater levels may fluctuate over time or in response to heavy precipitation events. Observations made during the field investigation are based on a limited amount of data and over a limited amount of time and should not be construed as a stabilized groundwater level for the area.

5. SLOPE STABILITY ASSESSMENT

5.1 General

The slope stability analysis was carried out using Slope/W, a two-dimensional slope stability program, based on the provided a cross-section of the existing slope from details provided by the client. In addition, soil conditions used in the analysis were based on samples collected in the field by AllRock.

5.2 Input Parameters

The soil conditions used in the stability analysis were based on the borehole advanced at the site. The slope generally consists of a minor layer of fill material overlying a generally compact gravelly sand, followed by a glacial till. Underlying the glacial till, the crest of the slope was modelled to incorporate the inferred bedrock that was encountered throughout. However, as the bedrock was not encountered at the toe of the slope, it was not modelling under the till neat the toe and instead continues a downward dip beyond the extents of the model envelope. The slope was modeled as such, using typical soil strength parameters for the site area. A summary of the assumed soil parameters at the site are shown in Table 5.1.

Table 5.1 – Summary of Assumed Soil Parameters

Soil Type	Effective Angle of Internal Friction, φ (degrees)	Effective Cohesion, c'(kPa)	Unit Weight, γ (kN/m^3)
Gravelly Sand	36	0	21
Glacial Till	35	0	20.5
Inferred Bedrock	Impenetrable		

Two cross-sections were taken through the perceived two worst case slope sections based on geometry, history, and water levels and used in AllRock's analyses. The results of the stability analysis are highly dependent on the assumed groundwater conditions. Based on the groundwater readings taken during the investigation, as well as the follow up readings taken on November 14th, 2025, it was determined the typical long term phreatic surface likely to be present at the site traverses the bedrock surface and does not result in a shallow groundwater table throughout the slope. Therefore, the phreatic surface for the model was set to approximately 2.0 mbgs along the tableland / crest, and at 3.5 mbgs at the slope toe to match the borehole termination depths advanced along the toe.

In accordance with the 2010 NBCC and the encountered soil conditions, the soil embankments at the Site can be classified as Site Class D – Stiff Soil. Per Table 4.1.8.4B from the 2010 NBCC, the horizontal acceleration multiplication factor for Site Class D soils with spectral acceleration of 0.2 seconds < 0.25g is 1.3. Using this factor, the PGA value for design can be assumed as 0.038g.

5.3 Factor of Safety

The slope stability analysis was carried out using conditions and slope profiles that attempt to model the slope in question. It should be noted that the selected parameters and site conditions may not represent actual conditions across the entire slope, and the assumed conditions are therefore limited to the locations where field data could be obtained.

For the purposes of this study, a computed factor of safety of less than 1.0 to 1.3 is considered to represent a global stability bordering on failure to marginally stable, respectively; a factor of safety of 1.3 to 1.5 is considered to indicate global stability that is less likely to fail in the long term and provides a degree of confidence against failure ranging from marginal (1.3) to adequate (1.4 and greater) should conditions vary from the assumed conditions. A factor of safety of 1.5, or greater, is considered to indicate adequate long-term stability. For pseudo-static conditions, a factor of safety of 1.1 or greater is considered adequate for long-term stability under cyclic loading conditions.

The slope stability analysis completed on both Section A-A' and Section B-B' indicates that the existing slope crest has a factor of safety of 1.3 against overall failure under normal operating conditions and is considered to be stable. The slope stability analysis completed on both Section A-A' and Section B-B' indicates that the existing slope crest has a factor of safety of 1.1 against overall failure under pseudo-static operating conditions and is considered to be stable.

The results of the slope stability carried out for section A-A using Slope/W can be found in Appendix B.

5.4 Conclusions

It should be noted that when considering the setbacks from the slope crest as limits to the developable footprint, the Credit Valley Conservation Authority considers a sum of three distances from the toe of the slope as the total set back. The setback first is the Long-Term Stable Slope Line (LTSSL) which was determined to be the top of slope as shown in [3]. The second is the Toe Erosion Allowance, and the third is the Erosion Access Allowance. As the toe of slope is more than 15 meters from a water course, there should not be a requirement for a Toe Erosion Allowance. The Erosion Access Allowance is intended to be utilized for emergency access to erosion prone areas as well as to provide access to construction equipment in the event of a slope failure. Given that there exists access to the bottom of the slope through the golf course should the slope be in need of repairs it may be unnecessary to apply the Erosion Access Allowance, however, this would be at the discretion of the Credit Valley Conservation Authority.

6. RECOMMENDATIONS AND GUIDELINES

6.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety, and equipment capabilities. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions.

The National Building Code of Canada 2020 Guidelines (hereafter NBCC 2020), and the 5th edition of the Canadian Foundation Engineering Manual, 2023 (hereafter CFEM 2023) were considered for these recommendations. Based on the collected information from the test pits excavated as part of this investigation, the geotechnical recommendations are presented in the following sections.

The recommendations and guidelines provided in this report pertain to the proposed site development. It is required to that geotechnical personnel (AllRock) confirm the soil conditions and recommendations at the time of construction.

6.2 Proposed Site Development

It is understood that the proposed residential development foundations will consist of either conventional strip and isolated spread footings for bearing. The following recommendations are based on this understanding. Should the proposed foundations design differ from this understanding, AllRock should be notified to update our recommendations.

6.2.1 Excavation

The excavation for the proposed development will be carried out through existing topsoil, fill, silty sand, and glacial till. The sides of the excavation should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the act, soils at this site can be classified as Type 3 above the water table and Type 4 below the water table. That is, open cut excavations above the groundwater table within overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where excavation side slopes cannot be accommodated due to space constraints, a shoring system may be required. Additional guidelines for the design and selection of a suitable shoring system could be provided as the design progresses.

In the event that a granular pad is necessary below the foundations, the excavations should be sized to accommodate a pad of imported granular material which extends at least 0.6 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

6.2.2 Groundwater and Pumping Management

Groundwater inflow, if any, from the overburden deposits should be controlled by pumping from filtered sumps within the excavation based on the groundwater readings taken by others. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services due to the isolated area of the proposed development. It is anticipated that groundwater inflow from the overburden deposits into the excavations could be handled from within the excavations.

It is noted that groundwater levels and surface water flows can increase during wet periods of the year such as the early spring or following periods of precipitation. As a single groundwater reading was taken, it is recommended that further monitoring events occur prior to finalizing any dewatering requirements at the site.

The groundwater handling should be carried out in accordance with provincial and local regulations. Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

6.2.3 Subgrade Preparation and Placement of Engineered Fill

Any existing topsoil or fill material should be removed from below the proposed structures and to a minimum of 3 meters beyond the perimeter of the footprint.

Imported granular material (engineered fill) should be used to raise the grade in areas where the proposed founding level is above the level of the native soil, or where sub-excavation of material is required below proposed founding level. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II and should be compacted in maximum 200-millimetre-thick lifts to at least 99 percent of the standard Proctor maximum dry density. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.6 metres beyond the footings and then down and out from the edges of the footings at 1 horizontal to 1 vertical, or flatter. The excavations should be sized to accommodate this fill placement.

It is noted that engineered fill in excess of 1 meter thick can be expected to experience post-construction settlement in the order of 0.5 to 1 precent of the height of the soil placed (depending on the composition of the engineered fill). It is anticipated that if engineered soil is sourced from the native onsite soils, it may take 2 to 4 months for the majority of post-construction settlement to occur; however, if imported granular fill as such as that meeting the (OPSS) requirements for Granular B Type II, settlement will likely occur within 1 to 2 weeks of placement. Furthermore, any proposed grade raises that would exceed more than 1 meter higher than the original elevation of the area should be evaluated for consolidation settlement of the soft fine-grained material encountered throughout the site.

6.2.4 Footing Design

6.2.4.1 Shallow Foundations

In the event that shallow foundations consisting of either conventional, continuous strip and isolated spread footings or grade beams are utilized for bearing at the site, the following recommendations should be consulted during the design of the footings. In general, the native silty sand and glacial till are considered suitable to support the proposed structures founded on shallow foundations. The existing topsoil and fill material are not considered suitable for the support of the proposed development and should be removed from the proposed development areas. Furthermore, any loose or soft soils encountered at footing subgrade elevations should be over excavated to competent material and backfilled with engineered fill as outlined in Section

6.2.3. Where extremely weathered bedrock is encountered at footing subgrade elevations, it should be over excavated to competent material. Where footings extend across different subgrades (soil / bedrock) a minimum of 300 mm of engineered fill placed as outlined in Section 6.2.3 to minimize differential settlement across the soil / bedrock interface.

For preliminary design purposes, footings prepared as outlined above and founded on the native sandy silt or on a pad of compacted engineered fill above native sandy silt or competent shale should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 125 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 150 kilopascals.

Footings prepared as outlined above and founded on the native till material should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 175 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 225 kilopascals.

The post construction total and differential settlement of footings should be less than 25 and 15 millimetres respectively, provided that all loose or disturbed soil is removed from the bearing surface and provided that any engineered fill material is compacted to the required density. Personnel from AllRock should be consulted to inspect the footing subgrades prior to placement of any foundations.

6.2.5 Frost Protection of Foundations

All exterior footings for heated buildings that consist of slab on grade construction or included basement should be provided with at least 1.2 metres of earth cover for frost protection purposes. Isolated, unheated and/or exterior pier footings adjacent to surfaces which are cleaned of snow cover during the winter months should be provided with a minimum of 1.6 metres of earth cover. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Further details regarding the insulation of foundations could be provided at the detailed design stage, if necessary.

6.2.6 Concrete Slab Support

Based on the results of the investigation, the area in the vicinity of the proposed structures is generally underlain by topsoil, fill, and native overburden deposits. The topsoil and fill should be removed from the slab on grade areas. The grade below the concrete slabs on grade could be raised, where necessary, with granular material meeting OPSS Specification book requirements for Granular B. The use of Granular B material is preferred under wet conditions. The granular base for the proposed slab on grade should consist of at least 150 millimeters of Granular A.

All imported granular materials placed below the proposed floor slab should be compacted in maximum 200-millimetre thick lifts to at least 99 percent of the standard Proctor maximum dry density value.

Proper moisture protection with a vapour retarder should be used for any slab on grade where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The "Guide for Concrete Floor and Slab Construction", ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the floor slab.

Underfloor drainage is not considered necessary provided that the floor slab level is 300 mm above the finished exterior ground surface level and the underside of slab is a minimum of 500 mm above the seasonal high groundwater level.

Thermal protection of the concrete slab on grade is required in areas that will remain unheated during the winter period. The type of insulation used below the slabs will depend on the stresses imposed on the insulation. The stress on the insulation should not exceed about 35 percent of the insulation's quoted compressive strength due to the time dependent creep characteristics of this material. Further comments could be provided as the design progresses.

6.3 Site Services

6.3.1 Excavation

Based on the investigation, the excavations for the services within the site will be carried out through topsoil, silty sand, and glacial till.

The sides of the excavations within overburden soils should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the soils at this site can be classified as Type 3 soils. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation slopes within the native soils at this site. As an alternative to sloping the excavations, all services installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose. It should be noted that soft clays were encountered throughout the site, as such, certain shoring systems may not be practical given the site conditions. Furthermore, over-excavation of weathered bedrock and backfill with engineered fill may be required to ensure a suitable bearing material for utilities at the site, AllRock personnel should be consulted to inspect the exposed subgrade prior to placement of utilities.

Excavations that extend a maximum 1.2 meters into bedrock may consist of vertical walls, however, if deeper excavations are required, the bedrock walls will need to be benched or sloped as necessary by inspection through AllRock personnel.

The groundwater inflow should be controlled throughout the excavation and pipe laying operations by pumping from sumps within the excavation.

6.3.2 Groundwater Pumping

Possible groundwater inflow from the overburden deposits into the excavations could be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant effect on nearby structures and services. The groundwater handling should be carried out in accordance with provincial and local regulations. To reduce the groundwater pumping requirements, we suggest that the excavation be planned for the dry period of the year (i.e., June to September). Furthermore, additional groundwater readings are recommended prior to finalizing any dewatering requirements for the site.

Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review. To minimize any required dewatering efforts, trench cut-offs within the till material should be considered.

6.3.3 Pipe Bedding and Cover

The bedding for the sanitary sewers, storm sewers and watermains should be in accordance with OPSD 802.010 and 802.031 for flexible and rigid pipes, respectively. The pipe bedding should consist of at least 150 millimetres of well graded crushed stone meeting OPSS requirements for Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A and Granular B Type II material.

Since the source of recycled material cannot be determined, it is suggested that any granular materials used in the service trenches be composed of virgin (i.e., not recycled) material only.
Allowance should be made for sub excavation of any existing fill, organic deposits, or disturbed material encountered at subgrade level.

Allowance should be made to place a sub bedding layer composed of 150 to 300 millimetres of OPSS Granular B Type II in areas where wet silty sand is encountered at the pipe subgrade level to reduce the potential for disturbance.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A.
The use of clear crushed stone should not be permitted for the installation of site services, since it could exacerbate groundwater lowering of the overburden materials due to “French Drain” effects.

The sun bedding, bedding and cover materials should be compacted in maximum 200-millimetre-thick lifts to at least 98 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

6.3.4 Seismic Site Classification

According to Table 4.1.8.4.A of the NBCC 2020, Site Class D should be used for the seismic design of the structures bearing on native soils or on engineered fill material over native soils.

For an improved site classification, performing and Multi-Channel Analysis of Surface Waves should be considered. The soils at this site are not considered to be liquefiable or collapsible under seismic loads.

6.3.5 Lateral Earth Pressures

The static "At Rest" thrust (Po) acting on any below grade walls, temporary shoring, or retaining walls should be calculated using the following formula:

$P_o = 0.5\ K_o\ \gamma\ H^2$

where;

- P_o: Static at rest thrust component (kN/m);
- γ: Moist material unit weight (kN/ m^3);
- K_o: "At Rest" earth pressure coefficient;
- H: Wall height (m).

Seismic shaking can increase the forces on the retaining walls. The total "At Rest" thrust acting on the wall (P_{oe}) during a seismic event should be calculated using the following formula:

$P_{oe} = 0.5\ K_{oe}\ \gamma\ H^2$

where;

- P_{oe}: Total "At rest" thrust (kN/m);
- γ: Moist material unit weight (kN/m^3);
- Ko: "At Rest" earth pressure coefficient;
- Koe: Dynamic at rest earth pressure coefficient;
- H: Wall height (m).

The static thrust component (P_o) acts at a point located H/3 above the base of the walls. During seismic shaking, the total "At Rest" thrust (P_{oe}) acts at a point located about H/2 above the base of the wall. It should be noted that the total "At Rest" thrust, P_{oe}, is composed of a static component and a dynamic component.

For design purposes, the parameters provided in Table 5.1 can be used to calculate the thrust acting on the wall during static and seismic loading conditions.

Table 6-1 Summary of Design Parameters (Building Foundation)

Parameter	OPSS Granular B Type II
Material Unit Weight, γ (kN/m^3)	22
Estimated Friction Angle (degrees)	38
"At Rest" Earth Pressure Coefficient K_o, assuming horizontal backfill behind the structure	0.38
Dynamic "At Rest" Earth Pressure Coefficient K_{oe}, assuming horizontal backfill behind the structure	0.52

According to the 2024 Ontario Building Code, the peak ground acceleration (PGA) for the site is 0.35 for stiff soil ground conditions, Site Class D. The dynamic at rest earth pressure coefficient was calculated using the method suggested by Mononobe and Okabe, assuming a horizontal coefficient k_h of 0.37 (taken as the PGA) and assuming that the vertical seismic coefficient k_v is zero.

7. ADDITIONAL CONSIDERATIONS

7.1 Effects of Construction Induced Vibration

Some of the construction operations (such as excavation, granular material compaction, etc.) will cause ground vibration on and off on the site. The vibrations will attenuate with distance from the source but may be felt at nearby structures. Assuming that any excavating is carried out in accordance with the guidelines in this report, the magnitude of the vibrations will be much less than that required to cause damage to the nearby structures or services in good condition but may be felt at the nearby structures.

7.2 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

7.3 Design Review and Construction Observation

It is recommended that the final design drawings be reviewed by the AllRock personnel to ensure that the guidelines provided in this report have been interpreted as intended.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

The subgrade surfaces for the proposed structures should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

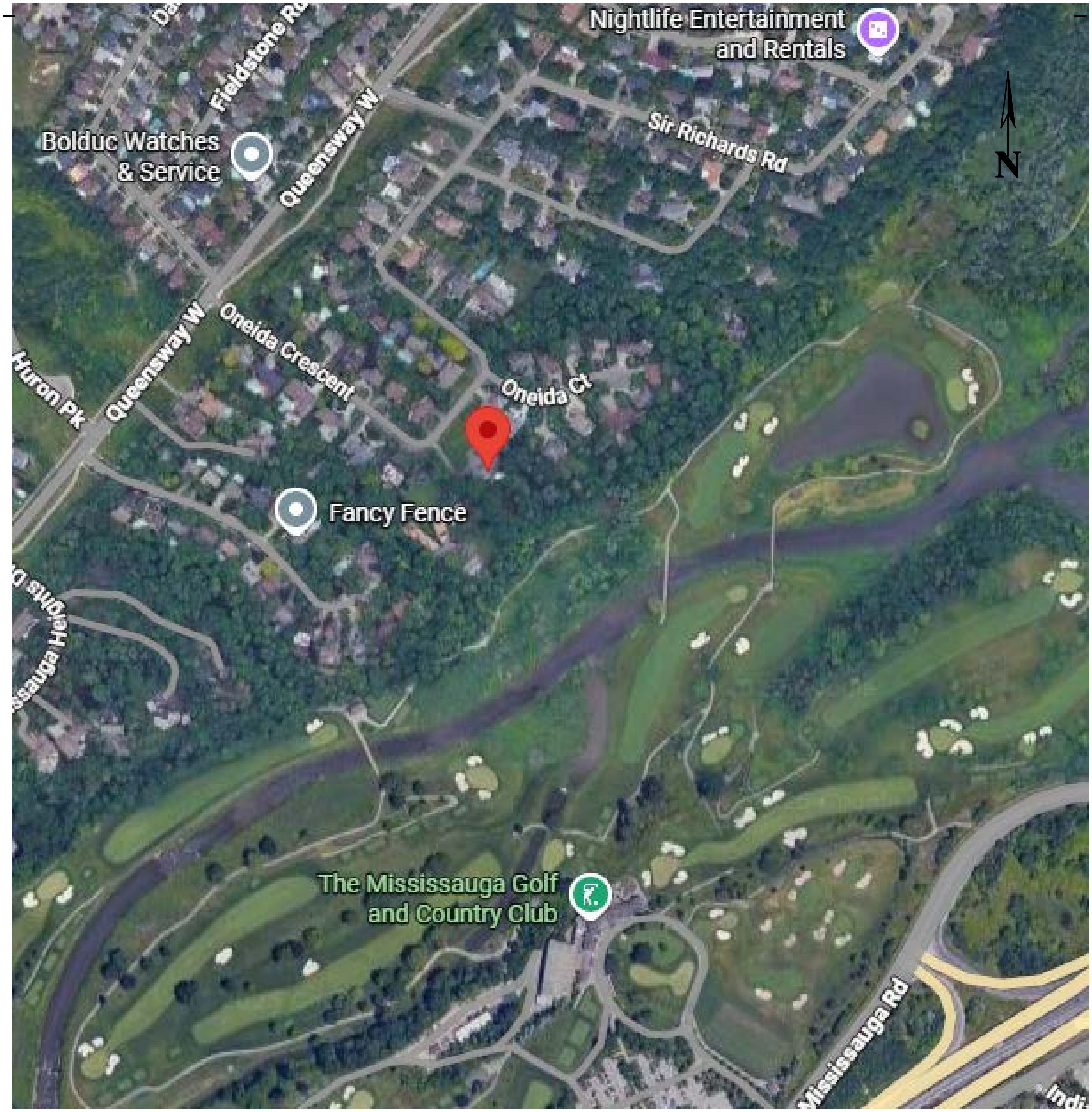
8. CLOSURE

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Jeremy Milsom, P.Geo
Geoscientist
jeremy.milsom@allrockconsulting.com

Mac Garrison, M.Eng., P.Eng.
Geotechnical Lead
mac.garrison@allrockconsulting.com

LICENSED PROFESSIONAL ENGINEER
M. D. GARRISON
100223043
April 6, 2026
PROVINCE OF ONTARIO



© OpenStreetMap contributors

FIGURE TITLE: KEY MAP	AllRock Consulting Ltd
PROJECT: GEOTECHNICAL INVESTIGATION	
CLIENT: CROZIER CONSULTING ENGINEERS	
ADDRESS: 2194 ONEIDA CRESCENT, MISSISSAUGA	DATE: DECEMBER 2025 / FIGURE NO.: 1
PROJECT NO: 25431 / APPROXIMATE SCALE: NTS	DRAWN BY: JM / CHECKED BY: MG

AllRock
Consulting Ltd

24 Brydon Dr #5, Etobicoke, ON M9W 5R6

25431 - Oneida Crescent BH Location Plan

Job No: 25431	
Client: Crozier & Associates Consulting Engineers	
Project: 2184 & 2194 Oneida Crescent	
Address: 2184 Oneida Crescent, Mississauga, ON L5C 1V6, Canada	
Legend: Borehole Locations	
Image Source: Google Maps	Viewed: 2025-11-17

Drawn By:	Checked By:	Date:	Figure:
ND	MG	2025-11-17	1

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Appendix A

Record of Borehole Sheets

AllRock Consulting Ltd

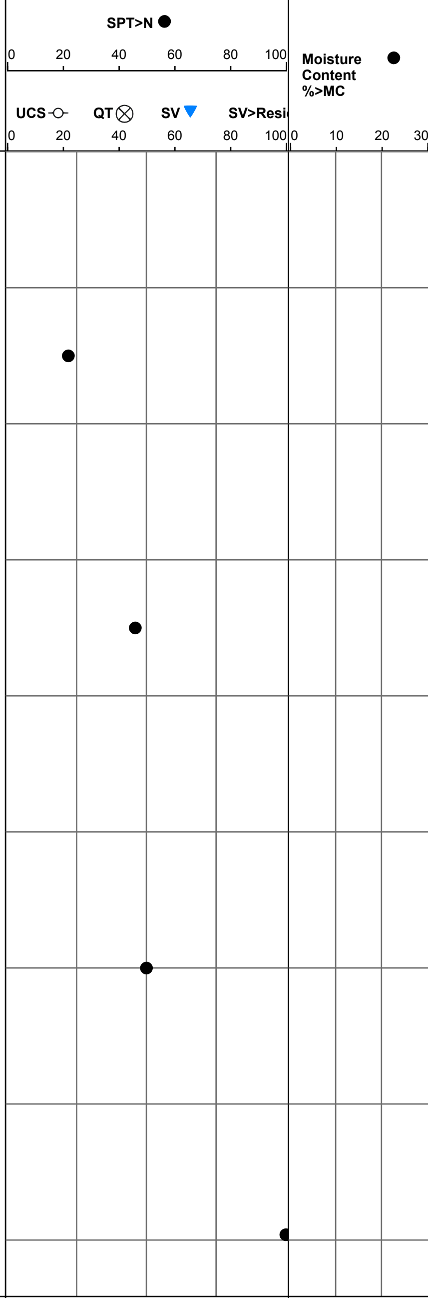
Borehole Record: BH25-1

Project: 2184 & 2194 Oneida Crescent
Location: 2184 Oneida Crescent, Mississauga, ON L5C 1V6, Canada
Loc Comment:
Job No: 25431

Position: Easting: 611090.79
Northing: 4823369.66
Elevation: 339.77(Ft)
Inclination:

Date Drilled: 05/11/2025
Drill Rig Truckmount Drill Rig
Drill Supplier Terra Firma
Logged/Checked: ND/MG

Depth (m)	ELEV DEPTH (m)	SOIL PROFILE: DESCRIPTION	NUMBER	TYPE	Blow Counts	Recovery (%)
		TOPSOIL	SS1A			100
0.204	103.4 0.2	**FILL** gravelly SILT and SAND brown to grey, moist, compact, trace organics (rootlets), trace clay	SS1B	SPT	2-12-10-18 N = 22	100
0.762	102.9 0.8	**SILTY SAND** grey, moist, very dense, some to trace gravel, trace clay, occasional cobbles	SS2	SPT	10-18-28-30 N = 46	100
1.0						
			SS3	SPT	14-25-25-39 N = N > 50	100
2.0						
2.287	101.3 2.3	**TILL** SILTY SAND, grey, dry, very dense, some gravel, trace to some clay	SS4	SPT	30-50/5" N = N > 50	100
		BH25-1 Refusal on inferred bedrock at 2.57 m				



Well Diagram	Remarks: Logged Remarks
	Borehole terminated in Till. Wet soils were not encountered. Standing water was not observed. Caving was not encountered.

Borehole Record: BH25-2

AllRock Consulting Ltd

Project: 2184 & 2194 Oneida Crescent
Location: 2184 Oneida Crescent, Mississauga, ON L5C 1V6, Canada
Loc Comment:
Job No: 25431

Position: Easting: 611131.47
Northing: 4823347.17
Elevation: 335.87(Ft)
Inclination:

Date Drilled: 05/11/2025
Drill Rig Truckmount Drill Rig
Drill Supplier Terra Firma
Logged/Checked: ND/MG

Depth (m)	ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	Blow Counts	Recovery (%)	SPT>N	Moisture Content %>MC	Remarks
		TOPSOIL	SS1	SPT	3-6-9-11 N = 15	25	15		
0.204	102.2 / 0.2	**FILL** SAND, brown, moist, compact, some silt, some gravel, occasional cobbles	SS2	SPT	5-6-7-6 N = 13	100	13		
			SS3	SPT	5-5-8-9 N = 13	75	13		- some silt to silty, trace clay
2.287	100.2 / 2.3	**TILL** SILTY SAND, brown/grey, moist, very dense, some gravel, trace clay	SS4	SPT	8-50/2" N = N > 50	100	>50		Water level recorded as dry on November 14, 2025
		BH25-2 Refusal on inferred bedrock at 2.59 m							Borehole terminated in Till. Wet soils were not encountered. Standing water was not observed. Caving was not encountered.

Depth scale: 1.0, 2.0

SPT>N ● scale: 0, 20, 40, 60, 80, 100
UCS -○- QT ⊗ SV ▼ SV>Resi: 0, 20, 40, 60, 80, 100
Moisture Content %>MC ●: 0, 10, 20, 30

Borehole Record: BH25-3

AllRock Consulting Ltd

Project: 2184 & 2194 Oneida Crescent	**Position:** Easting: 611135.11	**Date Drilled:** 04/11/2025
Location: 2184 Oneida Crescent, Mississauga, ON L5C 1V6, Canada	Northing: 4823434.56	**Drill Rig** Truckmount Drill Rig
Loc Comment:	**Elevation:** 340.38(Ft)	**Drill Supplier** Terra Firma
Job No: 25431	**Inclination:**	**Logged/Checked:** ND/MG

Depth (m)	ELEV DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	Blow Counts	Recovery (%)	Remarks
		TOPSOIL	SS1A			75	
0.152	103.7 / 0.2	**FILL** sandy SILT light brown, w~PL, firm, trace organics (rootlets), trace clay	SS1B	SPT	2-2-3-4 N = 5	75	
0.762	103.1 / 0.8	**SILTY SAND** light brown, moist, compact, trace gravel, trace clay	SS2	SPT	8-12-15-20 N = 27	100	
			SS3	SPT	8-10-10-28 N = 20	100	
2.287	101.5 / 2.3	**TILL** sandy SILT, brown/grey, dry (w<PL), hard, trace clay, trace gravel	SS4	SPT	20-34-50/3" N = N > 50	20	
			SS5	SPT	50/1" N = 50	100	
		BH25-3 Refusal on inferred bedrock at 3.07 m					Borehole terminated in Till. Wet soils were not encountered. Standing water was not observed. Caving was not encountered.

Depth scale: 1.0, 2.0, 3.0

Well Diagram

SPT>N ● 0 20 40 60 80 100

UCS -○- QT ⊗ SV ▼ SV>Resi

Moisture Content %>MC ● 0 10 20 30

Logged Remarks

Borehole Record: BH25-4

AllRock Consulting Ltd

Project: 2184 & 2194 Oneida Crescent
Location: 2184 Oneida Crescent, Mississauga, ON L5C 1V6, Canada
Loc Comment:
Job No: 25431

Position: Easting: 6111433.42
Northing: 4823387.56
Elevation: 337.84(Ft)
Inclination:

Date Drilled: 04/11/2025
Drill Rig Truckmount Drill Rig
Drill Supplier Terra Firma
Logged/Checked: ND/MG

Depth (m)	ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	Blow Counts	Recovery (%)	SPT>N	Remarks
		TOPSOIL						
0.176	102.9 0.2	**FILL** SAND and SILT light brown, moist, compact, trace gravel	SS1A SS1B	SPT	1-5-5-5 N = 10	100 100	10	
0.762	102.3 0.8	**Silty Sand** brown/grey, dry, compact, some gravel, trace clay	SS2	SPT	2-7-10-19 N = 17	100	17	
1.525	101.5 1.5	**TILL** SAND and SILT, brown/grey, dry, very dense, some gravel, trace clay	SS3	SPT	15-17-18-30 N = 35	100	35	
		BH25-4 Refusal on inferred bedrock at 2.29 m						Borehole terminated in Till. Wet soils were not encountered. Standing water was not observed. Caving was not encountered.

Legend: SPT>N ●; UCS -○-; QT ⊗; SV ▼; SV>Resi; Moisture Content %>MC ●

Borehole Record: BH25-5

AllRock Consulting Ltd

Project: 2184 & 2194 Oneida Crescent
Location: 2184 Oneida Crescent, Mississauga, ON L5C 1V6, Canada
Loc Comment:
Job No: 25431

Position: Easting: 611170.26
Northing: 4823264.94
Elevation: 258.1(Ft)
Inclination:

Date Drilled: 04/11/2025
Drill Rig Truckmount Drill Rig
Drill Supplier Terra Firma
Logged/Checked: ND/MG

Depth (m)	ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	Blow Counts	Recovery (%)	Logged Remarks
0.1	78.6 0.1	**TOPSOIL**	SS1A	SPT	6-4-6-7 N = 10	75	
		FILL gravelly SAND brown, moist, compact, trace silt	SS1B			75	
1.0			SS2	SPT	7-8-6-6 N = 14	25	- occasional cobbles
1.525	77.2 1.5	**FILL** gravelly SILTY SAND brown/black, wet, loose, trace clay	SS3	SPT	3-4-5-5 N = 9	50	
2.0			SS4	SPT	5-6-6-6 N = 12	50	
3.0 3.05	75.7 3.0	**TILL** SAND and SILT, grey, dry, very dense some gravel, some silt	SS5	SPT	30-27-41-50/5" N = N > 50		

SPT>N ● (0, 20, 40, 60, 80, 100)
UCS, QT, SV, SV>Resi (0, 20, 40, 60, 80, 100)
Moisture Content %>MC ● (0, 10, 20, 30)

BH25-5 Terminated at 3.63 m due to target depth achieved

Borehole terminated in Till. Wet soils were encountered at 1.5 mbgs. Standing water was not observed. Caving was not encountered.

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Appendix B

Laboratory Testing Results

SIEVE ANALYSIS OF AGGREGATES
LS-602

AllRock Consulting Ltd
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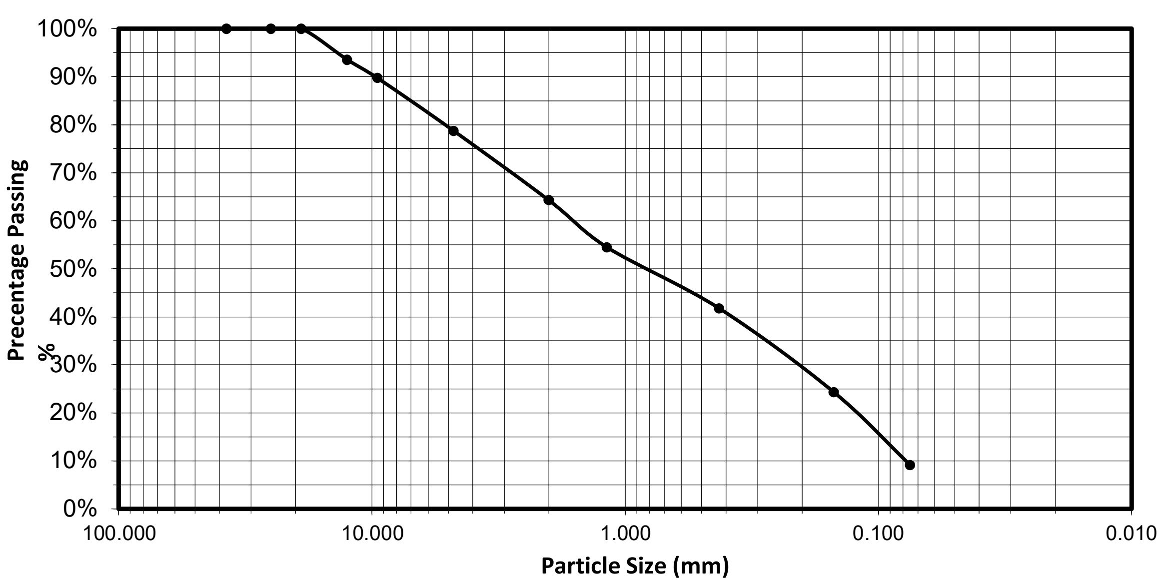
M9W 5R6, Toronto, Ont.

Project:	2184 & 2194 Oneida Crescent	**Project Number**	25431
Client:	C.F. Crozier & Associates Inc.	**Sample Classification:**	Gravelly Sand
Sample No.	BH25-2 SS3	**Sample Depth**	5m
Date Sampled	Oct 2 2025	**Date Tested:**	Oct 27 2025
Material Spec:			

Sieve Sizes					Remarks
Inches	mm	Lower Limit	Uppder Limit	Tested Sample	
1.5	37.5			100.0%	More Information Available Upon Request.
1	25.4			100.0%	
3/4	19			100.0%	
1/2	12.7			93.5%	Sampled By:
3/8	9.53			89.7%	ND
#4	4.75			78.7%	Tested By:
#10	2			64.3%	ND
#16	1.2			54.5%	Approved By
#40	0.425			41.7%	MG
#100	0.15			24.3%	Moisture Content
#200	0.075			9.1%	14.3%

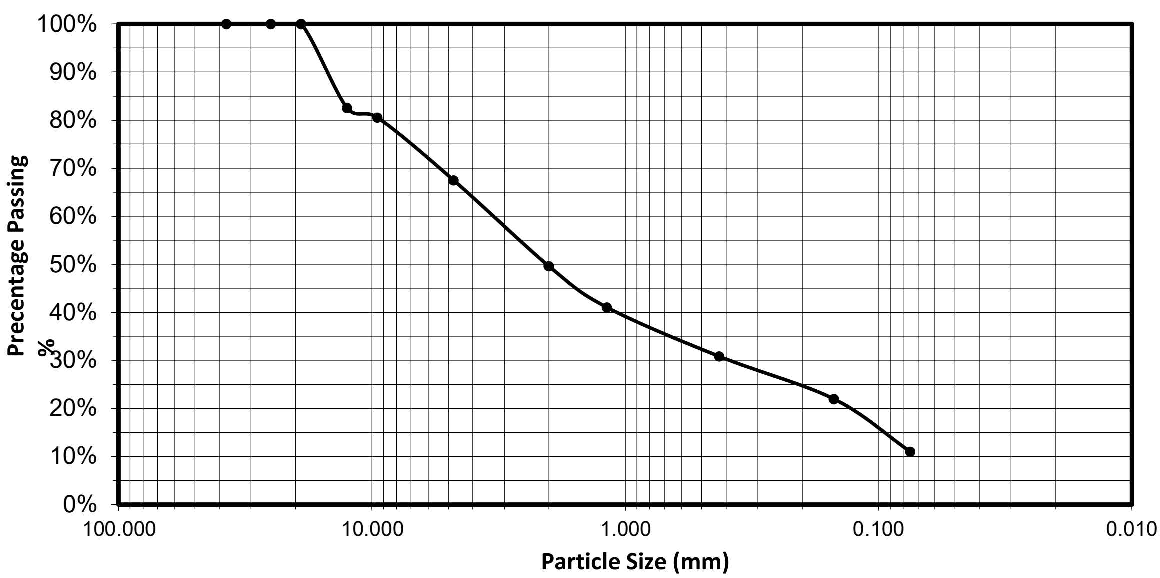


SIEVE ANALYSIS OF AGGREGATES
LS-602

AllRock Consulting Ltd
24 Brydon Drive, Unit #5
M9W 5R6, Toronto, Ont.

Project:	2184 & 2194 Oneida Crescent	**Project Number**	25431
Client:	C.F. Crozier & Associates Inc.	**Sample Classification:**	Gravel and Sand
Sample No.	BH25-4 SS2	**Sample Depth**	0.8m
Date Sampled	Oct 2 2025	**Date Tested:**	Oct 27 2025
Material Spec:			

Sieve Sizes					Remarks
Inches	mm	Lower Limit	Uppder Limit	Tested Sample	
1.5	37.5			100.0%	More Information Available Upon Request.
1	25.4			100.0%	
3/4	19			100.0%	
1/2	12.7			82.5%	Sampled By:
3/8	9.53			80.5%	ND
#4	4.75			67.5%	Tested By:
#10	2			49.6%	ND
#16	1.2			41.0%	Approved By
#40	0.425			30.8%	MG
#100	0.15			22.0%	Moisture Content
#200	0.075			11.0%	17.3%



Consulting Ltd

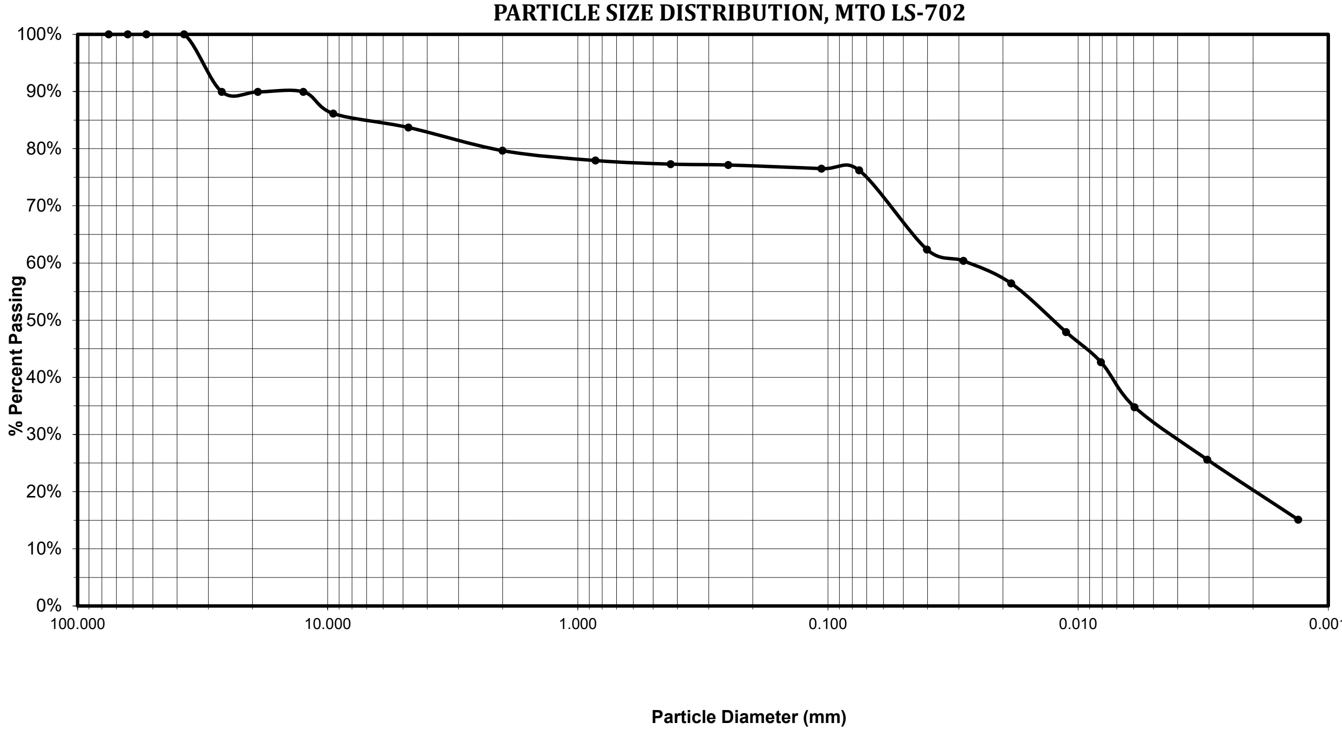
GRAIN SIZE ANALYSIS AND
HYDROMETER TEST REPORT
MTO LS-602, 702, AND 703/704

AllRock Consulting Ltd
24 Brydon Drive, Unit #5
Etobicoke, ON. M9W 5R6

Project Information	
Project Name:	2194 Oneida
Project No.:	25431
Client:	Crozier Consulting Engineers
Borehole / Test Pit No.:	BH25-1-SS4
Sample Depth:	NS
Sample No.:	25-209
Sampled By:	NS
Sample Description:	Soil Sample
Sample Natural M/C %:	8.5%
Date Sampled:	Friday, November 7, 2025
Tested By:	K.Awwadeh
Date Tested:	Tuesday, November 25, 2025
Reviewed By:	B.Goodings

Grain Size Analysis		Hydrometer Analysis	
Sieve Size (mm)	% Passing	Diameter (mm)	% Passing
75.0	100%	0.040138649	62.3%
63.0	100%	0.028683468	60.4%
53.0	100%	0.018516078	56.4%
37.5	100%	0.01115775	47.9%
26.5	90%	0.008099989	42.6%
19.0	90%	0.005939187	34.8%
12.5	90%	0.003040106	25.6%
9.5	86%	0.001317242	15.1%
4.8	84%	ATTERBERG LIMITS, %	
2.0	80%	Plastic Limit	-
0.850	78%	Liquid Limit	-
0.425	77%	Plastic Index	-
0.250	77%		
0.106	77%		
0.075	76%		

PARTICLE SIZE DISTRIBUTION, MTO LS-702



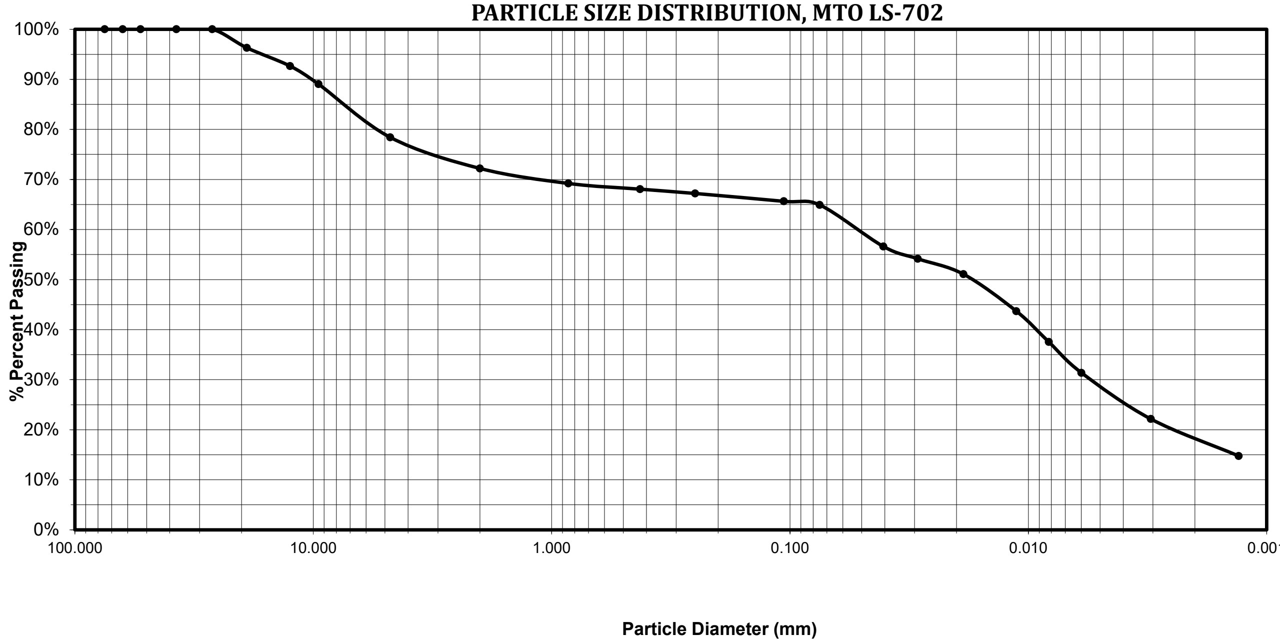
AllRock Consulting Ltd
24 Brydon Drive, Unit #5
Etobicoke, ON. M9W 5R6

GRAIN SIZE ANALYSIS AND HYDROMETER TEST REPORT MTO LS-602, 702, AND 703/704

Project Information	
Project Name:	2194 Oneida
Project No.:	25431
Client:	Crozier Consulting Engineers
Borehole / Test Pit No.:	BH25-3-SS4
Sample Depth:	NS
Sample No.:	25-208
Sampled By:	NS
Sample Description:	Soil Sample
Sample Natural M/C %:	12.3%
Date Sampled:	Friday, November 7, 2025
Tested By:	K.Awwadeh
Date Tested:	Tuesday, November 25, 2025
Reviewed By:	B.Goodings

Grain Size Analysis		Hydrometer Analysis	
Sieve Size (mm)	% Passing	Diameter (mm)	% Passing
75.0	100%	0.04056	56.6%
63.0	100%	0.02908	54.2%
53.0	100%	0.01870	51.1%
37.5	100%	0.01123	43.7%
26.5	100%	0.00820	37.5%
19.0	96%	0.00599	31.4%
12.5	93%	0.00306	22.2%
9.5	89%	0.00131	14.8%
4.8	78%	ATTERBERG LIMITS, %	
2.0	72%	Plastic Limit	-
0.850	69%	Liquid Limit	-
0.425	68%	Plastic Index	-
0.250	67%		
0.106	66%		
0.075	65%		

PARTICLE SIZE DISTRIBUTION, MTO LS-702



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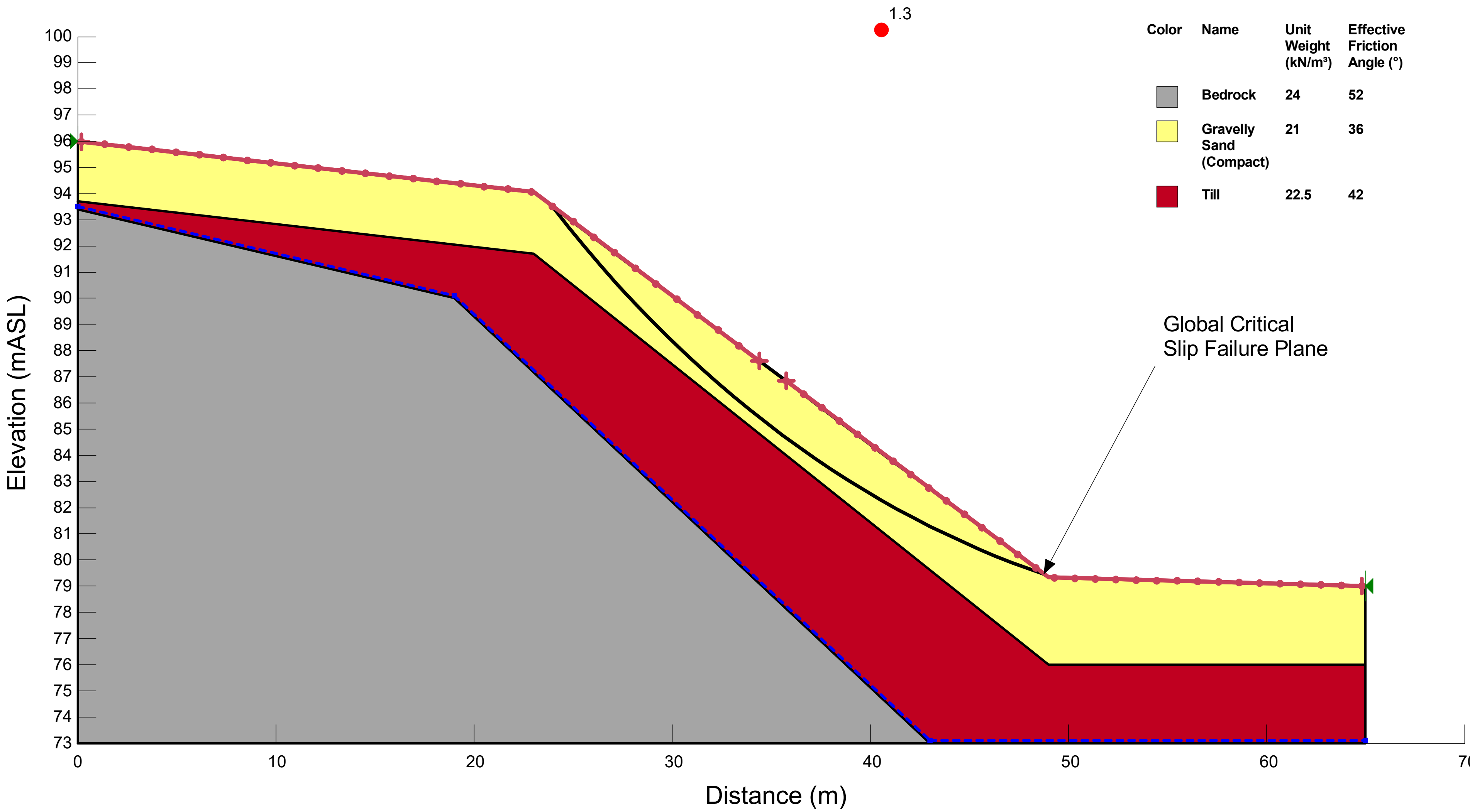
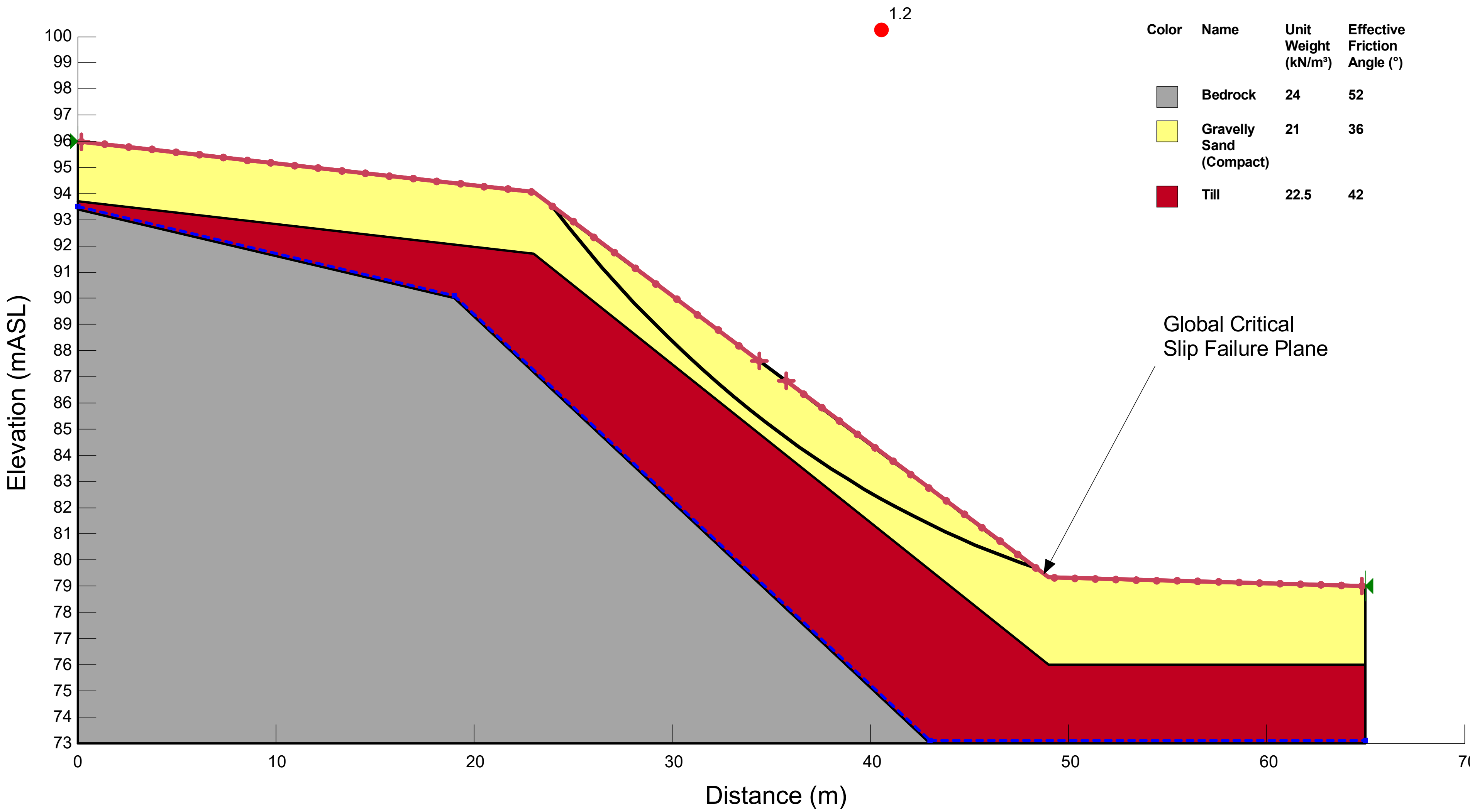
Appendix C

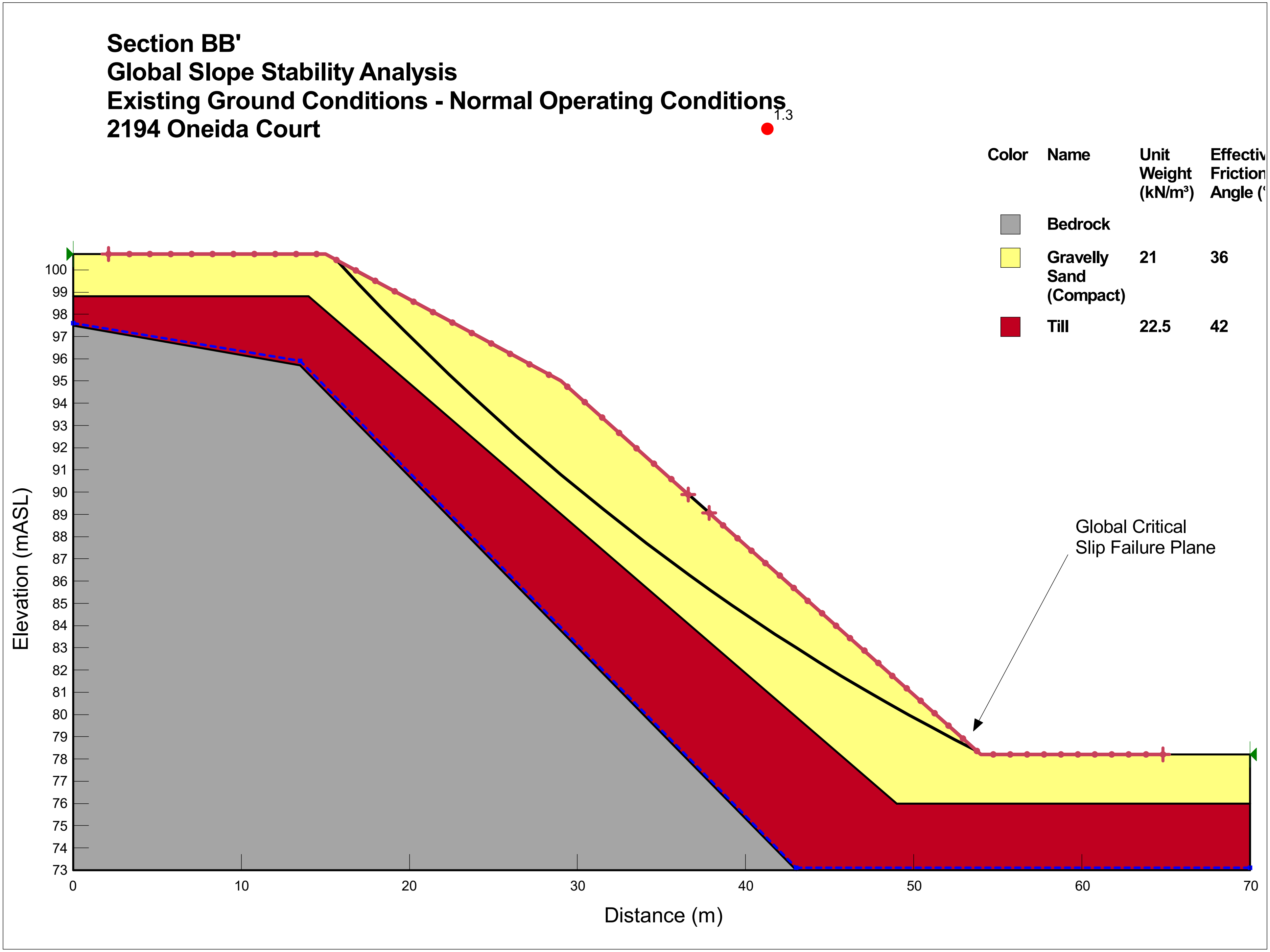
Slope Stability Models

Section AA'
Global Slope Stability Analysis
Existing Ground Conditions - Normal Operating Conditions
2194 Oneida Court

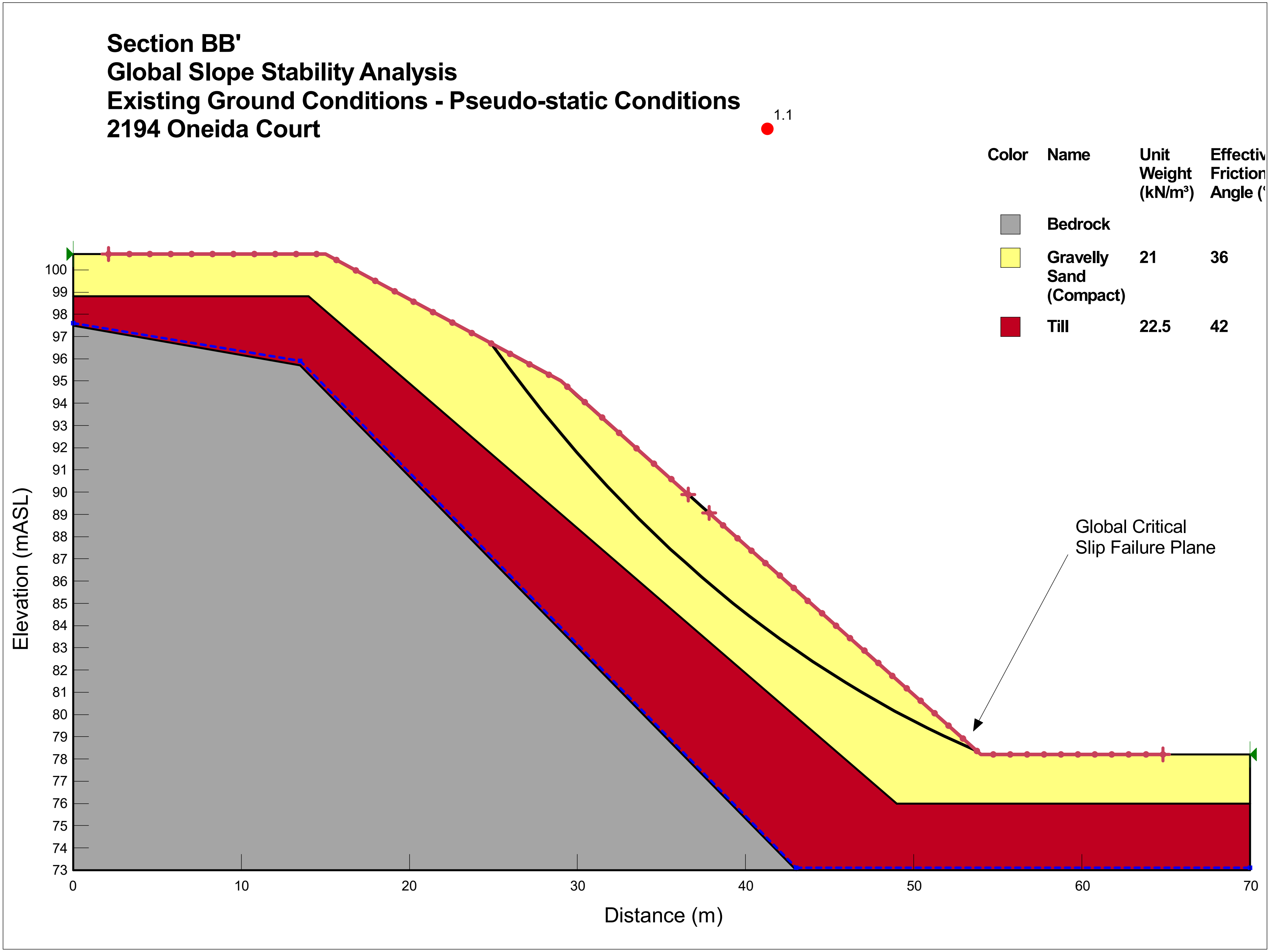


Section AA'
Global Slope Stability Analysis
Existing Ground Conditions - Pseudo-Static Conditions
2194 Oneida Court





Section BB'
Global Slope Stability Analysis
Existing Ground Conditions - Normal Operating Conditions
2194 Oneida Court
1.3
Color
Name
Unit Weight (kN/m³)
Effectiv Friction Angle (°
Bedrock
Gravelly Sand (Compact)
21
36
Till
22.5
42
Global Critical Slip Failure Plane
Elevation (mASL)
Distance (m)
100
99
98
97
96
95
94
93
92
91
90
89
88
87
86
85
84
83
82
81
80
79
78
77
76
75
74
73
0
10
20
30
40
50
60
70


Section BB'
Global Slope Stability Analysis
Existing Ground Conditions - Pseudo-static Conditions
2194 Oneida Court
1.1
Color
Name
Unit Weight (kN/m³)
Effectiv Friction Angle (
Bedrock
Gravelly Sand (Compact)
21
36
Till
22.5
42
Global Critical Slip Failure Plane
Elevation (mASL)
Distance (m)
100
99
98
97
96
95
94
93
92
91
90
89
88
87
86
85
84
83
82
81
80
79
78
77
76
75
74
73
0
10
20
30
40
50
60
70

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: 43.555N 79.624W **User File Reference:** 2194 Oneida 2025-12-05 17:17 UT

Requested by: M Garrison, AllRock Consulting

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)	0.239	0.116	0.059	0.011
Sa (0.1)	0.287	0.147	0.079	0.018
Sa (0.2)	0.238	0.125	0.071	0.020
Sa (0.3)	0.177	0.096	0.057	0.017
Sa (0.5)	0.122	0.069	0.043	0.014
Sa (1.0)	0.060	0.036	0.023	0.007
Sa (2.0)	0.029	0.017	0.011	0.003
Sa (5.0)	0.007	0.004	0.002	0.001
Sa (10.0)	0.003	0.002	0.001	0.000
PGA (g)	0.153	0.078	0.043	0.010
PGV (m/s)	0.096	0.053	0.031	0.008

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