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Geotechnical Investigation
Proposed Residential Development
1086 Antochi Lane
Ottawa, Ontario

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Submitted to:

1910753 Ontario Inc.
6900 Sunset Boulevard
Ottawa, Ontario
K4P 1C5

Geotechnical Investigation
Proposed Residential Development
1086 Antochi Lane
Ottawa, Ontario

February 7, 2023
Project: 100152.004

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February 7, 2023 File: 100152.004

1910753 Ontario Inc.
6900 Sunset Boulevard
Ottawa, Ontario
K4P 1C5

Attention: R. Verzeroli

**Re: Geotechnical Investigation
Proposed Residential Development
1086 Antochi Lane
Ottawa, Ontario**

Please find enclosed our geotechnical investigation report for the above noted project, in accordance with our proposal dated May 11, 2021. This report was prepared by Mr. Alex Meacoe, P.Eng., and reviewed by Mr. Brent Wiebe, P.Eng..

Alex Meacoe, P.Eng. Brent Wiebe, P.Eng.

FZ/WAM/BW

Enclosures
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed residential development to be located at 1086 Antochi Lane, Ottawa, Ontario. The purpose of the investigation was to identify the general subsurface and groundwater conditions at the site by means of a limited number of boreholes and test pits 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.

2.0 BACKGROUND

2.1 Project Description

Plans are being prepared for a residential development to be located at 1086 Antochi Lane in Ottawa (Manotick), Ontario. The proposed development consists of 18 semi detached and one single detached dwelling, with a maximum height of three stories above ground and one basement level.

The existing site is currently developed with eight houses on the site. The site is irregular in shape with plan dimensions of approximately 110 metres by 90 metres.

2.2 Review of Geology Maps

Surficial geology maps of the Ottawa area indicate that the site is underlain by silty clay over glacial till. Bedrock geology maps of the area show that the overburden deposits are underlain by dolostone of the Oxford formation. Drift thickness mapping indicates that the bedrock surface is expected at depths ranging from about 5 to 10 metres below ground surface. Fill material associated with the existing development of the site should be anticipated.

3.0 SUBSURFACE INVESTIGATION

3.1 Geotechnical Investigation

The fieldwork for the geotechnical investigation was carried out on November 18, 2021. Two boreholes (numbered 21-01 and 21-02) and one test pit (numbered 21-03) were advanced at the locations shown on the Site Plan, Figure 1.

The boreholes were advanced with a rubber tire, track mounted hollow stem auger drill rig supplied and operated by CCC Geotechnical and Environmental Drilling of Ottawa, Ontario. The boreholes were advanced to depths about 3.9 and 3.8 metres below the existing ground surface in boreholes 21-01 and 21-02, respectively.

Standard penetration tests were carried out in the boreholes at regular intervals of depth and samples of the soils encountered were recovered using a 50 millimetre diameter split barrel sampler.

Well screens were sealed in the overburden in boreholes 21-01 and 21-02 to measure the groundwater levels and for hydraulic conductivity testing.

The test pit was advanced with a vacuum truck supplied and operated by Badger Daylighting. The test pit was excavated to a depth of about 1.1 metres below the existing ground surface.

The subsurface conditions in the test pit were determined based on visual and tactile examination of soils exposed on the sides and bottom of the excavation.

The fieldwork was supervised throughout by a member of our engineering staff who directed the drilling and excavating operations, logged the samples and test holes, and carried out the in-situ testing. Following completion of the drilling, the soil samples were returned to our laboratory for examination by a geotechnical engineer and for laboratory testing. Selected soil samples were tested for water content and grain size distribution testing.

One sample of soil obtained from borehole 21-01 was sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

The test hole locations were selected by GEMTEC and positioned on site relative to existing features. The ground surface elevations at the test hole locations were determined using a Trimble R10 GPS. The elevations are referenced to geodetic datum NAD83 (CSRS) Epoch 2010, vertical network CGVD1928.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Test Hole Sheets in Appendix A. The results of the laboratory classification testing are provided on the Record of Borehole sheets and in Appendix B. The results of chemical testing completed on one soil sample are provided in Appendix C. The results of the slope stability analysis are provided in Appendix D. The approximate locations of the test holes are shown on the Site Plan, Figure 1.

3.2 Description of Slope

A site reconnaissance was carried out on September 17, 2021 by a member of engineering staff.

At the time of the site visits, the geometry of the slopes along the Rideau River were measured at a total of four locations using precision GPS surveying equipment. The cross sections were positioned at the site by GEMTEC personnel. The locations of the five cross sections considered are provided on Figure 1. Cross sections of the slopes are provided in Appendix D.

The geometries of the cross sections considered are summarized below in Table 3.1:

Table 3.1 – Slope Cross Section Height and Slope Inclination

Cross Section	Slope Height (metres)	Overall inclination from horizontal (degrees)
A-A	0.7	5 to 39
B-B	1.3	5 to 57
C-C	1.0	4 to 24
D-D	1.0	5 to 79[1]

Notes:

1. Slope cross section D-D was measured at the location of an existing retaining wall. The slope angle at the retaining wall is about 90 degrees from horizontal.

In general, the slopes of the Rideau River are vegetated with grass, shrubs, small to large trees, rip rap, and a retaining wall. Erosion protection consisting of rip rap was observed at the toe of the slope at sections BB and CC. Slope AA and DD had significant vegetation and a retaining wall, respectively, and, as such, the state of erosion was not observed. No signs of overall slope instability (i.e., rotational failures) were observed at the site.

4.0 SUBSURFACE CONDITIONS

4.1 General

The soil conditions logged in the test holes from the current investigation are provided on the Record of Test Hole Sheets in Appendix A. The test hole 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. Subsurface conditions at locations other than the test hole locations may vary from the conditions encountered in the test holes. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site.

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 GEMTEC does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered in the boreholes and test pits advanced as part of the current investigation.

4.2 Topsoil

Topsoil was encountered at the ground surface in borehole 20-01 and test pit 21-03. The topsoil has a thickness of about 250 and 400 millimetres at these locations, respectively.

4.3 Pavement Structure

Borehole 21-02 was advanced through the pavement structure of the drive lanes at 1086 Antochi Lane. The pavement structure consists of about 100 millimetres of asphaltic concrete over about 30 millimetres of sand and gravel base layer. A buried layer of asphaltic concrete with a thickness of about 50 millimetres was encountered below the base layer.

A sand and gravel base layer, with a thickness of about 380 millimetres, was encountered below the buried asphaltic concrete layer.

4.4 Silty Clay

A native deposit of silty clay was encountered below the pavement structure in borehole 21-02. The silty clay has a thickness of about 0.4 metres and extends to a depth of about 1.0 metres below the existing surface grade.

One standard penetration test carried out in the silty clay gave an N value of 16 blows per 0.3 metres of penetration. The results of the in situ testing reflects a stiff to very stiff consistency.

The water content measured on one sample of the silty clay is about 17 percent.

4.5 Silty Sand

Native deposits of silty sand were encountered below the topsoil in borehole 20-01 and test pit 21-03. The silty sand has a thickness of about 1.1 and 0.4 metres and extends to depths of about 1.3 and 0.8 metres at these locations, respectively. Grey brown silty clay layers were observed within the silty sand in borehole 21-01.

Two standard penetration tests carried out in the silty sand gave N values of 3 blows per 0.3 metres of penetration, which indicates a very loose relative density.

One grain size distribution test was carried out on a sample of the silty sand. The results are provided in Appendix B and summarized in Table 4.1.

Table 4.1 – Summary of Grain Size Distribution Test (Silty Sand)

Borehole Number	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
21-01	2	0.8 – 1.4	0	56	26	18

The water content measured on two samples of the silty sand was about 29 and 32 percent.

4.6 Glacial Till

Native deposits of glacial till were encountered below the topsoil pavement structure, silty sand and/or silty clay, where encountered, in the test holes. The glacial till deposit was not fully penetrated, but was proven to depths ranging from about 1.1 to 3.9 metres below the ground surface. Glacial till can be described as a heterogeneous mix of all grain sizes, which at this site is described as a silty sand with some gravel, cobbles and boulders.

Standard penetration tests carried out in the glacial till gave N values ranging from 4 to greater than 50 blows for less than 0.3 metres of penetration, but more generally between 4 and 16 blows, which indicates a very loose to compact relative density. The high blow counts likely represent the presence of cobbles or boulders within the glacial till deposit or the bedrock surface rather than the relative density of the soil matrix.

One grain size distribution test was carried out on a sample of the glacial till. The results are provided in Appendix B and summarized in Table 4.2.

Table 4.2 – Summary of Grain Size Distribution Test (Glacial Till)

Borehole Number	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
21-02	4	2.3 – 2.9	19	43	30	8

The water content measured on seven samples of the glacial till ranges from about 8 to 25 percent.

4.7 Refusal

Auger refusal was encountered in boreholes 21-01 and 21-02 at depths of about 3.9 and 3.8 metres below the existing ground surface, respectively. The auger refusal likely represents the presence of cobbles or boulders within the glacial till deposit or the bedrock surface.

Refusal to hydro-excavation advancement was encountered on cobbles and boulders within the glacial till in test pit 21-03 at a depth of about 1.1 metres below the existing ground surface.

A summary of the refusal depths and elevations is provided in Table 4.3.

Table 4.3 – Summary of Auger Refusal Depth and Elevation

Borehole/Test Pit Number	Ground Surface Elevation (metres)	Depth to Refusal (metres)	Refusal Elevation (metres)
21-01	86.8	3.9	82.9
21-02	86.6	3.8	82.8
21-03	86.7	1.1[1]	85.6[1]

Notes:

1. Refusal to hydro-excavation was encountered on cobbles and boulders in the glacial till

4.8 Groundwater Levels

Well screens were sealed in the overburden at boreholes 21-01, 21-02 for measurement of the groundwater levels. The groundwater levels in the monitoring wells were measured on December 3, 2021. The groundwater level depth and elevations are summarized in Table 4.4.

Table 4.4 – Summary of Groundwater Levels

Borehole/Test Pit Number	Groundwater Depth (metres)	Groundwater Elevation (metres)	Date
21-1	1.5	85.3	December 3, 2021
21-2	0.8	85.7	December 3, 2021

The groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

4.9 Hydraulic Test Results

The results of the hydraulic testing carried out in the monitoring wells are provided in Appendix E. A summary of the recovery measurements made during the hydraulic testing carried out by introducing/removing a slug into the monitoring wells is provided in Table 4.5.

Table 4.5 – Summary of Falling Head Test Results

Borehole	Geological Material Tested	Static Groundwater Depth (metres TPVC[1])	Slug Displacement (metres)	Recovery Time (minutes)	Recovery[2] (percent)
21-01	Glacial Till	1.53	1.05	1	100

Borehole	Geological Material Tested	Static Groundwater Depth (metres TPVC[1])	Slug Displacement (metres)	Recovery Time (minutes)	Recovery[2] (percent)
21-02	Glacial Till	0.84	0.54	12	87

Notes:
1. Static groundwater level measured from top of PVC.
2. Observed displacement greater or less than the slug displacement is often caused by rapid removal of the slug, which is captured by the datalogger, measuring at 0.5 second intervals.

A summary of the recovery measurements made during the rising head test carried out by removing the slug from the well screens is provided in Table 4.6.

Table 4.6 – Summary of Rising Head Test Results

Borehole	Geological Material Tested	Static Groundwater Depth[1] (metres TPVC)	Slug Displacement (metres)	Recovery Time (minutes)	Recovery[2] (percent)
21-01	Glacial Till	1.53	0.58	1.5	100
21-02	Glacial Till	0.75	0.68	15	98

Notes:
1. Static groundwater level measured from top of PVC.
2. Observed displacement greater or less than the slug displacement is often caused by rapid removal of the slug, which is captured by the datalogger, measuring at 0.5 second intervals.

Hydraulic conductivities calculated from the hydraulic test (falling and rising head tests) results are provided in Table 4.7.

Table 4.7 – Calculated Hydraulic Conductivities

Borehole	Geological Material Monitored	Calculated Hydraulic Conductivity, k (metres per second)[1,2]	
		Falling Head Test by Introducing a Slug	Rising Head Test by Removing a Slug
21-01	Glacial Till	8×10^{-5}	8×10^{-5}
21-02	Glacial Till	2×10^{-5}	5×10^{-6}

Notes:
1. The hydraulic conductivities were calculated using the Hvorslev analysis.
2. Displacement volume of slug used in analysis for all boreholes.

The hydraulic conductivities calculated from the rising and fall head test completed in the glacial till boreholes ranged from 5 x 10^{-6} to 8 x 10^{-5} metres per second. The calculated hydraulic conductivity are generally within the literature values (Freeze and Cherry, 1979) for glacial till, which has a hydraulic conductivity ranging from 5 x 10^{-12} to 5 x 10^{-6} metres per second. The slightly higher in situ hydraulic conductivity likely results from the high gravel and sand content in the glacial till of 19 and 43 percent, respectively, as shown in Table 4.2.

4.10 Chemistry Relating to Corrosion

One soil sample obtained from borehole 21-01 was sent to Paracel Laboratories for basic chemical testing relating to corrosion of buried concrete and steel. The results of chemical testing are provided in Appendix C and summarized in Table 4.8 below.

Table 4.8 – Summary of Corrosion Testing

Parameter	Borehole 21-1 Sample 3
Chloride Content (µg/g)	27
Resistivity (Ohm.m)	63.9
Conductivity (µs/cm)	157
pH	7.86
Sulphate Content (µg/g)	23

5.0 GEOTECHNICAL RECOMMENDATIONS AND GUIDELINES

5.1 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. As such, lot specific subgrade evaluations should be carried out by experienced geotechnical personnel to support the lot development plans and to confirm the recommendations presented in this report. 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 at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from off-site sources are outside the terms of reference for this report.

5.2 Grade Raise Restrictions

The site is underlain by native deposits of silty clay, silty sand, and glacial till.

Based on the test pit and borehole information, there are no grade raise restrictions at the site, from a geotechnical perspective. The settlement due to compression of the native soils as a result of fill placement should be relatively small and should occur during or shortly after the fill placement.

5.3 Proposed Houses

5.3.1 Overburden Excavation

The excavations for the foundations should be taken through any surficial topsoil, pavement structure and into the native overburden deposits. The sides of the excavations 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 shallow native overburden deposits can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical extending upwards from the base of the excavation.

Excavation of the native soils above the groundwater should not present any excavation constraints. In contrast, excavation in the native sandy deposits below the groundwater level could present constraints. Groundwater inflow from the sandy deposits could cause sloughing of the sides of the excavation and disturbance to the soils at the bottom of the excavation. Flatter side slopes of 3 horizontal to 1 vertical will be required if excavation is required below the groundwater level in sandy deposits.

Based on our observations on site, groundwater inflow from the overburden deposits into the excavations should be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant affect on nearby structures and services.

The silty clay deposit is sensitive to disturbance from ponded water, vibration and construction traffic. As such, care should be taken when excavating to avoid disturbance to the silty clay deposit, and it is suggested that final trimming to subgrade level be carried out using a hydraulic shovel equipped with a flat blade bucket. Allowance should be made to remove and replace any disturbed silty clay with compacted sand and gravel, such as that meeting OPSS Granular A or Granular B Type II, where required.

5.3.2 Groundwater Pumping

The groundwater levels measured on December 3, 2021 range from about 0.8 to 1.5 metres below existing ground surface.

Any groundwater inflow into the excavations should be handled from within the excavations by pumping from filtered sumps. It is not expected that short term pumping during excavation will have a significant effect on nearby structures.

Suitable detention and filtration will be required before discharging the water to a sewer or ditch. The amount of water entering the excavation for the construction of individual foundations at this site will likely not exceed 50,000 litres per day and, therefore, it is not anticipated that an Environmental Activity and Sector Registry (EASR) will be required.

In order to reduce, not eliminate, the requirement for long term pumping from sump pumps it is recommended that underside of footing elevations be set a minimum of 0.3 metres above the seasonally high groundwater level.

5.3.3 Placement of Engineered Fill

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 subexcavation of disturbed 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 95 percent of the standard Proctor maximum dry density. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.3 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.

In areas where wet sandy soils are encountered at subgrade level, it may be necessary to place a woven geotextile meeting the requirements of OPSS 1860 Class I below the engineered fill and to statically compact the first lift of granular material to prevent subgrade disturbance. All seams in the geotextile should overlap at least 0.5 metres.

The test pit represents an area of disturbed soil. If the test pit is located within a proposed building footprint, the test pit should be subexcavated and backfilled with engineered fill material as described above. The sides of the subexcavated test pits should be sloped at 1 horizontal to 1 vertical, or flatter.

5.3.4 Spread Footing Design

The proposed houses could be founded on spread footings bearing on or within the native soil or on engineered fill above the native deposits. The topsoil and fill material are not considered suitable for the support of the proposed houses or concrete floor slabs and should be removed from the proposed building areas.

The proposed houses may be partially or fully located within the footprint of the existing houses on site. Although not directly encountered, or sampled, during the drilling fieldwork, a layer of fill material of unknown composition associated with the construction of the existing houses on site will be located surrounding the houses to a depth of up to about 2.5 metres below ground surface. As such, the existing foundation elements and fill material associated with the past construction of the houses will need to be removed from the proposed building areas.

After the removal of the existing houses and associated fill material, and where the existing subgrade surface is below the proposed founding level, the grade could be raised with compacted granular material (engineered fill) with a Class II non-woven geotextile having an FOS not exceeding 100 microns (OPSS 1860) placed on the subgrade. The engineered fill should consist of granular material meeting OPSS requirements for Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. To provide adequate spread of load beneath the footings, the engineered fill should extend horizontally at least 0.5 metres beyond the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter.

For design purposes, the following allowable bearing pressures should be used to size the spread footing foundations:

Table 5.1 – Allowable Bearing Pressures for Foundations

Subgrade Material	Allowable Bearing Pressure for Foundations (kilopascals)
Silty clay, silty sand, and glacial till	100
Engineered fill material, over undisturbed, native deposits (minimum thickness of 0.6 metres of engineered fill)	150

It is pointed out that the deposits of silty clay and silty sand near or below the groundwater level may become disturbed following excavation. If disturbance to these deposits occurs, one solution would be to wait several days to allow the porewater pressures to dissipate. Alternatively, the groundwater level could be lowered in advance of excavation by pumping from sump pits, possibly combined with ditching around the perimeter of the excavations.

Some of the native soils at this site are sensitive to construction operations, from ponded water and frost action. The construction operations should therefore be carried out in a manner that minimizes disturbance of the subgrade surfaces.

The post construction total and differential settlement of footings designed for the above bearing values should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces and provided that any engineered fill material is compacted to the required density.

As indicated above, the underside of footing level should be set a minimum of 0.3 metres above the seasonally high groundwater level.

5.3.5 Frost Protection of Foundations

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces which are cleaned of snow cover during the winter months should be provided with a minimum of 1.8 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.

5.3.6 Basement Foundation Wall Backfill and Drainage

In accordance with the Ontario Building Code, the following alternatives could be considered for drainage of the basement foundation walls:

- Damp proof the exterior of the foundation walls and backfill the walls with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II; or,
- Damp proof the exterior of the foundation walls and install an approved proprietary drainage material on the exterior of the foundation walls and backfill the walls with native material or imported soil.

A perforated plastic foundation drain with a surround of clear crushed stone should be installed on the exterior of the foundation walls at the underside of footing level. A nonwoven geotextile should be placed between the top of the clear stone and any sandy foundation wall backfill material to avoid loss of sand backfill into the voids in the clear stone (and possible post construction settlement of the ground around the houses). The top of the drain should be located below the bottom of the floor slab. The drain should outlet to a sump from which the water is pumped or should drain by gravity to an adjacent storm sewer.

5.3.7 Garage Foundation and Pier Backfill

To avoid adfreeze between the unheated garage foundation walls and the wall backfill and possible jacking (heaving) of the foundation walls, the interior and exterior of the garage foundation walls should be backfilled with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II. The backfill within the garage should be compacted in maximum 300 millimetres thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

Alternatively, the interior of the garages could be filled with 19 millimetre clear crushed stone. In areas where the subgrade consists of silty clay, silty sand, or sand, a suitable nonwoven geotextile should be placed over the subgrade prior to the placement of clear stone to prevent ingress of fines into voids in the clear stone and possible settlement/cracking of the slab. Clear, crushed stone should be nominally compacted (at least 2 passes of a diesel plate compactor) in maximum 300 millimetre thick lifts to reduce the potential for post construction densification of the material.

The backfill against isolated (unheated) walls or piers should consist of free draining, non-frost susceptible material, such as sand/sand and gravel meeting OPSS Granular B Type I or II requirements. Other measures to prevent frost jacking of these foundation elements could be provided, if required.

5.3.8 Basement Concrete Slab Support

To provide predictable settlement performance of the basement slab, all topsoil, fill material, disturbed soil, and other deleterious materials should be removed from the slab area.

The base for the floor slab should consist of 19 millimetre clear crushed stone. Allowance should be made for between 150 and 200 millimetres of granular base material.

The clear crushed stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor. In areas where the subgrade consists of silty sand, sand, or silty clay, a suitable nonwoven geotextile should be placed over the subgrade prior to the placement of clear stone to prevent ingress of fines into voids in the clear stone and possible settlement/cracking of the slab.

Underfloor drainage should be provided below the floor slab. If clear crushed stone is used below the floor slab, underfloor drains are not considered essential provided that stub drains are installed to link any hydraulically isolated areas in the basement. The clear stone below the floor slab should by hydraulically connected to the sump pit.

Basement floor slabs should be constructed in accordance with guidelines provided in ACI 302.1R-04 “Guide for Concrete Floor and Slab Construction”.

A polyethylene vapour barrier should be installed below the basement floor slabs.

5.3.9 Swimming Pools

We do not anticipate any geotechnical concerns with swimming pool construction within the residential development.

5.3.10 Seismic Site Classification and Liquefaction Potential

Based on the results of the standard penetration and the vane shear strength testing carried out as part of this investigation, it is recommended that seismic Site Class D be used for the design of residential structures on the subject site.

Also, based on the results of the standard penetration testing, in our opinion, the native overburden deposits below the proposed foundations, which are composed of silty clay and glacial till are not prone to liquefaction. It is noted that deposits of very loose silty sand were encountered in borehole 21-01 and in test pit 21-03, however, these deposits are relatively thin and located within 1.3 metres of existing ground surface (i.e. above probable underside of foundation depths).

5.4 Site Services

5.4.1 Excavation

The overburden excavations for the site services will be carried out through topsoil, silty sand, silty clay, and into the glacial till.

In the overburden, the excavation for flexible service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.010 for Type 3 soil. The excavation for rigid service pipes should be in accordance with OPSD 802.031 for Type 3 soil.

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, most of 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. For excavations below the groundwater, an allowance should be made for 3 horizontal to 1 vertical, or flatter, excavation slope.

As an alternative or where space constraints dictate, the service installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

Based on our observations on site, groundwater inflow from the overburden deposits into the excavations should be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant affect on nearby structures and services.

5.4.2 Groundwater Pumping

The groundwater levels measured on December 3, 2021 range from about 0.8 to 1.5 metres below existing ground surface.

Any groundwater inflow into the excavations should be handled from within the excavations by pumping from filtered sumps. It is not expected that short term pumping during excavation will have a significant effect on nearby structures given that most of the anticipated drawdown will occur within the till unit. In effect, it is anticipated that the thickness of saturated silty clay that may be dewatered at the site is less than 1 metre.

Suitable detention and filtration will be required before discharging the water to a sewer or ditch. The amount of water entering the excavation for the construction of storm sewer, sanitary sewer and watermain at this site may potentially exceed 400,000 litres per day based on the hydraulic conductivity of the till, water levels and proximity to the river. Therefore, it is anticipated that a Permit to Take Water (PTTW) will be required.

In order reduce groundwater inflow volumes, if warranted based on water discharge options, the length of trench excavations open at any given time should be kept to a minimum.

5.4.3 Bedding and Cover

The bedding and cover for the proposed utilities should consist of least 150 millimetres of OPSS Granular A backfill placed in accordance with the applicable Ontario Standard Drawings (OPSD) for the type of underground utility installed. The use of 19 millimetre clear stone is not recommended and should not be permitted as bedding or cover.

The native overburden deposits below the groundwater level are sensitive to disturbance. An allowance should be made for a subbedding composed of at least 300 millimetres of OPSS Granular B Type II where these materials are encountered at subgrade level below the pipe.

Bedding, subbedding and cover materials should be placed in lifts not exceeding 200 millimetres thick and compacted to at least 98 percent of standard Proctor density (ASTM D698).

5.4.4 Trench Backfill

In areas where the service trench will be located below or in close proximity to existing or future areas of hard surfacing (i.e., access roadways and parking), acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent hard surfaced area. The depth of frost penetration in exposed areas can normally be taken as 1.8 metres below finished grade. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I.

It is anticipated that most of the inorganic overburden materials encountered during the subsurface investigation will be acceptable for reuse as trench backfill. Topsoil or other organic material should be wasted from the trench.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, curbs, driveways, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. The specified density for compaction of the backfill materials may be reduced where the trench backfill is not located below or in close proximity to existing or future areas of hard surfacing and/or structures, provided that some settlement above the trench is acceptable.

The silty clay deposits may have water contents that are too high for adequate compaction. Furthermore, depending on the weather conditions at the time of construction, some wetting of materials could occur. As such, the specified densities may not be possible to achieve and, as a consequence, some settlement of these backfill materials should be expected. Consideration could be given to implementing one or a combination of the following measures to reduce post construction settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to compaction;
- Reuse any wet materials in the lower part of the trenches and make provision to defer final paving of surface course (i.e., the Superpave 12.5 asphaltic concrete) in the roadway for 3 months, or longer, to allow the trench backfill settlement to occur and thereby improve the final roadway appearance.

5.4.5 Seepage Barriers

The granular bedding in the service trench could act as a “French Drain”, which could promote groundwater lowering. As such, we suggest that seepage barriers be installed along the service trenches at strategic locations. The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround to within the native backfill materials, and horizontally across the full width of the service trench excavation. The seepage barriers could consist of 1.5 metre wide dykes of compacted silty clay. The silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. The locations of the seepage barriers could be provided as the design progresses.

5.5 Sensitive Marine Clay – Effects on Trees

Silty clay deposits were encountered at this site, however, where encountered, the silty clay extends to a depth of only about 1 metres below ground surface. It is likely that the future grades at the site will generally match existing, and, as such, the silty clay will be located above the proposed underside of footing elevation, in which case, the City of Ottawa Tree Planting Guidelines do not apply.

5.6 Internal Roadways

5.6.1 Subgrade Preparation

In preparation for roadway construction at this site, all surficial topsoil, peat, and any soft, wet, disturbed, or deleterious materials should be removed from the proposed roadways. Any subexcavated areas could be filled with compacted earth borrow. Similarly, should it be necessary to raise the roadway grades at this site, material which meets OPSS specifications for Select Subgrade Material or Earth Borrow may be used. The select subgrade material or earth borrow should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment. Prior to placing granular material for the roadways, the exposed subgrade should be heavily proof rolled under suitable (dry) conditions, and inspected and approved by geotechnical personnel. Any soft areas evident from the proof rolling should be subexcavated and replaced with suitable earth borrow approved by the geotechnical engineer.

The subgrade should be shaped and crowned to promote drainage of the roadway granular materials.

5.6.2 Pavement Design

The following minimum pavement structure is suggested for the internal roadway at this site.

- 90 millimetre thick layer of asphaltic concrete (40 millimetres of Superpave 12.5 Traffic Level B over 50 millimetres of Superpave 12.5 Traffic Level B); over
- 150 millimetre thick layer of base (OPSS Granular A); over
- 400 millimetre thick layer of subbase (OPSS Granular B Type II).

The above pavement structures assumes that the roadway subgrade surface is prepared as described in this report. If the roadway subgrade surface is disturbed or wetted due to construction operations or precipitation, the granular thickness given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or to incorporate a woven geotextile separator between the roadway subgrade surface and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction. In our experience, a geotextile will likely be required in most cases where the subgrade consists of overburden, if the roadway construction is planned during the wet period of the year (such as the spring or fall).

Similarly, if the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the Granular B Type II, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

5.6.3 Granular Material Compaction

The pavement granular materials should be compacted in maximum 300 millimetre thick lifts to at least 99 percent of standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.6.4 Asphaltic Cement

Performance graded PG 58-34 asphaltic cement is recommended for local roadways while performance graded PG 64-34 asphalt is recommended for collector/arterial roadways and bus routes.

5.6.5 Pavement Transitions

As part of the roadway reconstruction, the new pavement will abut existing pavement at Antochi Lane. The following is suggested to improve the performance of the joint between the new and the existing pavements:

- Neatly saw cut the existing asphaltic concrete;
- Remove the asphaltic concrete and slope the bottom of the excavation within the existing granular base and subbase at 1 horizontal to 1 vertical, or flatter, to avoid undermining the existing asphaltic concrete.
- To avoid cracking of the asphaltic concrete due to an abrupt change in the thickness of the roadway granular materials where new pavement areas join with the existing pavements, the granular depths should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the existing pavement structure.
- Remove (mill off) 40 to 50 millimetres of the existing asphaltic concrete to a distance of 300 millimetres at the joint and tack coat the asphaltic concrete at the joint in accordance with the requirements in OPSS 310.

5.6.6 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. The subgrade should be crowned and shaped to drain to the ditches and/or catch basins to promote drainage of the pavement granular materials.

5.7 Corrosion of Buried Concrete and Steel

According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate in the soil sample recovered from borehole 21-01 can be classified as low. For low exposure conditions, any concrete that will be in contact with the native soil or groundwater could be batched with General Use (GU) type cement. The effects of freeze thaw in the presence of de-icing chemical (sodium chloride) near the buildings should be considered in selecting the air entrainment and the concrete mix proportions for any exposed concrete.

Based on the resistivity and pH of the soil samples tested the soil can be generally classified as non aggressive toward unprotected steel. It is noted that the corrosivity of the soil could vary throughout the year due to the application sodium chloride for de-icing.

6.0 SLOPE STABILITY ANALYSIS

6.1 General

The purpose of this preliminary stability assessment is to establish the ‘Erosion Hazard Limit’ for the site. This limit constitutes a safe setback for any proposed development at the site with respect to slope stability. The Erosion Hazard Limit was determined based on the Natural Hazard

Policies set forth in Section 3.1 of the Provincial Policy Statements of the Planning Act of Ontario. Current regulations restrict development within the Erosion Hazard Limit.

The slope stability analysis was carried out at Section ‘B-B’ using Slope/W, a two dimensional limit equilibrium slope stability program. The results of the slope stability analysis are provided in Appendix D.

6.2 Soil Strength Parameters

The soil conditions used in the stability analyses were based, in part, on the results of the boreholes and test pit advanced across the site. The slope stability analyses were carried out using glacial till strength parameters based on site specific studies in the area of the site. To determine the existing factor of safety against overall rotational failure, the slope stability analysis was carried out using drained soil parameters, which reflect long term conditions

The following table summarizes the soil parameters used in the analyses:

Table 6.1 – Slope Stability Soil Strength Parameters

Soil Type	Effective Angle of Internal Friction, φ (degrees)	Effective Cohesion, c′ (kilopascals)	Unit Weight, γ (kN/m^3)
Silty Sand	32	0	18
Glacial Till	34	0	20

The results of a stability analysis are highly dependent on the assumed groundwater conditions. The groundwater levels measured during this investigation range from about 85.3 to 85.7 metres, geodetic datum, which are generally consistent with the water level in the Rideau River (i.e. about 85.5 metres, geodetic datum), which was selected for this analyses.

The slope stability analyses were carried out using soil parameters, groundwater conditions and a slope profile that attempt to model the slopes in question but do not exactly represent the actual conditions.

For the purposes of this study, a computed factor of safety of less than 1.0 to 1.3 is considered to represent a slope bordering on failure to marginally stable, respectively; a factor of safety of 1.3 to 1.5 is considered to indicate a slope 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.

6.3 Existing Conditions

Based on the results of the analysis, the slopes along the Rideau River are not considered to be stable in their current configuration.

6.4 Setback Requirements

For unstable slopes, the distance from the unstable slope to the safe setback line is called 'Erosion Hazard Limit'. In accordance with the Ministry of Natural Resources (MNR) Technical Guide "Understanding Natural Hazards" dated 2001, the Erosion Hazard Limit consists of three components: (1) Stable Slope Allowance, (2) Toe Erosion Allowance, and (3) Erosion Access Allowance.

The Stable Slope Allowance, as described in the MNR procedures, encompasses the area where a factor of safety of less than 1.5 against overall rotational failure is calculated. At Section 'B-B' the slope stability analyses indicate that the existing slope along the Rideau River, in its current configuration, has a factor of safety against failure of less than 1.0 (refer to Figure D5, in Appendix D). The Stable Slope Allowance described in the MNR procedures extends about 2.5 metres horizontally from the toe of the slope.

In accordance with the MNR documents, a minimum Toe Erosion Allowance of between 5.0 to 8.0 metres is required for coarse granular (gravels) tills. Given that erosion protection was provided observed along the Rideau River, a Toe Erosion Allowance of 5.0 metres should be used.

The MNR procedures also include the application of a 6 metre wide Erosion Access Allowance beyond the Toe Erosion Allowance to allow for access by equipment to repair a possible failed slope. Based on the relatively low height of the slopes, relatively small equipment could likely be used for any slope repair. As such, it is considered that access between proposed residential dwellings or blocks should be sufficient to access the slope and an Erosion Access Allowance will not be required.

Based on the above information, the Erosion Hazard Limit for the slopes along the Rideau River will be 7.5 metres, as measured from the toe of the slope.

7.0 ADDITIONAL CONSIDERATIONS

7.1 Winter Construction

Provision must be made to prevent freezing of any soil below the level of any footings, slabs or services. Freezing of the soil could result in heaving related damage.

Any service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The materials on the sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

7.2 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, hoe ramming, foundation construction etc.) will cause ground vibration on and off of the site. The vibrations will attenuate with distance from the source, but may be felt at nearby structures.

7.3 Disposal of Excess Soil

It is noted that the professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination, including naturally occurring source of contamination, are outside the terms of reference for this report.

7.4 Design Review and Construction Observation

The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the design drawings be reviewed by the geotechnical engineer as the design progresses to ensure that the guidelines provided in this report have been interpreted as intended.

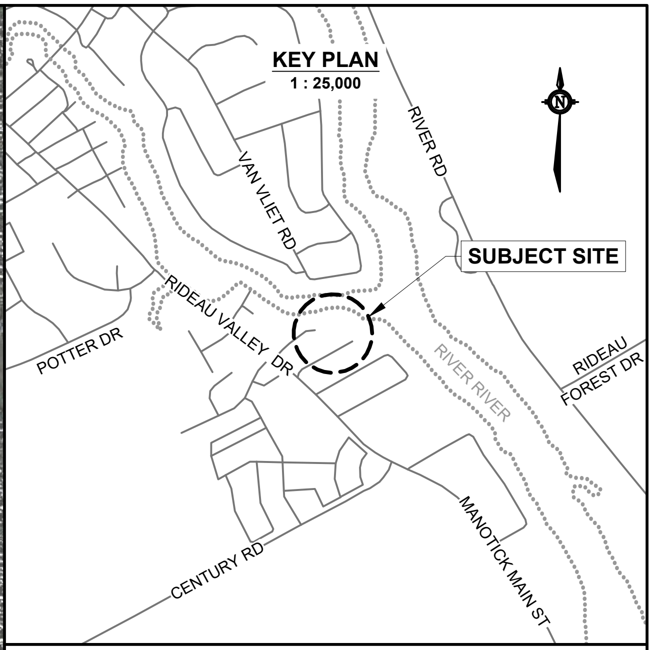
The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the individual houses, site services and roadways 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. In accordance with Ontario Building Code requirements, full time compaction testing is required for engineered fill below buildings.

8.0 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.

Alex Meacoe, P.Eng.
Geotechnical Engineer

Brent Wiebe, P.Eng.
VP Operations - Ontario



LEGEND

BH/ TP # — BOREHOLE/ TEST PIT ID

XX.XX — GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

BOREHOLE LOCATION (current investigation by GEMTEC)

TEST PIT LOCATION (current investigation by GEMTEC)

A A' SLOPE CROSS SECTION LOCATION IN PLAN (current investigation by GEMTEC)

APPROXIMATE PROPERTY BOUNDARY

Scale

1:1000

0 20 40 60m

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

32 Steacie Drive
Ottawa, ON K2K 2A9
Tel: (613) 836-1422
www.gemtec.ca
ottawa@gemtec.ca

Drawing	SITE PLAN		
Client	1910753 ONTARIO INC.		
Project 100152.004	PERMIT TO TAKE WATER PROPOSED RESIDENTIAL DEVELOPMENT ANTOCHI LANE OTTAWA, ONTARIO		
Drwn by S.L.	Chkd by W.A.M.		
Date DECEMBER, 2021	Rev. 0	FIGURE 1	

\\LUCID\DRAWINGS AND FILES\PROJECTS\100100\100152.004\06_DRAFTING\1.DRAWINGS\100152.004_PTTW_FG1-SP_R0_2021-12.DWG

APPENDIX A

Record of Boreholes and Test Pit Sheets
List of Abbreviations and Symbols
Boreholes 21-01, 21-02 and Test Pit 21-03

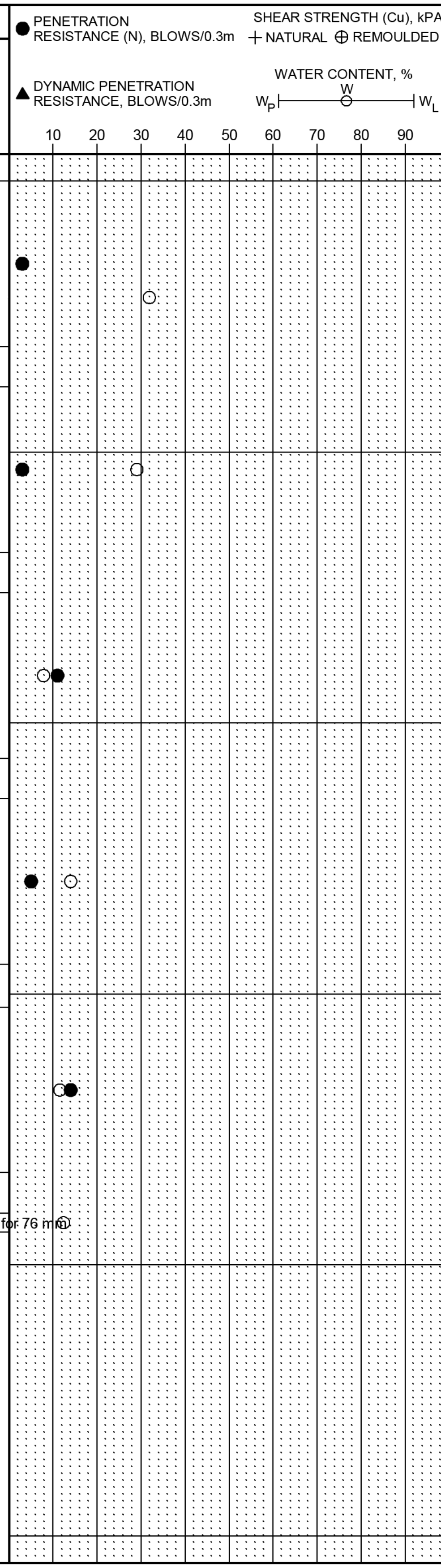
RECORD OF BOREHOLE 21-01

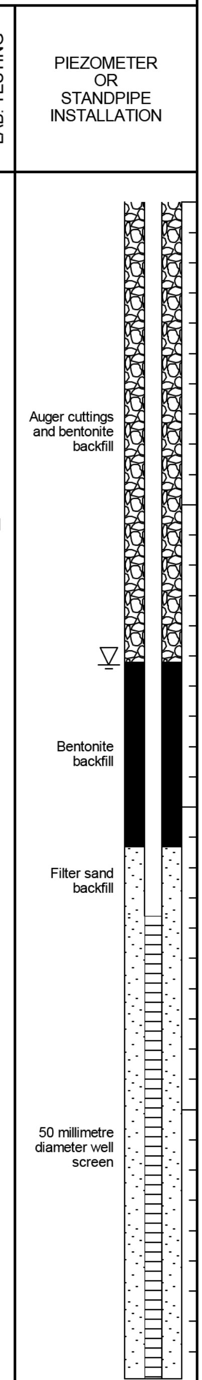
CLIENT: 1910753 Ontario Inc.
PROJECT: Proposed Residential Development, Antochi Lane, Ottawa, Ontario
JOB#: 100152.004
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Nov 18 2021

BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
0	Ground Surface	86.81					
	TOPSOIL						
	Very loose, grey brown SILTY SAND, with grey brown silty clay layers	86.56 0.25	1	SS	255	3	
1			2	SS	510	3	MH
	Loose to compact, grey brown SILTY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)	85.49 1.32	3	SS	230	11	
2			4	SS	510	5	
3			5	SS	430	14	
			6	SS	50	>50 for 76 mm	
4	End of Borehole Auger Refusal	82.92 3.89					
5							





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
21/12/03	1.5 ∇	85.3

GEMTEC Consulting Engineers and Scientists

LOGGED: AN
CHECKED: WAM

RECORD OF BOREHOLE 21-02

CLIENT: 1910753 Ontario Inc.
PROJECT: Proposed Residential Development, Antochi Lane, Ottawa, Ontario
JOB#: 100152.004
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Nov 18 2021

BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	86.56					
ASPHALTIC CONCRETE	86.46 / 0.13					
Grey sand and gravel (BASE MATERIAL)						
ASPHALTIC CONCRETE	0.18					
Grey sand and gravel (BASE MATERIAL)	86.00 / 0.56	1	GS			
Stiff, dark grey SILTY CLAY	85.57 / 0.99					
Loose to compact, grey brown SILTY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)		2	SS	255	16	
		3	SS	50	10	
		4	SS	405	4	MH
	82.78 / 3.78	5	SS	430	16	
End of Borehole Auger Refusal						

DEPTH SCALE METRES: 0, 1, 2, 3, 4, 5

PENETRATION RESISTANCE (N), BLOWS/0.3m
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA: + NATURAL ⊕ REMOULDED
WATER CONTENT, %: W_P — W — W_L
10 20 30 40 50 60 70 80 90

PIEZOMETER OR STANDPIPE INSTALLATION

Auger cuttings and bentonite backfill
Bentonite backfill
Filter sand backfill
50 millimetre diametre well screen

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
21/12/03	0.8 ∇	85.7

RECORD OF TEST PIT 21-03

CLIENT: 1910753 Ontario Inc.
PROJECT: Proposed Residential Development, Antochi Lane, Ottawa, Ontario
JOB#: 100152.004
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Nov 18 2021

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SAMPLE TYPE	SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED / WATER CONTENT, % W_P W W_L (10 20 30 40 50 60 70 80 90)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		86.7					
	TOPSOIL		86.3 0.4					
	Grey brown SILTY SAND		85.9 0.8					
1	Grey brown SILTY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)		85.5 1.1					
	End of Test Pit Refusal on cobbles and boulders							
2								
3								
4								
5								

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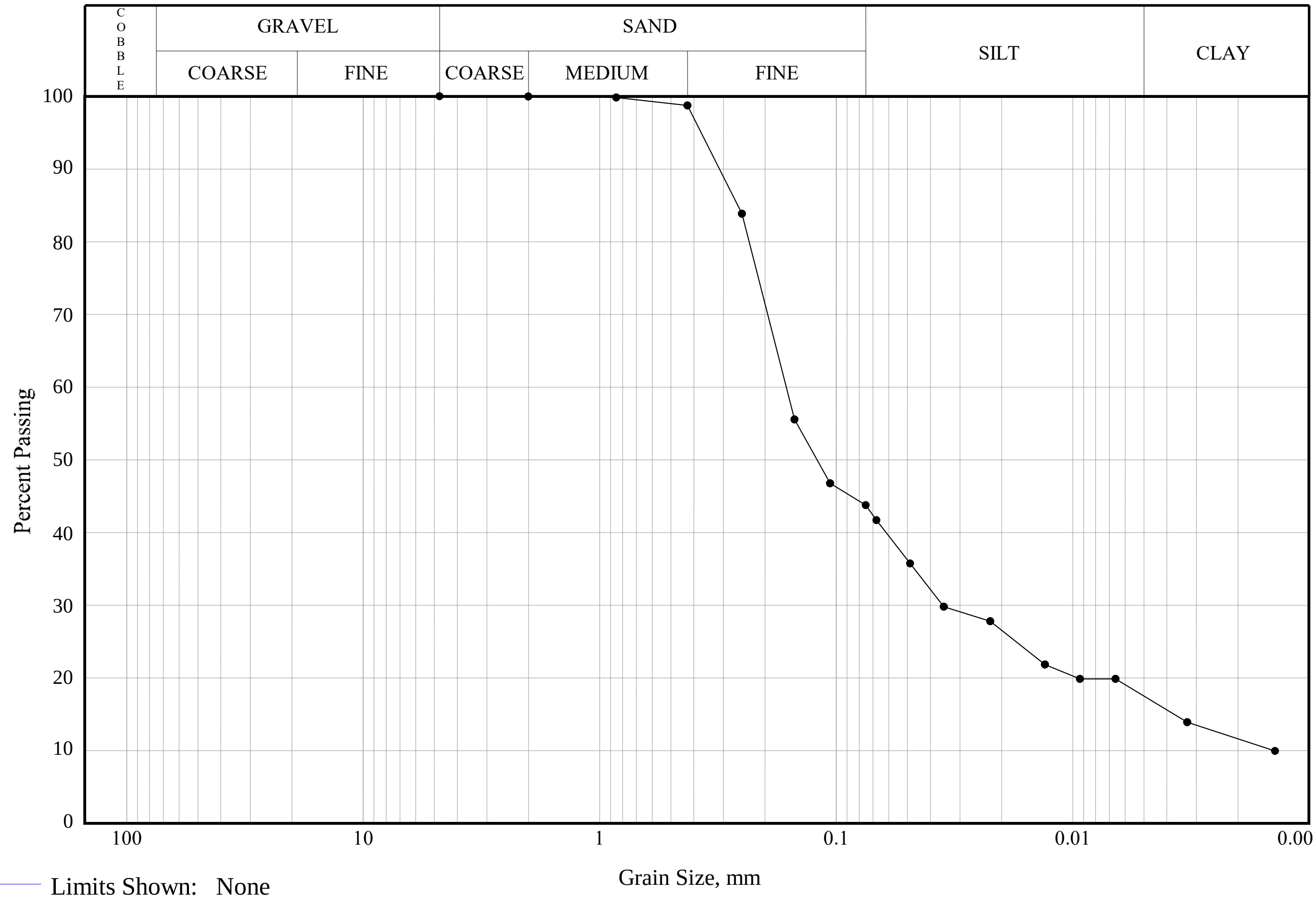
LOGGED: AN
CHECKED: WAM

APPENDIX B

Laboratory Test Results
Grain Size Distribution Testing

GEMTEC Consulting Engineers and Scientists	Client: Cavanagh Construction (Developments)	Soils Grading Chart (T88)
	Project: Geotechnical and Hydrogeological Investigation, Propose	
	Project #: 100152004	

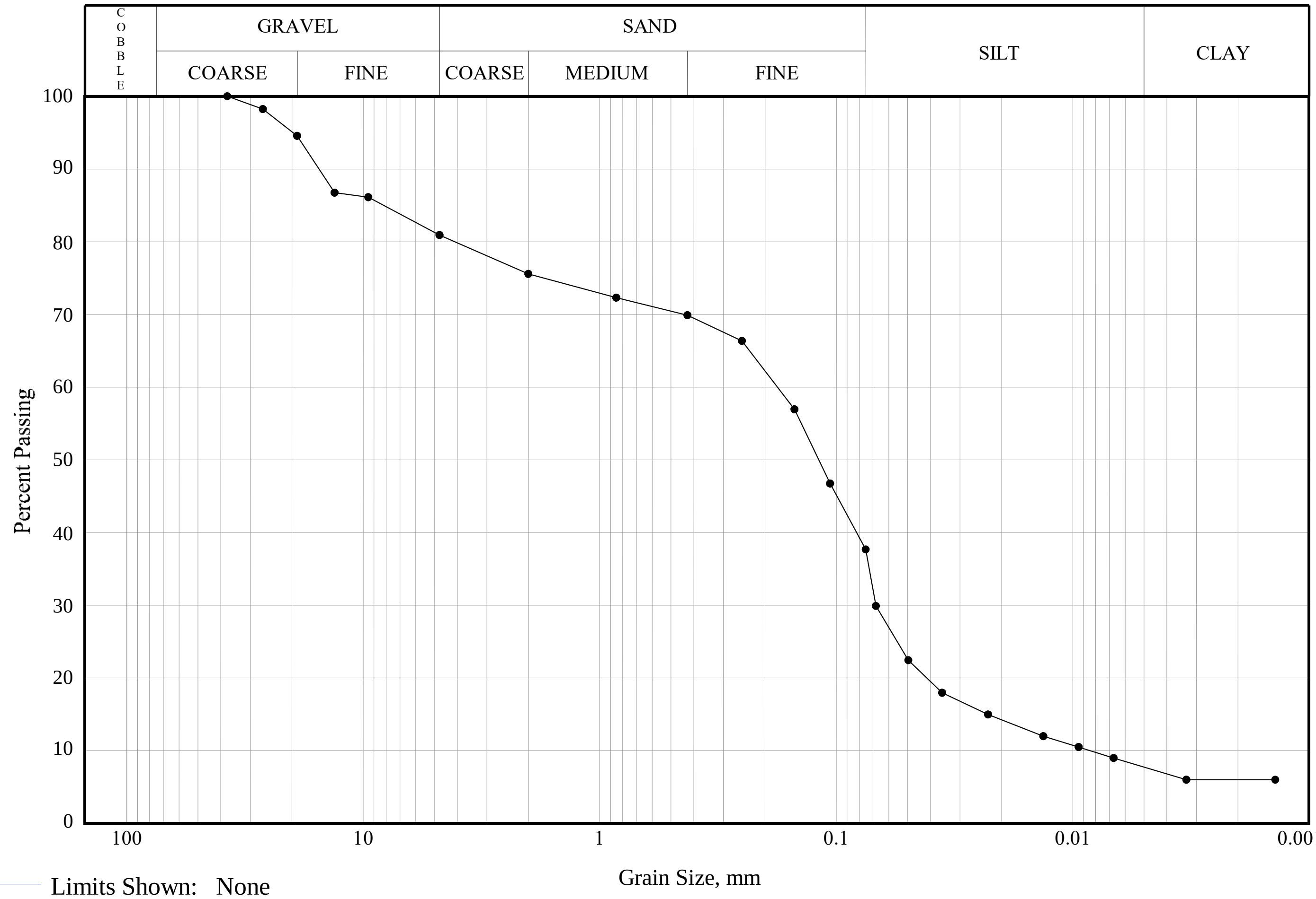


Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	SILTY SAND	21-01	SA 2	0.76-1.37	0.0	56.3	26.2	17.5

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—	Silty sand , some clay	N/A	0.00	0.00	0.04	0.12	0.16	0.26	26.2

GEMTEC Consulting Engineers and Scientists	Client: Cavanagh Construction (Developments)	Soils Grading Chart (T88)
	Project: Geotechnical and Hydrogeological Investigation, Propose	
	Project #: 100152004	



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	GLACIAL TILL	21-02	SA 4	2.28-2.89	19.1	43.2	29.9	7.7

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—	Silty sand , some gravel , trace clay	N/A	0.01	0.02	0.07	0.12	0.18	8.19	29.9

APPENDIX C

Chemical Analysis of Soil Sample
Sample Relating to Corrosion
(Paracel Laboratories Ltd. Order No. 2148330)

Certificate of Analysis

Client: **GEMTEC Consulting Engineers and Scientists Limited**

Client PO:

Report Date: 01-Dec-2021

Order Date: 23-Nov-2021

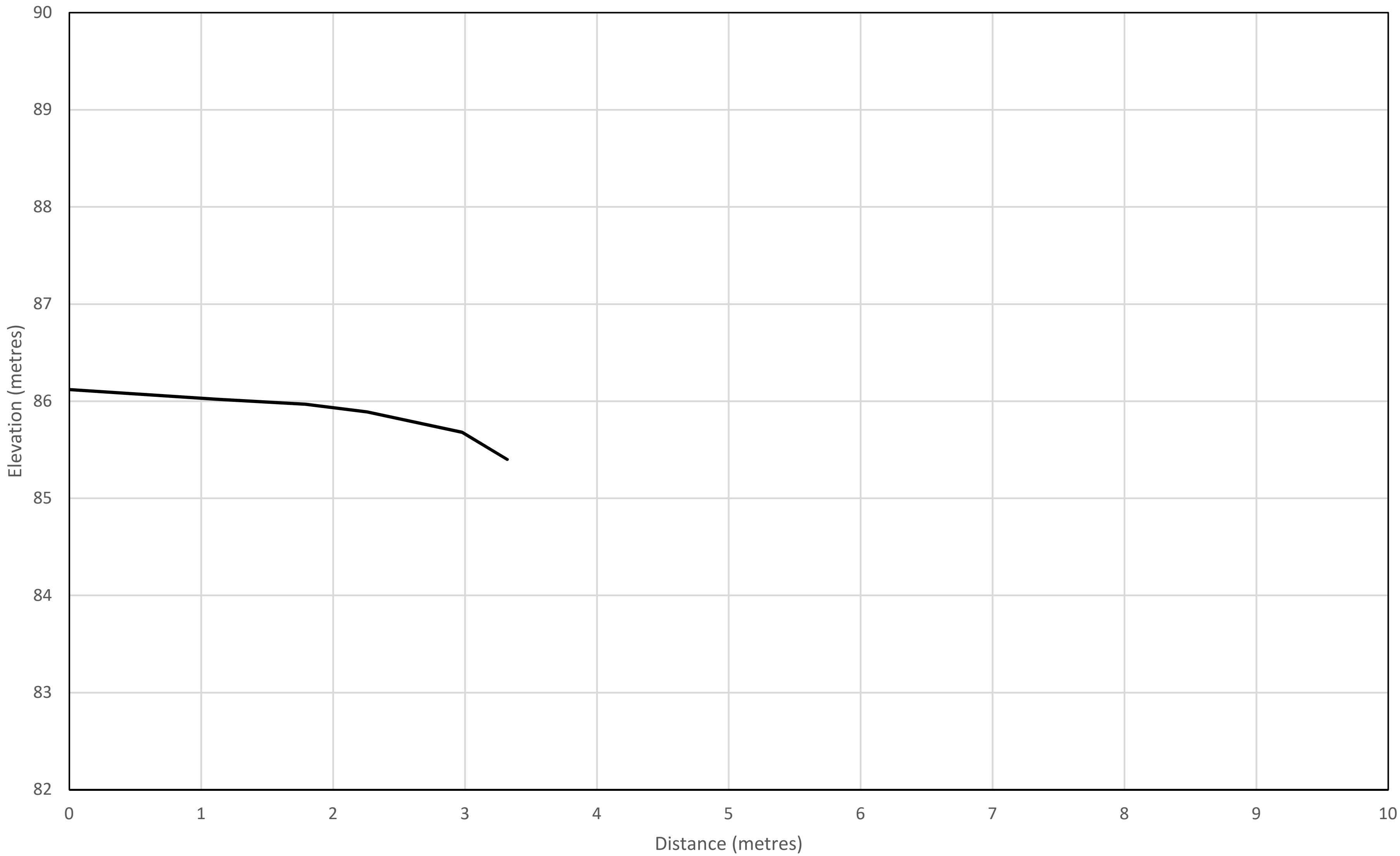
Project Description: 100152.004

	Client ID:	21-01 SA-3	-	-	-
	Sample Date:	23-Nov-21 15:00	-	-	-
	Sample ID:	2148330-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.0	-	-	-
General Inorganics					
Conductivity	5 uS/cm	157	-	-	-
pH	0.05 pH Units	7.86	-	-	-
Resistivity	0.10 Ohm.m	63.9	-	-	-
Anions					
Chloride	5 ug/g dry	27	-	-	-
Sulphate	5 ug/g dry	23	-	-	-

APPENDIX D

Results of Slope Stability Analysis
Figure D1 to D5

Report to: 1910753 Ontario Inc.
Project: 100152.004 (February 7, 2023)



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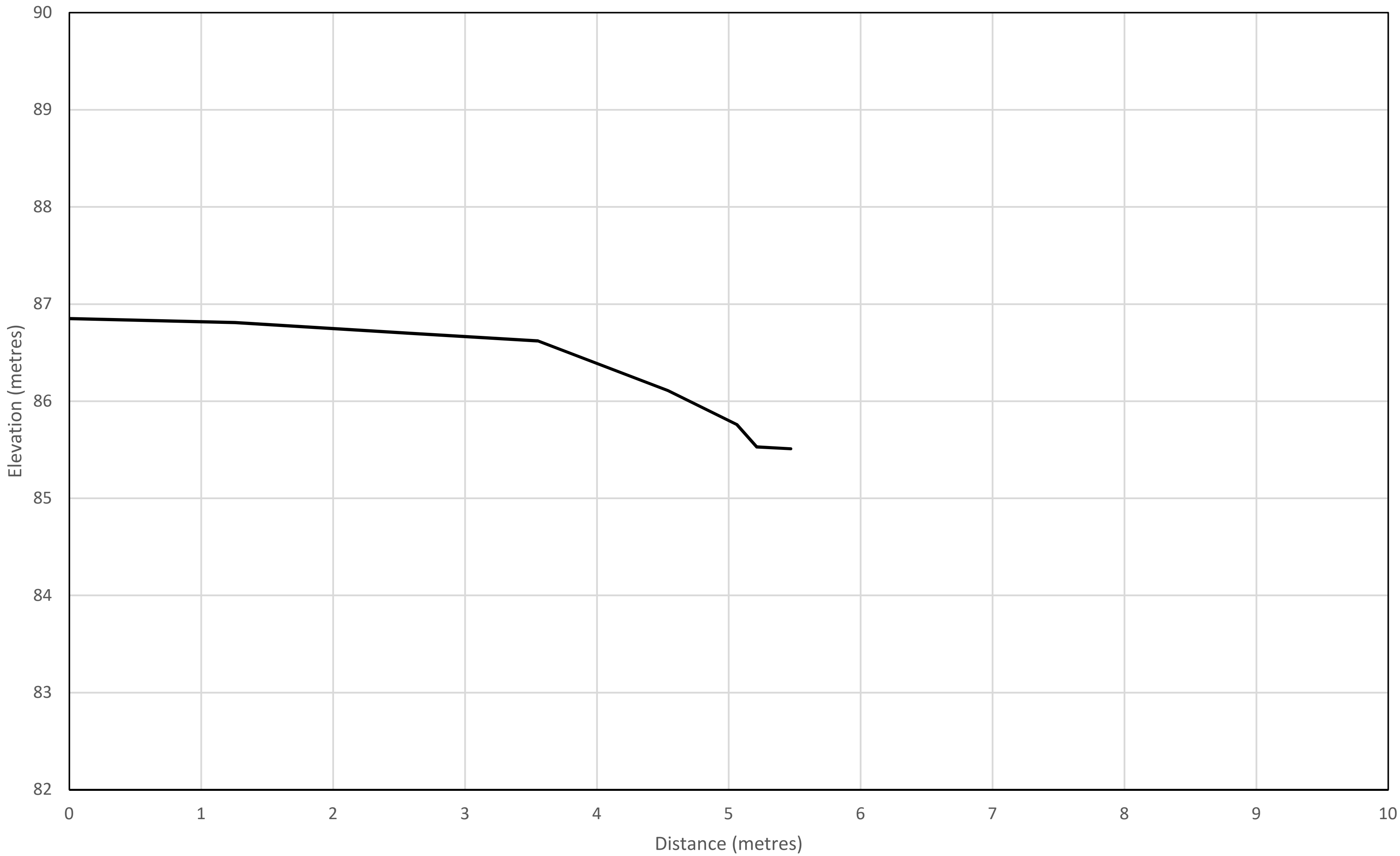
Slope Cross Section A-A

1086 Antochi Lane

Ottawa, Ontario

Project No.	100152.004
Drawn:	WAM
Date:	10/12/2021

Figure D1



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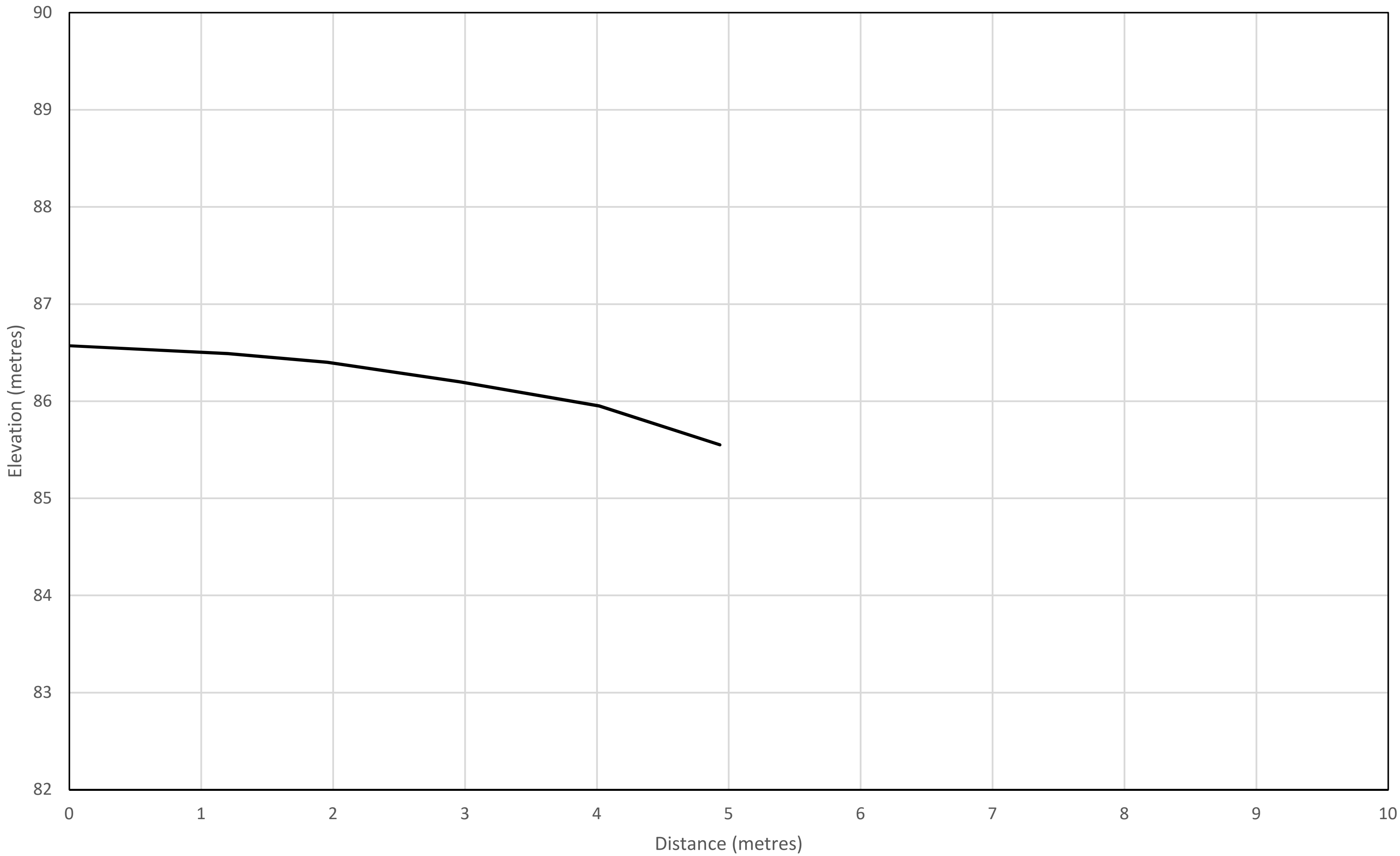
Slope Cross Section B-B

1086 Antochi Lane

Ottawa, Ontario

Project No.	100152.004
Drawn:	WAM
Date:	10/12/2021

Figure D2



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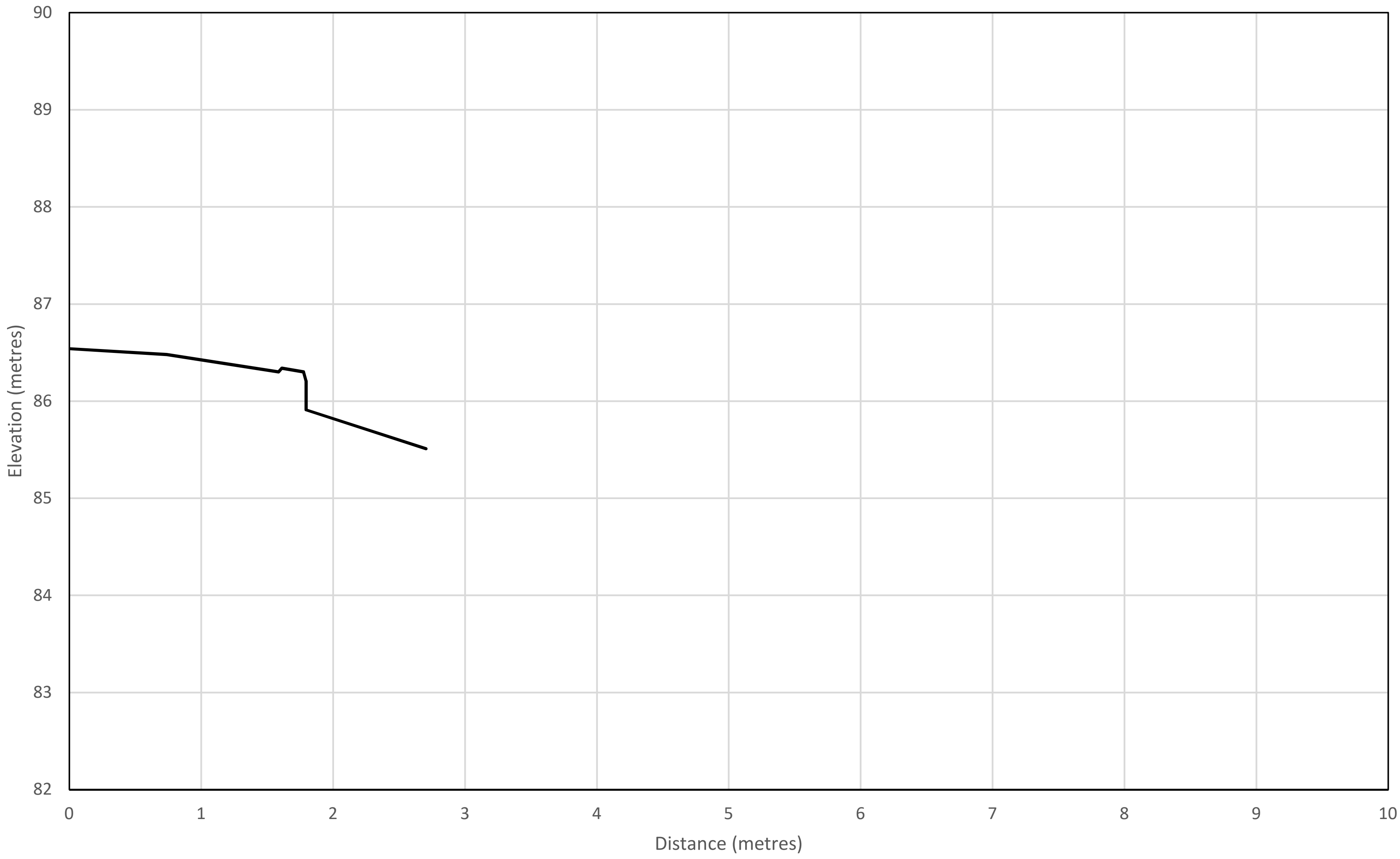
Slope Cross Section C-C

1086 Antochi Lane

Ottawa, Ontario

Project No.	100152.004
Drawn:	WAM
Date:	10/12/2021

Figure D3



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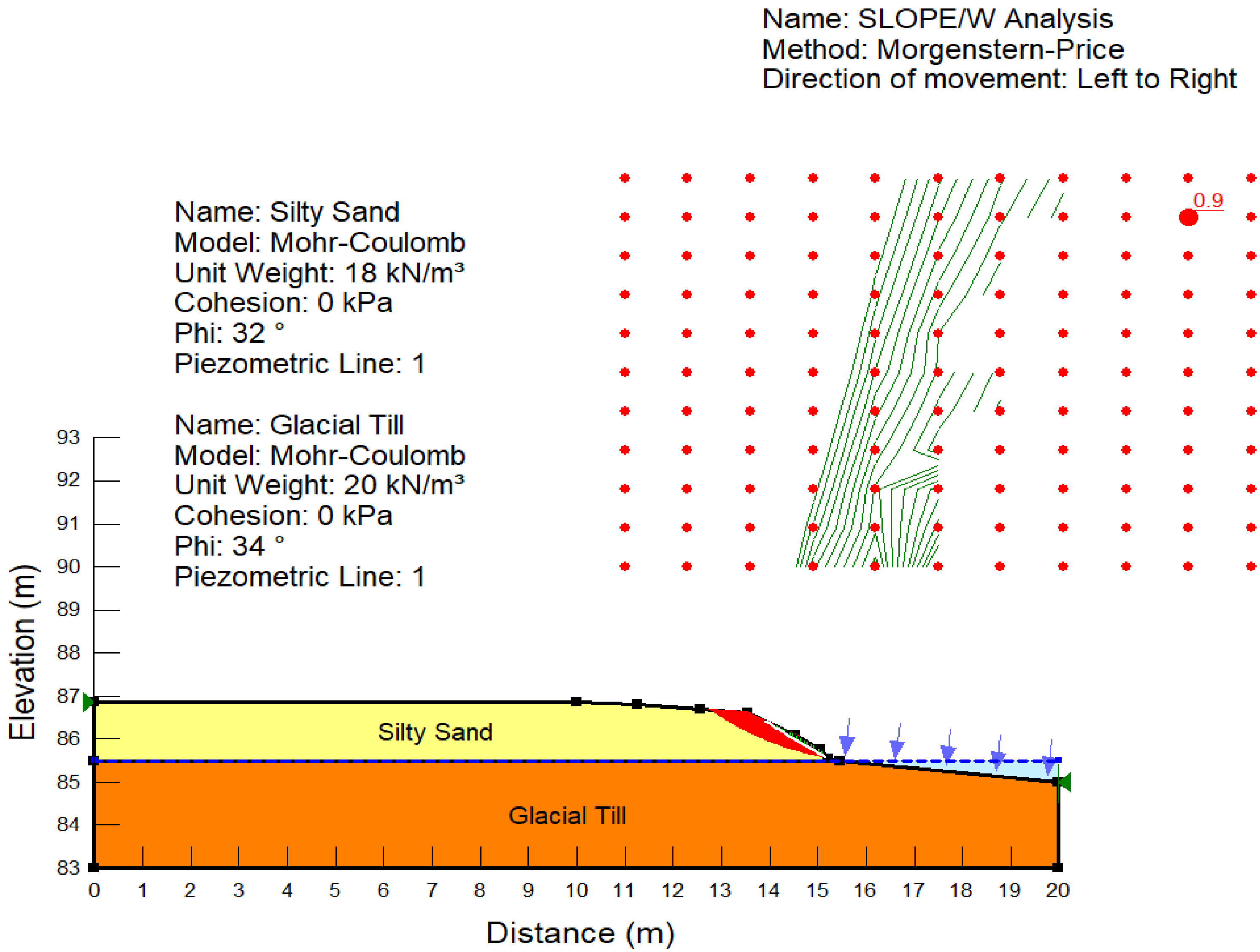
Slope Cross Section D-D

1086 Antochi Lane

Ottawa, Ontario

Project No.	100152.004
Drawn:	WAM
Date:	10/12/2021

Figure D4



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Consulting Engineers and Scientists

Slope Cross Section B-B

1086 Antochi Lane

Ottawa, Ontario

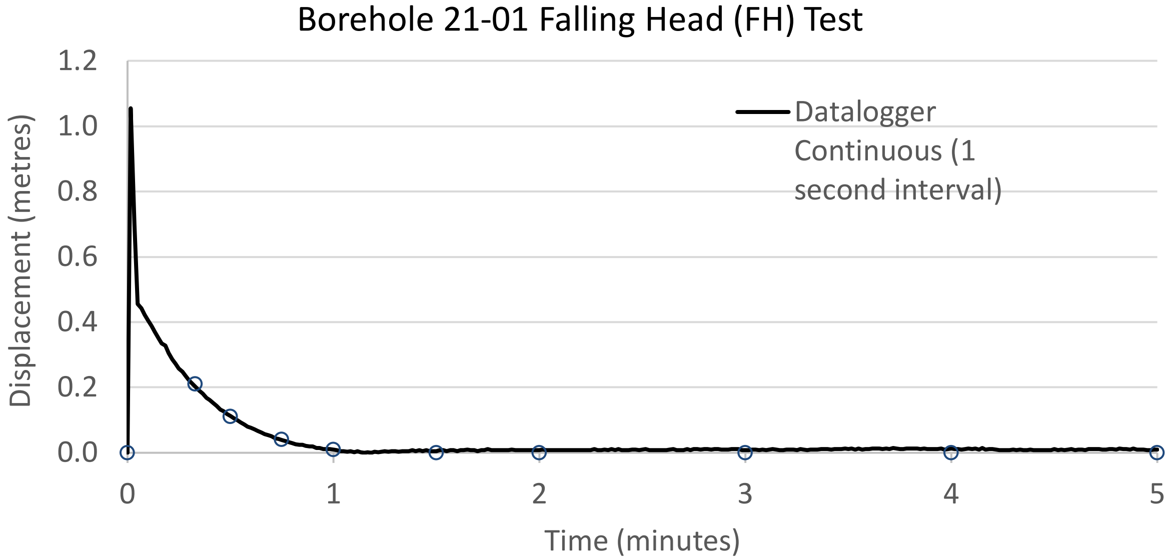
Project No.	100152.004
Drawn:	WAM
Date:	10/12/2021

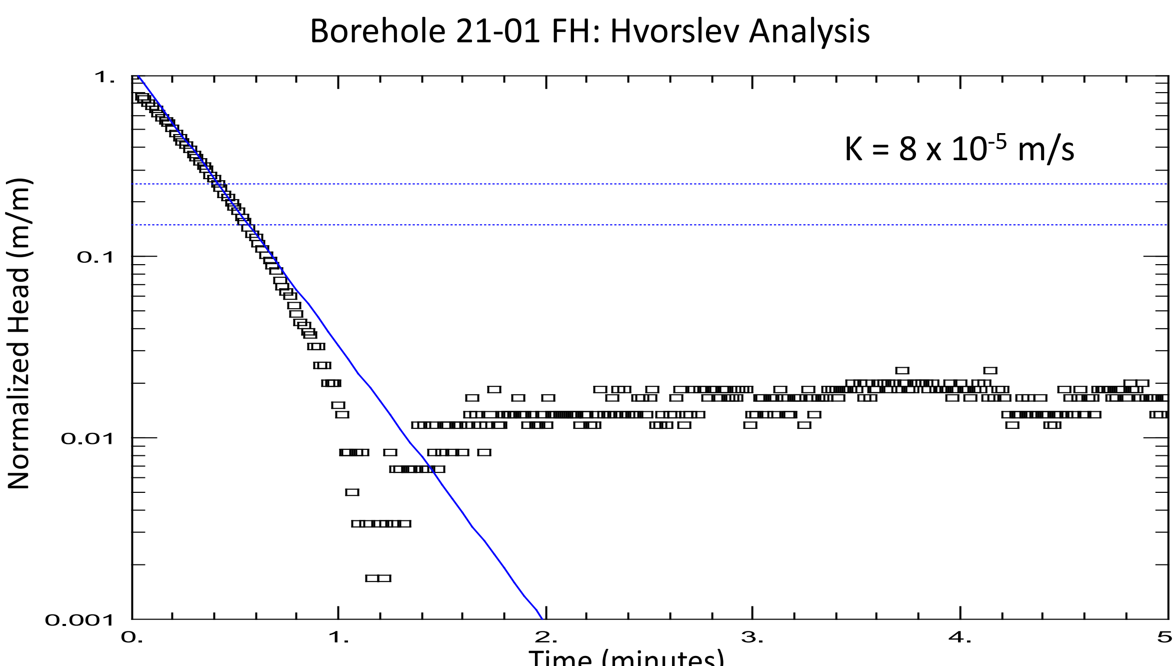
Figure D5

APPENDIX E

Hydraulic Conductivity Testing
Figure E1 to E4

Slug Test Data	FIGURE E1





<u>Well Data:</u>

Displacement observed (slug size): 1.05 metres (0.60 m)

Well Depth: 3.83 metres

Screen Length: 1.5 metres

Well Radius: 0.0255 metres

<u>Aquifer Data</u>

Saturated Thickness: 2.3 metres

Anisotropy Ratio (Kz/Kr): 0.1

Aquifer Model: Unconfined, Hvorslev

Static Water Level: 1.53 metres bgs

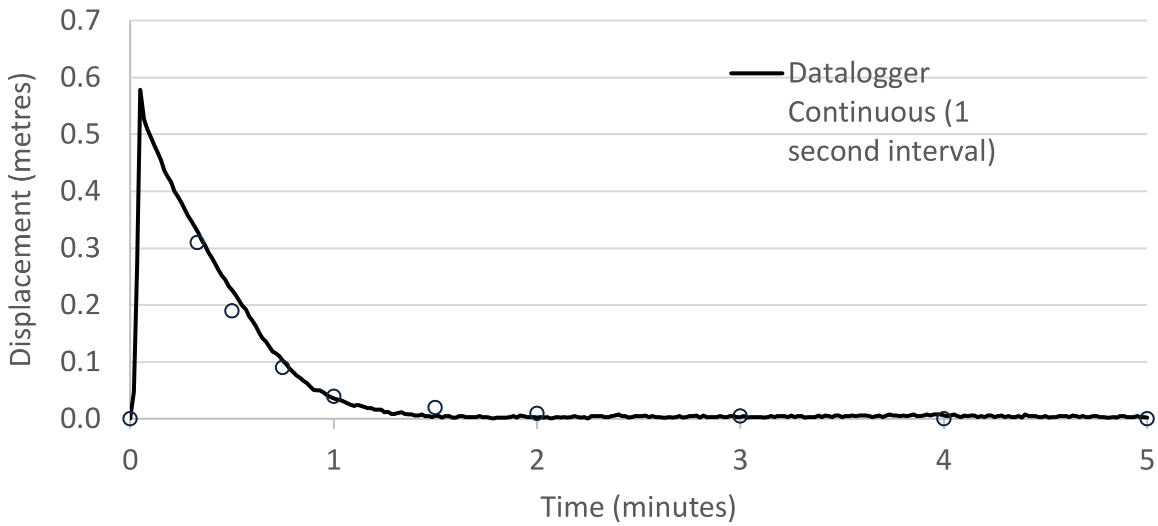
GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

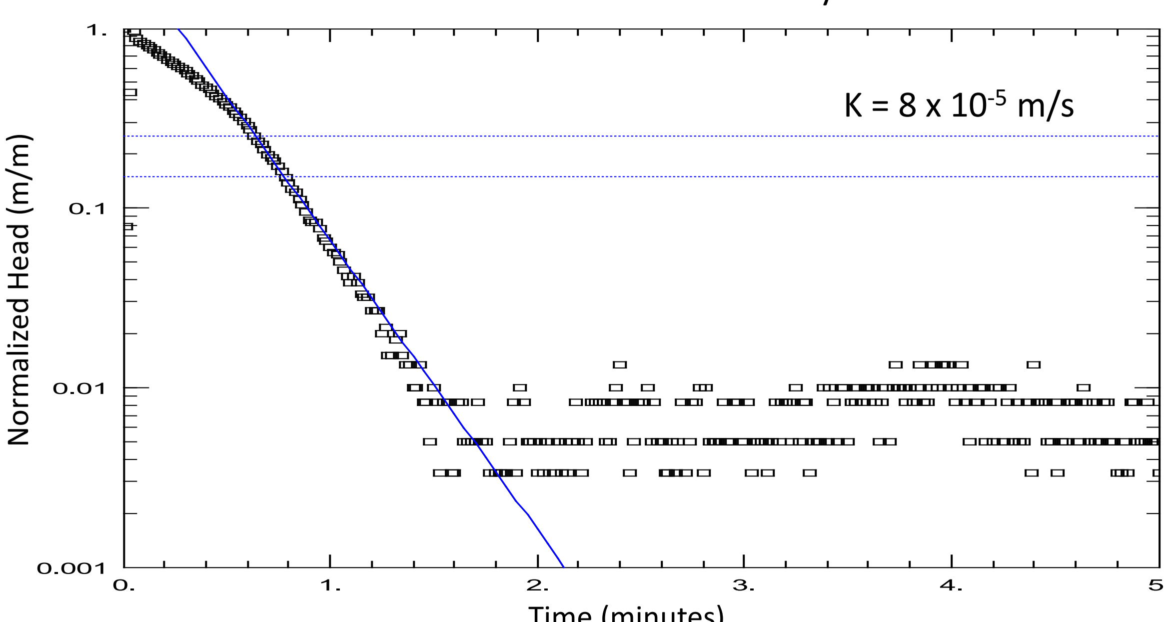
Date: December 2021

Project: 100152.004

Slug Test Data

FIGURE E2





Well Data:

Displacement observed (slug size): 0.58 metres (0.60 m)

Well Depth: 3.83 metres

Screen Length: 1.5 metres

Well Radius: 0.0255 metres

Aquifer Data

Saturated Thickness: 2.3 metres

Anisotropy Ratio (Kz/Kr): 0.1

Aquifer Model: Unconfined, Hvorslev

Static Water Level: 1.53 metres bgs

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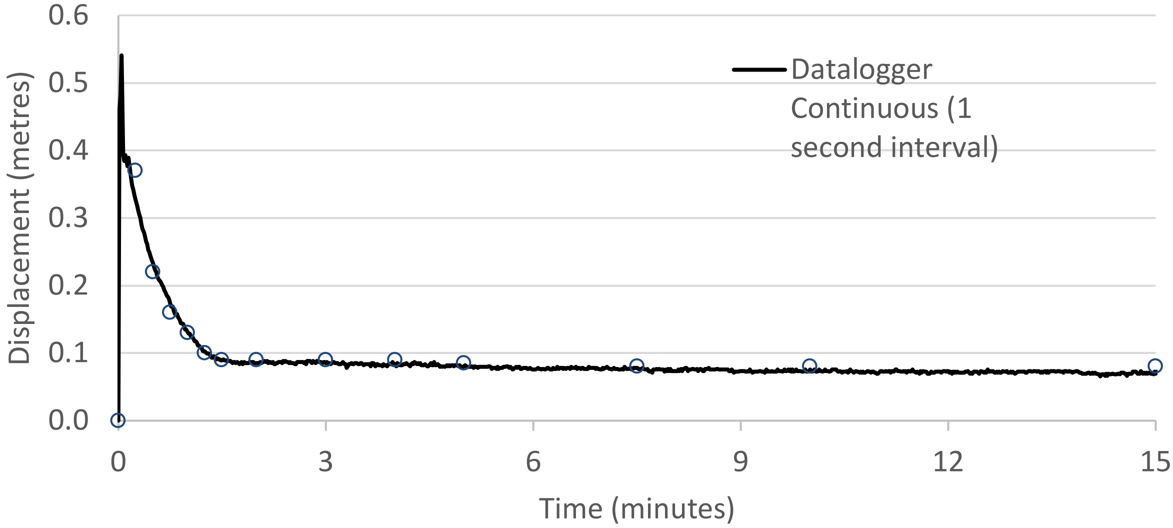
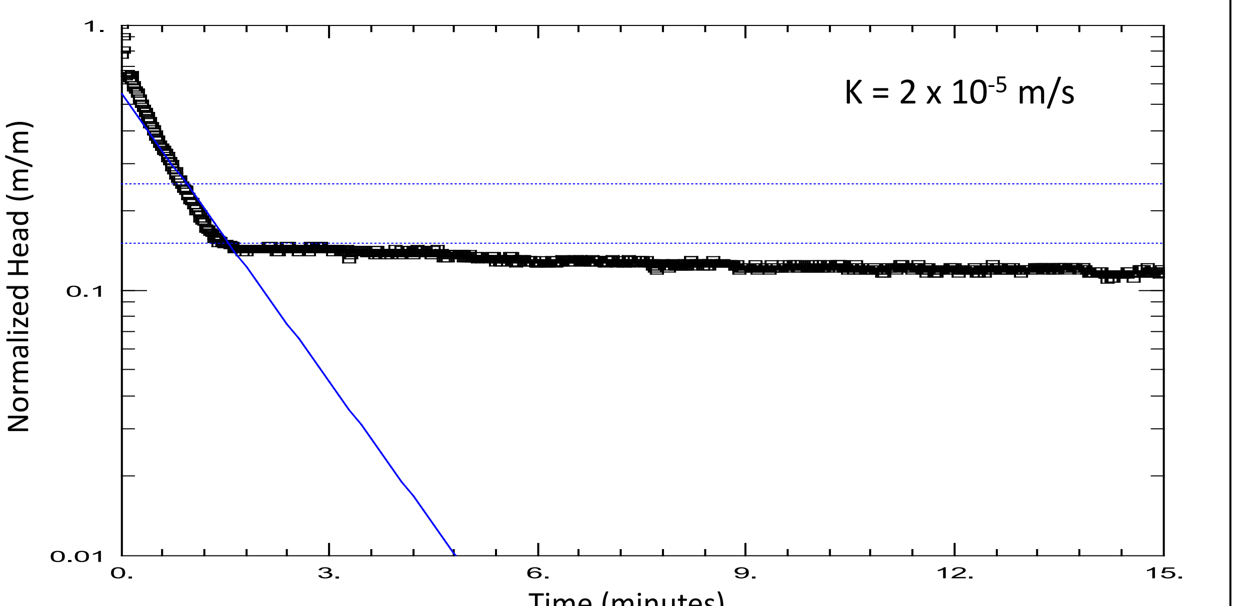
Slug Test Data

FIGURE E3

Borehole 21-02 Falling Head (FH) Test



Borehole 21-02 FH: Hvorslev Analysis



<u>Well Data:</u>
Displacement observed (slug size): 0.54 metres (0.60 m)
Well Depth: 3.70 metres
Screen Length: 1.5 metres
Well Radius: 0.0255 metres

<u>Aquifer Data</u>
Saturated Thickness: 2.9 metres
Anisotropy Ratio (Kz/Kr): 0.1
Aquifer Model: Unconfined, Hvorslev
Static Water Level: 0.83 metres bgs

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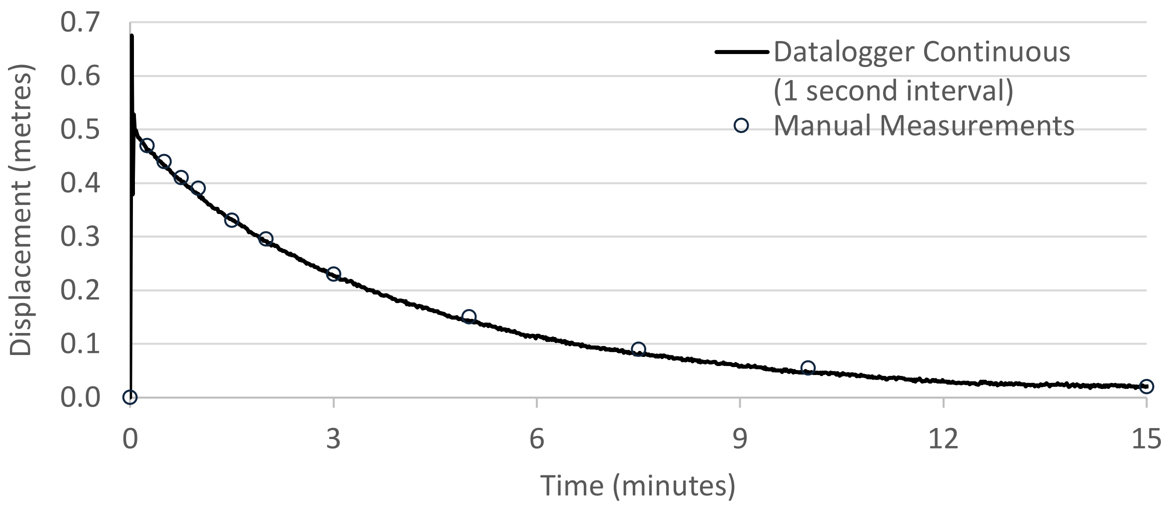
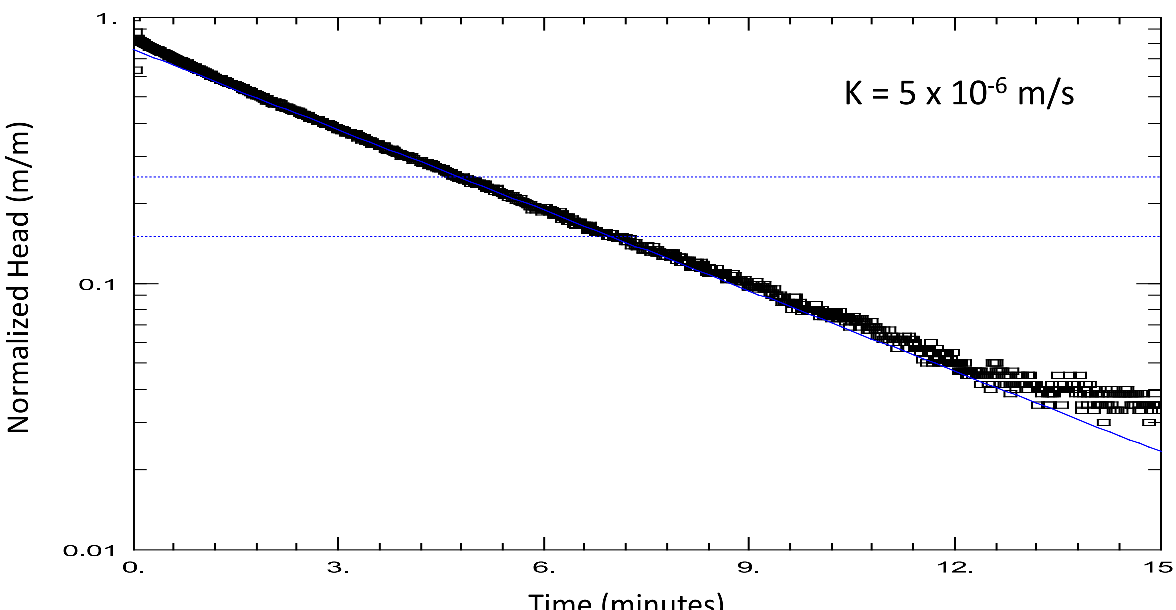
Slug Test Data

FIGURE E4

Borehole 21-02 Rising Head (RH) Test



Borehole 21-02 RH: Hvorslev Analysis



<u>Well Data:</u>
Displacement observed (slug size): 0.68 metres (0.60 m)
Well Depth: 3.70 metres
Screen Length: 1.5 metres
Well Radius: 0.0255 metres

<u>Aquifer Data</u>
Saturated Thickness: 2.9 metres
Anisotropy Ratio (Kz/Kr): 0.1
Aquifer Model: Unconfined, Hvorslev
Static Water Level: 0.83 metres bgs

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experience • knowledge • integrity

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geotechnical	géotechnique
environmental	environnementale
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

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