GEMTEC

www.gemtec.ca

Geotechnical Investigation and Slope Stability Assessment
Proposed Carp Airport Phase 2 Commercial Development
Part of Lots 13, 14 and 15. Concession 3

Ottawa, Ontario

experience • knowledge • integrity   expérience • connaissance • intégrité

GEMTEC
www.gemtec.ca

Submitted to:

Novatech
240 Michael Cowpland Drive, Suite 200
Ottawa, Ontario
K2M 1P6

Geotechnical Investigation and Slope Stability Assessment Proposed Carp Airport Phase 2 Commercial Development Part of Lots 13, 14 and 15. Concession 3

Ottawa, Ontario

November 4, 2025
GEMTEC Project: 100011.049

experience • knowledge • integrity

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the Carp Airport Phase 2 commercial development located at Part of Lots 13, 14, and 15, Concession 3 in 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, 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.

The work program was completed in accordance with GEMTEC Consulting Engineers and Scientists (GEMTEC) Proposal No. P100011.049 dated February 13, 2023.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Site Description

Plans are being prepared for a new commercial development to be located at the Carp Airport Phase 2 Business Park, located on Part of Lots 13, 14 and 15, Concession 3, in the City of Ottawa, Ontario. This location is referred to herein as the Site.

Based on a review of recent aerial photographs, the Site is generally vacant land, which was previously (and possibly currently) used for agricultural purposes. North of the Site a mixture of residential and commercial/industrial building have been constructed. South of the Site is the Carp Airport and associated structures.

A drainage channel referred to as the Northeast Tributary flows to the north across the site, in addition to a series of other smaller channels and ditches. The site is generally level and appears to grade from south to north.

2.2 Anticipated Site Geology

Based on previous geotechnical investigation in the area and surficial geology maps, soils at the Site are likely composed of thick deposits of silt and clay with minor sand. Bedrock geology maps indicate that the soils are underlain by limestone and shale bedrock of the Verulam Formation. Bedrock mapping indicates the bedrock surface is expected at depths ranging from about 10 to 50 metres sloping down towards the southeast. Aerial photographs of the site suggest that fill material associated with previous development in the area is present across portion of the Site.

2.3 Proposed Development

The proposed development within the Site will include 21 commercial blocks. It is understood that the blocks will be sold individually and developed by the purchaser according to their needs. As such, details of the proposed buildings are not available at this time. However, it is anticipated that one to two storey buildings will be constructed on the blocks. The buildings will be commercial 'slab on grade' type buildings (i.e., no basement level). Access to the blocks will be provided via a series of internal roadways (named Streets Fifteen to Eighteen), connecting the blocks to Thomas Argue Road or Russ Bradley Road.

The blocks will be serviced by watermains and sanitary sewers with diameters up to about 300 millimetres and invert depths ranging from about 5 metres below the existing ground surface. In addition to this, five storm water management ponds are proposed along the east side of the site, with two larger ponds along the Northeast Tributary and three smaller ponds along Carp Road. Storm drainage will be completed using open ditch drainage connecting to the stormwater management ponds, generally without crossing the Northeast Tributary.

Within Block 27 a utility crossing of the Northeast Tributary will be constructed. At this location trenchless construction techniques are being considered for the proposed watermain, sanitary sewer crossing, and other possible utilities. No further details on the proposed services are known at the time of writing this report.

A pump station will be constructed within the development. The location of the proposed pumping station is not known at this time; however, it is expected to be located near Blocks 23 and 24 with the depth of about 6 to 7 metres below the existing ground surface.

2.4 Summary of Information Provided

2.4.1 Paterson (2013) Geotechnical Investigation

A previous geotechnical investigation was carried out over a portion of the Site by Paterson Group. The results of which are provided in the following report:

- Report to West Capital Developments titled "Geotechnical Investigation, Carp Airport Servicing and Residential Development – Phase 1, Carp Road – (Carp) Ottawa" dated July 22, 2013 (Report No. PG2450-2)

This investigation and report is referred to herein as Paterson (2013).

As part of Paterson (2013) nine boreholes and six test pits were advanced within or adjacent to the site. The subsurface conditions encountered in the boreholes and test pits generally consists of about 0.5 to 1.5 metres of fill material or 0.2 to 0.6 metres of topsoil over deposits of silty sand to sandy silt or silty clay. The boreholes and test pits were terminated at depths ranging from about 2.9 to 4.9 metres below the existing ground surface. The groundwater levels were measured in the monitoring wells at depths ranging from about 2.5 to 2.7 metres below the existing ground surface.

2.4.2 Design Drawing, NOVATECH

The following drawings were provided to GEMTEC which were prepared by NOVATECH Engineers, Planners & Landscape Architects (NOVATECH):

- Drawing No. 102085-GP5 titled “Grading Plan” (Project No. 102085-14) Revision No. 1, dated August 29, 2024;
- Drawing No. 102085-2BP-GP6 titled “Grading Plan” (Project No. 102085-14) Revision No. 1, dated August 29, 2024;
- Drawing No. 102085-2BP-GP7 titled “Grading Plan” (Project No. 102085-14) Revision No. 1, dated August 29, 2024;
- Drawing No. 102085-2BP-GP8 titled “Grading Plan” (Project No. 102085-14) Revision No. 1, dated August 29, 2024; and,
- Drawing No. 102085-GP9 titled “Grading Plan” (Project No. 102085-14) Revision No. 15, dated August 29, 2024.
- Drawing Nos. 102085-P60 to 102085-64 titled “Plan & Profile” (Project No. 102085-14) Revision No. 8, dated August 22, 2024.
- Drawing Nos. 102085-P69 to 102085-74 and 102085-76 titled “Plan & Profile” (Project No. 102085-14) Revision No. 2, dated August 22, 2024.

3.0 METHODOLOGY

The fieldwork for this investigation was carried out between April 3 and 14, 2023. During that time, a total of 14 boreholes (numbered 23-01 to 23-14, inclusive) and three cone penetration tests (numbered CPT 23-03, CPT 23-13, and CPT 23-14) were advanced at the Site using a track mounted hollow stem auger drill rig supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Quebec.

Details for the boreholes advanced for the commercial development are provided below:

- Boreholes 23-01, 23-06, 23-08, 23-13, and 23-14 were advanced to a depth of about 15.1 metres below the existing ground surface.
- Boreholes 23-02, 23-03, 23-04, 23-07, 23-10, 23-11, and 23-12 were advanced to depths ranging from about 8.2 and 9.0 metres below the existing ground surface.
- Boreholes 23-05 and 23-09 were advanced to depths of about 5.2 and 5.9 metres below the existing ground surface, respectively.
- Cone penetration tests (CPT 23-03, CPT 23-13, and CPT 23-14) were advanced adjacent to boreholes 23-03, 23-13, and 23-14, respectively, to depths ranging from about 10.9 to 30.7 metres below the existing ground surface.

The approximate locations of the boreholes from the current and previous investigation are shown on the Site Plan, Figure 1.

Standard penetration tests were carried out in the boreholes and samples of the soils encountered were recovered using a 50-millimetre diameter split spoon sampler. In situ vane shear testing was carried out, where possible, in the boreholes to measure the undrained shear strength of the silty clay. Relatively undisturbed samples of the silty clay deposit were obtained from boreholes 23-02, 23-04 and 23-11.

A single well screens was installed in the overburden in each of boreholes 23-02, 23-05, 23-06, 23-08, 23-09, 23-12 and 23-14. The well screens were installed to measure the groundwater levels and for hydraulic conductivity testing.

Hydraulic testing was carried out to estimate the hydraulic conductivity of the overburden soils within the assumed depth of excavations and to provide an estimate of the potential quantity of water entering future excavations. A rising head test was performed in each on-site well by purging the well to a known depth and allowing it to recover. Water level recovery within the wells were monitored using a data-logging pressure transducer and an electric water level tape. Analysis of the data was performed under the assumption that the introduced change in head within the well was near instantaneous.

The fieldwork was supervised throughout by a member of our engineering staff who directed the drilling operations, logged the samples and carried out the in-situ testing. Following the fieldwork, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of soil were tested for water content testing, Atterberg Limit testing, and grain size distribution testing. Oedometer consolidation testing was carried out on one sample of the silty clay from borehole 23-11. In addition, two samples of the soil, one each from boreholes 21-03 and 21-08, were sent to an accredited laboratory for basic chemical testing relating to corrosion of buried concrete and steel.

The borehole and CPT probe locations were positioned in the field and subsequently surveyed by GEMTEC personnel using high precision GPS surveying equipment. The elevations are referenced to geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The results of the boreholes are provided on the Record of Borehole sheets in Appendix A. The results of the CPT probes are provided in Appendix B. The results of the laboratory classification tests on the soil samples are provided on the borehole logs and in Appendix C. The borehole logs from the previous investigation by Paterson are provided in Appendix D. The results of the laboratory testing related to corrosion of buried elements are provided in Appendix E. The results of the hydraulic conductivity testing are provided in Appendix F.

The following presents an overview of the subsurface conditions encountered in the boreholes advanced by GEMTEC during this investigation.

4.2 Topsoil

Topsoil was encountered at surface at all borehole locations and ranges in thickness from about 80 to 250 millimetres.

4.3 Silt and Sand

Native deposits of silty sand to sandy silt, with varying amounts of clay were encountered in the boreholes 23-04, 23-05, and 23-07 to 23-14 (herein referred to as “silt and sand”). The silt and sand deposits were encountered at varying depths, being present below the topsoil at some locations, and lower in the soil profile at others.

The thickness of the silt and sand layers at the various depths where it was encountered, ranges from about 0.5 to 4.9 metres. The silt and sand deposits generally extend to depths ranging from about 0.9 to 5.8 metres below the existing ground surface, but may be encountered at other depths.

Standard penetration tests carried out in the silt and sand gave SPT N values of 1 to 16 blows per 0.3 metres of penetration, which reflects a very loose to compact relative density.

Grain size distribution testing was carried out on three finer grained samples of the silt and sand layer. The results are provided in Appendix C and summarized in Table 4.1 below. The measured water content of eight samples of the silt and sand deposit ranges from about 16 to 30 percent. Based on the results of the laboratory testing, the silt and sand deposits can be classified as SM and ML under the United Soil Classification System (USCS).

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

Borehole ID	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
23-09	1B	0.3 to 0.6	0	27	58	14
23-11	2	0.8 to 1.4	0	28	55	16
23-14	2	0.8 to 1.4	0	32	51	16

4.4 Silty Clay to Clayey Silt

Native deposits of silty clay to clayey silt, with varying amounts of sand were encountered in the boreholes.

The silty clay was generally encountered below the topsoil and silt and sand deposits (discussed in Section 4.3, above), where encountered. The full depth of the silty clay was not fully penetrated, but was proven to depths of up to about 15.1 metres below the existing ground surface. Based on the results of the CPT probes, probable silty clay deposits extend to depths up to about 30.7 metres below the existing ground surface.

The upper portion of the silty clay to clayey silt has been weathered to a grey brown crust, within the exception of that at borehole 23-14. The thickness of the weathered crust is variable, noting that at some locations silt and sand layers (as described in Section 4.3) are present within the weathered crust. The weathered crust extends to depths ranging from about 1.5 to 3.5 metres below the existing ground surface.

Standard penetration test carried out in the weathered silty clay crust ranges from “weight of hammer” to 8 blows per 0.3 metres. In situ vane shear strength tests carried out in the weathered crust gave undrained shear strengths ranging from about 87 to greater than 100 kilopascals. The results of the in situ indicates a stiff to very stiff consistency.

Grain size distribution testing was carried out on one sample of the weathered crust. The results are provided in Appendix C and summarized in Table 4.2 below.

Table 4.2 – Summary of Grain Size Distribution Test (Weathered Crust)

Borehole Number	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
23-01	2	0.8 to 1.4	0	11	57	31
23-07	2	0.8 to 1.4	0	24	55	21

Atterberg limit testing was carried out on three samples of the weathered silty clay crust. The results are provided in Appendix C and are summarized in Table 4.3. The measured water content of eight samples of the weathered silty clay ranges from about 23 to 33 percent.

Table 4.3 – Summary of Atterberg Limit Test (Weathered Crust)

Borehole / Sample No.	Water Content (%)	Liquid Limits (%)	Plastic Limits (%)	Plasticity Index
23-06 / 3	29		Non-plastic	
23-08 / 4	27	22	17	5
23-13 / 3	25	22	16	5

The testing indicates that the samples of the weathered silty clay to clayey silt from the boreholes generally have a low plasticity to non-plastic. Based on the results of the laboratory testing, the weathered crust deposits can be classified as CL and ML under the USCS.

The silty clay below the depth of weathering in boreholes 23-01 to 23-12 and the full profile of the silty clay in borehole 23-14 is generally grey brown to grey in colour, indicative of lesser weathering.

In situ vane shear strength tests carried out in the unweathered silty clay gave undrained shear strengths ranging from about 24 to 95 kilopascals, which indicate a soft to stiff consistency, generally increasing with depth.

Atterberg limit testing was carried out on eight samples of the unweathered silty clay. The results are provided in Appendix B and are summarized in Table 4.4. The measured water content of 34 samples of the unweathered silty clay ranges from about 25 to 59 percent. The testing indicates that the samples of unweathered silty clay from the boreholes generally have a low plasticity. Based on the results of the laboratory testing, the unweathered silty clay deposits can be classified as CL under the USCS.

Table 4.4 – Summary of Atterberg Limit Test (Silty Clay)

Borehole ID / Sample No.	Water Content (%)	Liquid Limits (%)	Plastic Limits (%)	Plasticity Index
23-05 / 6	44	34	19	15
23-06 / 7	38	31	17	14
23-08 / 11	44	34	18	16
23-11 / 5	26	18	15	3

Borehole ID / Sample No.	Water Content (%)	Liquid Limits (%)	Plastic Limits (%)	Plasticity Index
23-11 / 7	38	23	15	13
23-13 / 9	31	27	15	12
23-14 / 8	36	26	16	11

One laboratory oedometer consolidation test was carried out on a Shelby tube sample from borehole 23-11. The results are summarized in Table 4.5, below. A plot of the variation in void ratio with applied stress from the consolidation test is presented in Appendix C.

Table 4.5 – Summary of Oedometer Testing

Borehole ID	Sample Depth (metres)	Estimated Apparent Past Preconsolidation Pressure, P_c', (kilopascals)	Calculated Existing Vertical Effective Stress, P_o' (kilopascals)	Initial Void Ratio, e_o	Recompression Index, C_r	Compression Index, C_c
23-11	6.1	170	65	1.07	0.008	0.68

4.5 Groundwater Levels

Well screens were sealed in boreholes 23-02, 23-05, 23-06, 23-08, 23-09, 23-12, and 23-14. The groundwater levels in the monitoring wells were measured on April 27, 2023 and the groundwater level depth and elevation are summarized in Table 4.6.

Table 4.6 – Summary of Groundwater Levels

Borehole ID.	Ground Surface Elevation (metres)	Groundwater Depth (metres)	Groundwater Elevation (metres)
23-02	110.2	0.5	109.7
23-05	103.7	0.6	103.1
23-06	106.0	3.3	102.7
23-08	106.3	5.1	101.1
23-09	103.8	0.7	103.2

Borehole ID.	Ground Surface Elevation (metres)	Groundwater Depth (metres)	Groundwater Elevation (metres)
23-12	108.7	0.4	108.2
23-14	110.0	1.0	109.0

4.6 Hydraulic Conductivity of Soils

Single-well hydraulic tests were performed in all on-site wells by purging the wells to a known depth and monitoring water level recovery. A summary of this testing is provided in Table 4.7.

Table 4.7 – Summary of Rising Head Hydraulic Test Results

Borehole ID.	Geological Material Screened	Displacement (metres)	Recovery Time (minutes)	Recovery (percent)
23-02	Silty clay	7.2	26	5
23-05	Silt and sand	1.4	31	97
23-06	Silty clay	10.2	20	8
23-08	Silty clay	8.8	20	1
23-09	Silt and sand	3.5	20	59
23-12	Silty clay	6.8	20	11
23-14	Silty clay	11.2	20	3

Recovery during the hydraulic tests were too slow for detailed analysis of the data in wells other than borehole 23-05. The bulk hydraulic conductivity calculated by applying the Hvorslev method of analysis for unconfined aquifers to the recovery data of borehole 23-05 was about 7×10^{-7} metres per second; the results of this test are provided in Appendix F. An assessment of the recovery data for boreholes 23-09 and 23-12 reflect bulk hydraulic conductivities of less than about 3×10^{-6} and 1×10^{-6} metres per second, respectively. For dewatering purposes, the remaining screened layers within the monitoring wells may be assumed to have hydraulic conductivities of less than about 1×10^{-7} metres per second.

The estimated hydraulic conductivity value for the silty clay deposits on site (less than about 10^{-7} metres per second) are reasonable in the context of literature values for unweathered marine clays (approximately 10^{-12} to 10^{-8} metres per second; Freeze and Cherry, 1979). The observed weathering and sand seams reported in the silty clay deposits are anticipated to increase the hydraulic conductivity of these layers.

Silt and sand layers of variable thicknesses were recorded in the upper six metres of overburden across most of the site. Literature values for silt and silty sand range from approximately 10^{-9} to 10^{-3} metres per second (Freeze and Cherry, 1979), and on-site measurements fall within the range of literature values. Given that the highest estimated hydraulic conductivity (3×10^{-6} metres per second) reflects a bulk measurement of various soil types and that thicker silt and sand deposits have been record on site (e.g., borehole 23-14), it is advisable to adopt a higher hydraulic conductivity for dewatering calculations.

4.7 Corrosion Potential of Soil and Groundwater

Two soil samples obtained, one each from boreholes 23-03 and 23-08 were sent to Paracel Laboratories Ltd. 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.

Table 4.8 – Summary of Corrosion Testing

Parameter	Borehole 23-03 Sample 3	Borehole 23-08 Sample 9
Chloride Content (µg/g)	<10	<10
Resistivity (Ohm.m)	94.6	22.1
Conductivity (µs.cm)	106	452
pH	7.53	7.57
Sulphate Content (µg/g)	16	174

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions. The 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 offsite sources are outside the terms of reference for this report and have not been addressed.

5.2 Proposed Commercial Buildings

5.2.1 Overview

It is assumed that the commercial buildings within the development will consist of one or two storey buildings with slab on grade construction (i.e., no basement level). Further it is assumed that the buildings will be supported on shallow footing not deeper than about 1.8 metres below existing surface. Further assumptions are provided in the relevant subsections of the report.

The following sections provide preliminary recommendations for the commercial buildings according to the assumed building configurations. Site specific geotechnical assessment and/or investigation should be carried out where the buildings differ from these assumptions.

5.2.2 Excavation

The excavations for building foundations will be carried out through topsoil, silt and sand deposits and weathered silty clay. Deeper excavation may extend into the grey silty clay deposit, however, this should be avoided in general to reduce loading on this portion of the soil profile.

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 overburden soils at this site can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, above the groundwater level. Below the groundwater level, the sands/ granular soils can be classified as Type 4 soils and, an allowance should be made for excavation side slopes of 3 horizontal to 1 vertical, or flatter.

The overburden deposits are sensitive to disturbance from ponded water, vibration and construction traffic. As such, 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 or silty sand with compacted sand and gravel, such as that meeting Ontario Provincial Standards Specification (OPSS) Granular A or Granular B Type II, where required.

5.2.3 Groundwater Management

Depending on the amount of grade raise filling at the proposed buildings, the excavations for the foundations may extend below the measured groundwater levels at the site. Consideration could be given to dewatering the overburden deposits in advance of excavation (e.g., using regularly spaced well points) in combination, where necessary, with pumping from within the excavation.

To reduce the amount of dewatering, it is suggested that the proposed construction not be undertaken during spring conditions. Refer to Section 5.3 for commentary on groundwater pumping and permitting requirements.

5.2.4 Gade Raise Restriction

The site is underlain by deposits of sensitive silty clay, which has a limited capacity to support loads imposed by grade raise fill material, pavement structures, and foundations. The placement of fill material on this site must therefore be carefully planned and controlled so that the stress imposed by the fill material does not result in excessive consolidation of the silty clay deposit. Concrete slabs, granular base materials, overall grade raise and pavement structures are considered grade raise filling. Groundwater lowering also results in a stress increase on the underlying sensitive silty clay deposit.

Based on the results of the subsurface investigation, the maximum thickness of any grade raise filling should be limited to **1.5 metres** above the existing ground surface. This value should be considered preliminary, and could be refined by GEMTEC (either up or down) following investigations on a block by block basis according to the proposed building configuration and soil conditions encountered.

The grade raise restriction for the commercial development has been calculated in order to limit the total settlement of the ground to about 25 millimetres in the long term. For design purposes, we have made the following assumptions:

- The groundwater lowering due to the development at this site will be at most 0.5 metres below the underside of footing elevation;
- The unit weight of the grade raise material used in the vicinity of the buildings will not be greater than 22.0 kilonewtons per cubic metre; and,
- The grade raise fill material used below the buildings, where required, will be composed of compacted granular material having a unit weight of 22.0 kilonewtons per cubic metre.

If heavier grade raise fill material is used, the maximum grade raise will have to be reduced accordingly.

5.2.5 Foundation Design

Based on the subsurface conditions which were encountered during the investigation, it is considered that the proposed structures can be founded on spread footings bearing on or within the native weathered silty clay crust, or silt and sand layers, subject to confirmation of the building configuration and loading. The topsoil and any fill material, if encountered, is considered to be compressible and is not considered suitable for the support of the foundations. Therefore, all topsoil and fill material should be removed from the proposed building areas.

From a spread footing design perspective, it is preferable to maximize the vertical separation between the underside of the footings and the surface of the softer, grey silty clay to distribute the foundation loads onto the softer, grey silty clay at depth. This can be achieved by founding the structure as high as practical within the soil profile and minimizing the amount of fill (surcharge) on the site.

5.2.5.1 Preliminary Estimates of Bearing Resistances

The details on the proposed buildings, and underside of foundation depths/elevations are not known at the time of writing this report. Preliminary foundation sizes, depths, and bearing pressures are provided in Table 5.1 below, which assume that the foundations will bear upon stiff to very stiff silty clay. These values should be considered preliminary in nature, and should be verified by GEMTEC on a block by block basis once the designs are known. The post construction total and differential settlement of footings at SLS should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces.

Table 5.1 – Summary of Preliminary Bearing Resistances

Type of Footing	Underside of Footing Depth (metres)	Maximum Footing Size (metres)	Serviceability Limits States Bearing Resistance, SLS (kilopascals)	Factored Ultimate Limits States Resistance, ULS (kilopascals)
Strip	1.5	1.5	100	200
Pad	1.0	2.4 square	100	200

Should increased bearing resistance be required it may be necessary to consider alternative approaches such as deep foundation systems (piles), or ground improvement such as rigid inclusions. Further details can be provided by GEMTEC on a block by block basis as the designs of the structures are developed further.

5.2.5.2 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) footings that are located in areas that are to be cleared of snow should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.

If the foundation and/or slab on grade are insulated in a manner that will reduce heat flow to the surrounding soil, the foundation depth shall conform to that required for foundations for an unheated space.

5.2.5.3 Foundation Wall Backfill and Drainage

The native deposits at this site are frost susceptible and should not be used as backfill against foundations. To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material such as that meeting the requirements of OPSS Granular A, or Granular B Type I or II.

The frost susceptible native soils could be considered for foundation wall backfill purposes in soft landscaped areas provided that a suitable bond break is applied to the surface of the foundations to prevent frost jacking. A suitable bond break could consist of at least 2 layers of 6 MIL polyethylene sheeting or a proprietary plastic drainage system. It is also pointed out that the native soils at this site can be impacted by changes in moisture content and this could affect the ability to compact this material to the required density.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density value using suitable vibratory compaction equipment. Light walk behind compaction equipment should be used next to the foundation walls to avoid excessive compaction induced stress on the foundation walls.

Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the material’s standard Proctor maximum dry density value using suitable vibratory compaction equipment.

Where areas of hard surfacing (concrete, sidewalks, pavement, etc.) abut the proposed structures, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible fill material to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from 1.5 metres below finished grade to the underside of the granular subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter. Further, it is recommended that downspouts outlet in such a way as to prevent saturation of soils below hard surfaced areas.

Perimeter foundation drainage is not considered necessary for a slab on grade structure provided that the floor slab level is above the finished exterior ground surface level.

5.2.6 Slab on Grade Support

The topsoil is not considered suitable for support of the slab on grade. To prevent long term settlement of floor slabs, all organic material and any fill material, if encountered, should be removed from below the proposed slab to expose the native overburden deposits.

The grade within the proposed building could then be raised, where necessary, with material meeting OPSS requirements for Granular A and Granular B Type I or II. The granular base for the proposed slab on grade should consist of at least 200 millimetres of OPSS Granular A.

OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the floor slab be composed of virgin material only, for environmental reasons.

All imported granular materials placed below the proposed floor slab should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the material's standard Proctor maximum dry density value.

Underfloor drainage is not considered necessary provided that the floor slab levels are above the finished exterior ground surface level. If any areas of the buildings are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

The floor slabs should be wet cured to minimize shrinkage cracking and slab curling. The slab should be saw cut to about 1/3 the thickness of the slab as soon as curing of the concrete permits, in order to minimize shrinkage cracks.

Proper moisture protection with a vapour retarder should be used for floor slabs 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 slabs.

5.3 Seismic Design Considerations

An assessment of the (geotechnical) seismic design considerations relative to Site Classification and liquefaction potential has been carried out based on the (relatively) widely spaced borehole and CPT records available at this time. Ground conditions between the investigation points are variable, in particular discontinuous sand layers have been identified in some ground investigation points, and these can be of importance for the assessment.

5.3.1 Potential for Soil Liquefaction

According to the information on the current subsurface conditions available at this time there is a potential for soil liquefaction to be triggered in the discontinuous sand layers at the site, and also to some extent in the upper silt layers.

In the case that liquefaction is triggered, compression of the soils will occur. The magnitude of settlement and the associated impact that could occur is dependent on several factors inclusive of the characteristics of the earthquake, the thickness and composition of the soils in which liquefaction is triggered, and the groundwater conditions at the time of the earthquake.

GEMTEC has considered the data obtained from the CPT probe performed during the geotechnical investigation and using the CPeT-iT and CLiq software packages finds that cyclic liquefaction is likely to be triggered for 1 in 2,500 year event for current ‘greenfield’ conditions using the Boulanger & Idriss (2014) analysis method. The method developed includes an assessment of the cyclic stress ratio (CSR), which is cyclic shear stresses resisting cyclic softening, and the cyclic resistance ratio (CRR), which is the CSR that is required to trigger a liquefaction event in the soils. A PGA value of 0.37 was applied, in combination with a magnitude 6.2 earthquake.

In the case that liquefaction was to occur liquefaction induced settlements are likely to be variable across the site, but may be up to 100 millimetres in magnitude according to the information currently available. The use of shallow spread footings is likely still viable, however, it is recommended that investigation and assessment on a block by block basis be performed to confirm the potential for liquefaction to be triggered, the likely impacts to be estimated, and potential mitigation measures to be evaluated.

5.3.2 Site Classification

The presence of potentially liquefiable soils can affect the applicable seismic Site Class and can in some instances dictate that a seismic Site Class F be applied to the design of structures.

However, as per Section 4.1.8.4.(6) of the Ontario Building Code, the seismic Site Class can be determined assuming the soil is not liquefiable for structures in which the fundamental period of vibration is less than to equal to 0.5 seconds. GEMTEC is not aware of the fundamental period of the proposed structures, given that no information on the structures is available at this time.

If the structures have a fundamental period of less than 0.5 seconds, or other measures are applied to remove and/or improve the potentially liquefiable soils, based on available geotechnical information, Site Class F would not be applicable. Seismic Site Class D would then be considered applicable to the site.

5.4 Pumping Station and Associated Structures

5.4.1 Overview

It is understood that the pumping station will be located near Blocks 23 and 24 (near the north corner of the site) with a depth of about 6 to 7 metres below the existing ground surface. The conditions encountered in boreholes 23-05, 23-06, 23-08, and 23-09 are therefore considered relevant. At the time of preparing this report, only limited details on the proposed pumping station were available. The following sections provide preliminary recommendations on the proposed pumping station.

5.4.2 Excavation and Shoring

Based on the proposed depth of the pump station, relatively deep excavations will be undertaken at the site (up to about 7metres). Shoring of the excavation in the overburden will likely be required if there is insufficient space available to construct the pump station by open cut excavation methods.

Different retaining wall systems will provide different amounts of stiffness and ability to resist ground movements, manage groundwater, etc. However, some unavoidable inward horizontal movement and settlement should be anticipated with all the available options. Retaining wall systems commonly used to provide shoring to such excavations include:

- Proprietary shoring systems;
- Steel soldier piles and lagging (timber or concrete);
- Driven steel interlocking sheet piles.

Proprietary trenching/shoring systems, similar to trench boxes, are advanced as the excavation proceeds and allow some movement of soils around the perimeter of the system to occur. The magnitude of movement depends on how tightly fitting the system is to the surrounding soils. In extreme cases instability of the soils around the perimeter of the system may occur. These systems do not provide a cut off to groundwater.

A soldier pile and lagging wall system may be acceptable to reduce the impact of excavation on nearby structures which can accommodate a higher degree of ground movement, such as roadways. The soldier piles (typically steel H sections) would have to be driven through the silty clay layers to a more competent layer (such as glacial till). The depth to such a layer is not known at this time but may be significant and should be confirmed as the design progresses.

Sheet piles can control or cut off ground water inflow. The sheet piles would have to be driven through the silty clay layers to a sufficient depth below the bottom of the excavation.

Depending on the depth of excavation, the shoring methods listed may require some form of lateral support depending on the wall height and configuration. Commonly used lateral restraint systems which may be considered in this instance include:

- Interior struts which are connected to the opposite side of the excavation; and,
- Circular or rectangular waler beams (ring beams).

Grouted bedrock anchors are likely not suitable, or if to be considered further the depth to bedrock should be confirmed.

The design and implementation of the excavation shoring is the responsibility of the contractor. It is recommended that any successful bidder submit a shoring system design, including lateral earth pressure design details, expected movements, and a monitoring plan for review by the geotechnical engineer prior to the start of the shoring construction.

The design of the shoring system to support the excavation must consider: the soil stratigraphy, groundwater conditions, methods of groundwater management, possible ground movements associated with the construction of the shoring system, excavation and other potential impacts, and/or requirements to protect the system during freezing weather conditions (if applicable).

When designing shoring, lateral earth pressures resulting from the weight of the retained earth and other dead and surcharge loads will need to be considered. Hydro-static water pressures should also be accounted for where non-permeable or low permeability wall systems are used, or where there is potential for hydro-static pressures to develop on the wall. The earth pressure distribution used for shoring design is dependent on the shoring wall design and the lateral support provided. The earth pressure parameters can be provided as the design progresses.

Other stiffer retaining wall systems such as concrete (reinforced) contiguous piling and concrete secant piling (reinforced and unreinforced depending on the system) are also available but are likely not warranted for this application and will carry significant extra costs over the other systems. These types of shoring can also control or cut off ground water inflow.

5.4.3 Groundwater Management

Excavation for the pump station will extend below the groundwater level. The level of groundwater management required during excavation for the pump station will depend in part on the excavation approach – i.e. open cut or shoring (and the type of shoring used).

Groundwater inflow into the excavation is anticipated from the sandy layers within the silty clay unit. Groundwater inflow from the sides and base of the excavation could likely be handled by pumping from sumps within the excavations. Sump pumps should be installed in perforated casings surrounded by graded granular sand to reduce the potential for loss of fines into the sump.

5.4.4 Below Ground Slab Support and Drainage

The base of the proposed pump station is likely to be underlain by firm silty clay; therefore, the base may be supported on a mat type foundation, or spread footing foundations.

Within the excavation for the pump station, the silty clay at the base may soften because of stress relief, and upward seepage, and as such it may be necessary to place a confining layer of coarse aggregate in combination with a geotextile separation layer following excavation to maintain the integrity of the base and provide a working platform. To provide predictable settlement performance of the below ground slab within the pump station, all disturbed soil, and other deleterious materials should be removed from below the slab area.

The base for the floor slab should consist of at least 300 millimetres of OPSS Granular A or 19 millimetre clear crushed stone with a non-woven geotextile meeting OPSS 1860 Class I requirements wherever the clear stone will be in contact with the native soils. The OPSS Granular A should be compacted in maximum 150 millimetre thick lifts to at least 95 percent of the material’s standard Proctor maximum dry density value. Nominal compaction of the clear stone with at least 2 passes of a diesel plate compactor is recommended to consolidate the material into place.

Underfloor drains should be provided below the pump station floor slab (assuming that the structure is not designed to resist hydro-static uplift pressures). If OPSS Granular A material is used below the basement floor slab, it is suggested that drainage be provided by means of plastic perforated pipes spaced at about 5 metres on centre or as required, to link any hydraulically isolated areas.

If clear crushed stone is used below the floor slab, underfloor drains are not considered essential provided that the clear stone can outlet to the sump from which the water is pumped, and drains are installed to link any hydraulically isolated areas.

In the case that underfloor drainage is not provided, the floor slab should be designed to resist uplift due to hydro-static pressure below the base of the structure. Resistance to hydro-static pressure could be provided by:

- Increasing the dead weight of the structure;
- Extending the base of the structure beyond the foundation walls; and,
- Installation of rock anchors (although this may not be cost effective for the likely depth to bedrock at the site).

As a conservative approach the groundwater level should be assumed to be at ground surface to determine uplift acting on the structure. Further details on these options can be provided, if required, as the design progresses.

5.4.5 Pump Station Associated Structures - Shallow Foundations

For preliminary foundation design of structures associated with the pump station, which will be supported on shallow spread footings, all fill material, deleterious materials including any organics should be removed from below the foundations of the pump station. In addition, any disturbed or water softened soils should also be removed. This includes any disturbed ground around the perimeter of the deep excavation for the pump. Refer to Section 5.2 of this report for further guidelines.

5.4.6 Foundation Wall Backfill and Drainage

The foundation wall backfill of the pumping station should be carried out as per Section 5.2, above.

Perimeter drainage should be applied for the below ground portions of the pump station (assuming the structure is not designed to be watertight).

5.5 Proposed Services

5.5.1 Excavations

Based on the plan and profile drawings provided, it is understood that the proposed services will have an invert depth of up to about 5 metres below the existing ground surface. The excavations for the services will be carried out through topsoil, weathered silty clay crust, silt and sand, and in some locations into the grey silty clay. The excavation works will likely be undertaken below the measured groundwater level.

Excavations for the installation of the services should be carried out as per Section 5.2, above.

As an alternative to sloped excavations where deeper excavations are anticipated, 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, in combination with suitable groundwater management measures.

As noted above, excavations for the services are likely to extend below the measured groundwater levels at the site. Excavation of the native overburden deposits above the groundwater level should not present significant constraints. Below the groundwater level, sloughing of the silt and sand layers into the excavation should be anticipated along with disturbance to the soils in the bottom of the excavation. Sloughing of the excavation side slopes below the groundwater level could be reduced, where necessary, by advancing thick steel plates along the sides and front of the trench box to below the level of the excavation (and into the less permeable native silty clay deposits) in combination with pumping from within the excavation.

Alternatively, sloughing of the excavation side slopes and disturbance to the soils in the bottom of the excavation could be reduced by dewatering the overburden deposits in advance of excavation. As an example, this could be achieved by pumping from regularly spaced well points in combination, where necessary, with pumping from within the excavation. It is noted that groundwater flow to the well points will be limited by the amount of fine grained particles in the coarser soil layers. Although well points are not expected to be wholly effective, the amount of sloughing and subgrade disturbance is expected to be less compared with dewatering solely from within the open excavation.

Saturated deposits of weathered silty clay crust or layers of silt and sand may be encountered at subgrade level along the alignment of the proposed services. These deposits are susceptible to weakening under vibration and/or repeated loading and it is suggested that final trimming to subgrade level be carried out using a hydraulic shovel equipped with a flat blade bucket. We recommend that a contingency allowance be made in the contract for a 300 millimetre thick subbedding layer of OPSS Granular B Type II granular material and a woven geotextile separator meeting OPSS 1860 Class I requirements in the event that the subgrade soils are disturbed during construction.

5.5.2 Groundwater Management

The depth of excavation for the proposed services will be up to about 5 metres below the existing ground surface, for the purpose of dewatering calculations. Calculations were performed assuming a single open excavation with dimensions of 30 metres long by 5 metres wide. Assumptions and variables adopted for the preliminary dewatering calculations are presented in Appendix F.

Based on the preliminary dewatering calculations (Appendix F), it is anticipated that groundwater management will be required to maintain dry excavations, particularly where sandy layers are encountered. Where sandy layers are not encountered, dewatering requirements are expected to remain below 50,000 Litres per day. Nonetheless, an Environmental Activity and Sector Registry (EASR) is recommended for construction dewatering purposes. An EASR is required for construction dewatering between 50,000 litres per day and 400,000 litres per day.

GEMTEC can prepare a water taking and discharge report to support the permit application under a separate scope of work, upon request. It is recommended that the design drawings and development plans are reviewed once they are available to confirm permitting requirements in advance of report preparation and application for a permit.

It is anticipated that groundwater inflow will be manageable by pumping from filtered sumps within the assumed excavations. Any groundwater discharge or disposal should be carried out in accordance with provincial and local regulations. Settlement concerns related to dewatering activities should be considered once more information is available regarding the proposed development and construction sequencing.

5.5.3 Effects of Temporary Groundwater Lowering

It is noted that the radius and depth of influence and, consequently, the risk of negatively impacting adjacent structures, is expected to be greater for a well point dewatering system when compared with only dewatering from within an open excavation.

We recommend that preconstruction surveys of existing buildings including a visual evaluation of accessible walls, windows, floors and interior finishes for spalling, cracks, etc. be carried out so that possible construction related claims can be dealt with in a fair manner.

5.5.4 Pipe Bedding

The pipe bedding material should consist of at least 150 millimetres of well graded crushed stone meeting OPSS for Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A. 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.

In areas where the subsoil is disturbed, the disturbed material should be removed and replaced with a subbedding layer of compacted granular material, such as that meeting OPSS Granular B Type II (50 or 100 millimetre minus crushed stone). As previously indicated, saturated silt and sand deposits may be encountered at subgrade level along the proposed alignments. It is noted that these deposits are susceptible to weakening under vibration and/or repeated loading. It is recommended that a contingency allowance be made in the contract for a 300 millimetre thick subbedding layer of OPSS Granular B Type II granular material and a woven geotextile separator meeting OPSS 1860 Class I requirements in the event that the subgrade soils are disturbed during construction.

The use of clear crushed stone as a bedding or subbedding material should not be permitted at this site.

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 subbedding, bedding and cover materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the material’s standard Proctor maximum dry density value using suitably sized vibratory compaction equipment.

5.5.5 Thrust Restraint for Watermain

The anticipated subsurface conditions at the depth of the proposed watermain may consist of silt and sand deposits and/or weathered silty clay crust.

In areas where the subgrade below the thrust bock is disturbed or where unsuitable material exists below the pipe subgrade level, the disturbed/unsuitable material should be removed and replaced with a layer of compacted granular material, such as that meeting OPSS Granular B Type II. Any compacted Granular B Type II should extend at least 1.5 metres horizontally beyond the thrust block and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the material’s standard Proctor dry density value. The following parameters could be used for design purposes:

Coefficient of friction between granular backfill and smooth PVC pipe:	0.25
Bearing pressure for thrust blocks bearing on native soil or on a pad of compacted granular material on native soil:	75 kilopascals

The above allowable bearing pressure for the thrust blocks assumes that they are vertical and bear on native, undisturbed soil. The bearing pressure should be reduced if fill material is encountered at the depth of the thrust block, if the soil is not excavated vertically, or if the soils are disturbed.

5.5.6 Trench Backfill

To reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway, acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration (i.e., 1.8 metres below finished grade). Where these cover requirements are not practicable, the pipe could be protected from frost using a combination of earth cover and insulation. Further details regarding insulation could be provided, if required. The backfill materials within the zone of frost penetration should match the 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 or II.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, curbs, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the material’s standard Proctor maximum dry density value using suitably sized vibratory compaction equipment. In landscaped areas, the trench backfill could be compacted to at least 90 percent of the material’s standard Proctor maximum dry density value, provided that some settlement of the finished ground surface is acceptable.

The soils at this site may have moisture contents that are too high for compaction to the required density levels. Furthermore, some of these materials are sensitive to changes in moisture content due to precipitation. As such, the specified densities will not be possible to achieve unless management of the water content of the excavated soils is implemented, and, therefore, some settlement of these backfill materials could occur.

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 in the roadway for 3 months, or longer, to allow the trench backfill settlement to occur and thereby improve the final roadway appearance.

If additional material is required for trench backfill, consideration could be given to importing relatively dry earth fill material or imported OPSS Select Subgrade Material (SSM) below the zone of frost penetration.

5.5.7 Seepage Barriers

Seepage barriers should be installed within the service trench to prevent the granular bedding in the service trench from acting as a 'French Drain' and possibly promote groundwater lowering. The seepage barriers could be located just inside the project limits, and additional seepage barriers placed at approximately 100 metre intervals along the services. Additional seepage barrier locations could be provided as the design progresses.

The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround, to the surface of the native soil, and horizontally across the full width of the service trench excavation. The seepage barrier could consist of at least 1.5 metre wide dyke of compacted weathered silty clay, or a synthetic impermeable membrane.

5.6 Trenchless Crossing of the Northeast Tributary

5.6.1 General

Based on the plan and profile drawings, provided by NOVATECH, it is understood that the proposed 150 and 300 millimetre diameter watermain and 150 millimetre diameter sanitary sewer will cross the Northeast Tributary by trenchless construction methods. The watermains and sanitary sewers will have invert depths of at least 2.4 and 2.2 metres below the underside of the Northeast Tributary, respectively (with the bottom of the tributary located at an elevation of about 102.3 metres). A length of about 75 metres of piping is anticipated to be installed by trenchless works.

Boreholes 23-05, 23-06, 23-08, and 23-09 were advanced on opposite sides of the Northeast Tributary and encountered mixed soil conditions inclusive of weathered silty clay crust and layers of silt and sand to depths ranging from about 3.1 to 3.8 metres below the ground surface. These overly generally firm silty clay to a depth of greater than 15 metres below the existing ground surface. Installation below the groundwater level may be required. The conditions below the Northeast Tributary have not been investigated and may differ from those encountered in the boreholes described above.

5.6.2 Crossing Alternatives

We have considered the following possible alternatives for the proposed services crossing of the Northeast Tributary;

1. Jack and Bore (or auger boring);
2. Pipe Ramming; and,
3. Horizontal Directional Drilling

The preliminary geotechnical issues associated with each of these construction alternatives are discussed in the following sections.

Consideration should be given to carrying out additional investigation once the alignment, trenchless method selection, and invert elevation(s) are finalized to assess the subsurface conditions within the tributary at the crossing location.

5.6.3 Alternative 1: Jack and Bore

Jack and bore or auger boring involves jacking a casing pipe with a cutting head while rotating helical augers behind the head return the spoil to the jacking pit. The spoil can then be removed from the jacking pit by mechanical means. The following geotechnical issues should be considered as part of a jack and bore tunnel alternative:

- The tunnel length of about 75 metres required to cross beneath Northeast Tributary could be achieved using jack and bore equipment.
- The horizontal tunnel bore will be carried out mostly within deposits of firm, grey silty clay with seams of silt and sand. However, sandy soil layers are also likely to be encountered and in such soils there is potential for failure of the face and surface water intrusion into the bore and jacking pit.
- Allowance should be made for jacking and receiving pits on each side of the creek crossing. The jacking pit would have to be constructed to provide sufficient reaction to advance the casing and auger through the soil. Where space allows, temporary jacking/receiving pits in the existing silty clay could be carried out in open cut or within an adequately braced supported excavation. Additional details regarding jacking and receiving pits could be provided as the design progresses.
- The obvert of the tunnel below the Northeast Tributary should be located at least two casing diameters below the underside of the base of the river.
- Allowance should be made for a small amount of overcut of the tunnel casing along with adequate lubrication (such as bentonite slurry) between the casing and the soil. This overcut, combined with a stress reduction in the overburden will likely cause some unavoidable ground loss (settlement) above the tunnel. To minimize the potential for ground loss above the tunnel alignment, the tunnel casing should be advanced immediately behind the head of the augering equipment and any significant voids between the casing and the native soil could be filled with cementitious grout.
- In order to install multiple services using jack and bore tunnelling, multiple bores will be required, with one casing for each service. For this case, the separation distance between the bores should be at least two casing diameters.

5.6.4 Alternative 2: Pipe Ramming

Pipe ramming is a trenchless method of pipeline installation that using a percuss hammer that drives a casing or pipe through the ground from a launch pit to a receiving pit. The hammer is attached to an open-ended casing. The spoil within the casing is removed when the casing is fully driven into place. The method is not steerable and hence pipes are laid in a straight line.

- Pipe ramming is similar to Jack and bore (auger boring), however the technique is more suitable for soil conditions below the ground water level as the pipe is installed prior to removal of the spoil.

- However, the length of the crossing may be prohibitive for pipe ramming and consultation with a trenchless contractor is recommended to evaluate if this option is viable with equipment available in the local market. However, installation lengths of up to about 90 metres have been completed.

5.6.5 Alternative 3: Directional Drilling

Horizontal Direction Drilling or HDD is a method that involves the drilling of a pilot hole using a steerable drill bit on a flexible string of drill rods. Once completed, the pilot bore is reamed to a larger bore diameter, in one or more passes, to the final bore diameter. The pipe is then pulled into the prepared bore. Drilling fluids are used to transport cuttings, stabilize the bore, and cool the cutting tools. Deep entrance and exit pits are generally not required. Common pipe materials used include HDPE and PVC. HDD is typically employed for creek crossings as it is a steerable method capable of creating a curved tunnel profile beneath the creek bed. The following should be noted for HDD:

- The tunnel length required to cross beneath the Northeast Tributary could be achieved using directional drilling equipment.
- The location of the entry and exit points will depend on the maximum curvature of the bore that could be achieved. It may be necessary to lower to the proposed entry and exit points of the bore on either side of the Northeast Tributary (i.e., excavate for the entry and exit points). Similarly, limitations on the curvature of the pipe can require longer deeper installations.
- Selection of appropriate drilling fluids will be required suitable for the range of ground conditions anticipated.
- With HDD there is a risk of hydraulic fracturing and unexpected loss or escape of drilling fluids which can affect existing structures and features at a site. The profile of the crossings below the Northeast Tributary should be selected such that the potential for hydraulic fracturing and loss of drilling mud into the creek is avoided.
- Sufficient working space is required to string out and pull back the pipe through the bored hole.
- Multiple borings may be required to install the services below the Northeast Tributary using directional drilling techniques. A separation distance of at least 1 metre should be maintained between borings.

5.7 Roadway Construction

5.7.1 Subgrade Preparation

In preparation for roadway construction at this site, all surficial topsoil and any soft, wet 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 could 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 material's standard Proctor maximum dry density value using suitably sized vibratory compaction equipment.

Prior to placing granular material for the roadway, the exposed subgrade should be heavily proof rolled 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 roadway subgrade surfaces should be made smooth and crowned or sloped prior to placing the granular materials to promote drainage of the roadway base and subbase materials.

5.7.2 Pavement Design

The following minimum pavement structure is suggested for local roadways at this site, assuming that the roadways will not be used for heavy truck traffic:

- 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
- 450 millimetre thick layer of subbase (OPSS Granular B Type II);

In the absence of detailed traffic data, the thickness of asphaltic concrete and OPSS Granular B Type II subbase should be increased for heavy truck traffic, as follows:

- 120 millimetre thick layer of asphaltic concrete (50 millimetres of Superpave 12.5 Traffic Level D over 70 millimetres of Superpave 19.0 Traffic Level D); over
- 150 millimetre thick layer of base (OPSS Granular A); over
- A biaxial geogrid consisting of Tensar BX1200, or equivalent; over
- 400 millimetre thick layer of subbase (OPSS Granular B Type II);

5.7.3 Effects of Subgrade Disturbance

If the roadway subgrade surface becomes disturbed or wetted due to construction operations or precipitation, or the granular pavement materials are to be used by construction traffic (i.e., if the granular pavement materials are placed during installation of the sewers, watermains, and laterals), the Granular B Type II thicknesses provided above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase. The contractor should be responsible for providing suitable access for construction equipment.

The required thickness of the subbase materials will depend on a number of factors, including contractor workmanship and schedule, contractor methodology, soil types and weather conditions, and should be assessed by geotechnical personnel at the time of construction. In our opinion, the preferred approach from a geotechnical point of view is to:

- Proof roll the subgrade conditions at the time of construction under the supervision of experienced geotechnical personnel.
- Adjust the thickness of the subbase material and include a woven geotextile separator, as required. Unit rate allowances should be made in the contract for subexcavation and replacement with OPSS Granular B Type II.

5.7.4 Granular Material Placement

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

5.7.5 Asphaltic Cement

Performance graded PG 58-34 asphaltic cement is recommended for light duty roads while performance graded PG 64-34 asphalt is recommended for roadways with heavy truck traffic.

5.7.6 Transition Treatments

In areas where the new pavement structure will abut existing pavements (e.g., Thomas Argue Road and Russ Bradley Road), the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

5.7.7 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. It is suggested that the pavement granular material extend to suitable ditches. The bottom of the OPSS Granular B Type II should be at least 0.3 metres above the bottom of the ditch and the granular material should extend to the ditch slopes.

5.8 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 samples recovered from boreholes 21-03 and 21-08 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.

5.9 Stormwater Management Pond

2 stormwater management (SWM) ponds identified as SWM Pond A and Pond B will be constructed. The ponds will be located in the northern portion of the Site either side of the Northeast Tributary. The following is known about the SWMs:

- SWM Pond A, located on the north side of the tributary, will have a pond base elevation of about 103.3 to 103.6 metres, existing ground surface ranges from about 106.0 to 104.3 metres elevation.
- SWM Pond B, located on the south side of the tributary, will have a pond base invert elevation of about 102.2 to 102.5 metres, existing ground surface ranges from about 105.0 to 103.2 metres elevation.
- It is understood that the SWM ponds are designed to be dry during normal operations, and will outlet into the Northeast Tributary.

From GEMTEC's investigation boreholes 23-05 and 23-06, and borehole 23-09 were advanced in the general area of SWM Ponds A and B, respectively. In addition, borehole 34-11 and TP11 from Paterson (2013) were advanced in the area of SWM Pond A, and TP19 from Paterson (2013) was advanced in the area of SWM Pond B. The following preliminary recommendations are provided for the design and construction of the stormwater management pond.

5.9.1 Excavations, Backfill, and Bedding

Pond base levels are up to about 3 metres below existing ground levels, and excavations will be carried out below the surficial topsoil layer through mixed deposits of silt, sand and clay. The excavations for the stormwater management pond should be carried out as per Section 5.2.

The excavations will be carried out below the measured groundwater level and accordingly groundwater management will be required during construction. The level of groundwater management required may be significant where granular soils are encountered – noting that permeability testing borehole 23-05 resulted in higher values than other locations. Lowering of the groundwater level in sandy soils in advance of excavation may be beneficial to avoid construction difficulties.

From a geotechnical perspective any excavated soils generated during construction of the pond may be reused as general fill in landscaped areas (where settlement of the soils / frost effects is not important considerations). The soils may also be used for other purposes if some sorting of soil types and management of the moisture content of the soils can be carried out.

Appropriate permitting for groundwater management activities should be obtained in advance of construction.

5.9.2 Pond Side Slopes

The pond side slopes should be constructed no steeper than 3 horizontal to 1 vertical.

The native soil deposits, in particular sandy or silty soils, are highly susceptible to erosion from flowing water. The slopes should be provided with protection either by means of vegetation or other systems as soon as practical. The pond side slopes should be protected from erosion immediately following construction using suitable erosion mats. Seeding and shrub/vegetation planting should then be implemented for long term erosion protection.

If the restriction of water flow is not required, the pond side slopes could be constructed with imported earth fill or well graded blast rock with a maximum particle size of about 100 millimetres.

Earth fill material should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the material's standard Proctor dry density value using suitable, vibratory compaction equipment. Well graded blast rock should be nominally compacted in 500 millimetres thick lifts with the hauling and spreading equipment.

5.9.3 Inlet and Outlet Structures

Concrete inlet and outlet structures, are likely to be founded on the weathered silty clay crust, the silt and sand layers, or a pad of engineered fill on the native overburden deposits. For preliminary design purposes, the headwall footings should be sized using the allowable bearing pressures provided in Section 5.2.4. And in this case the post construction total and differential settlement of the footings should be less than 25 and 15 millimetres, respectively, provided that all topsoil, loose or disturbed soil is removed from the bearing surfaces, and any engineered fill is placed and compacted as described previously.

It is recommended that depth of earth cover for frost protection be taken as 1.8 metres. If the structures are bearing on engineered fill material, the required cover could be reduced by the thickness of the engineered fill. Where the foundations will be exposed or have minimal earth cover, the subgrade surface materials below founding level could be protected with a combination of earth cover and extruded polystyrene insulation.

The inlet and outlet structures should be backfilled with free draining, non-frost susceptible sand or sand and gravel. The material should meet OPSS gradation requirements for Granular B Type I or II. The structure backfill material should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the material's standard Proctor maximum dry density value. The granular backfill material should extend at least 1.5 metres horizontally beyond the inside face of the headwall. Light, hand operated equipment should be used to compact the backfill material to prevent excessive compaction induced stress on the structures.

5.9.4 Base of Pond

Mixed soil conditions are likely to be encountered at the base level of the pond, which are likely to range from sands, silts and clays over the extent of the excavation. Estimates of hydraulic conductivity rates for these units have been provided based on a limited data set. The bedrock surface is not anticipated to be encountered within the likely depth of excavation for the pond, based on the available information.

Some disturbance and loosening/softening of the subgrade materials should be expected. Construction of haul roads and working platforms within the pond or staging / benching of the excavation will likely be required. It is suggested that final trimming to subgrade level be carried out using a hydraulic shovel equipped with a flat blade bucket.

In areas where subexcavation of disturbed material is required below the base of the stormwater management pond, the grade can be raised using imported material consisting of engineered fill meeting the requirements of OPSS Granular B Type II. The engineered fill should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the material’s standard Proctor maximum dry density.

5.9.5 Clay Liner

The groundwater level was measured to be at or above the underside of the pond elevation, and, as such, a clay liner will be required to maintain dry stormwater management ponds.

The decision to provide the proposed stormwater management pond with a pond liner, the appropriate liner type (consisting of natural materials or prefabricated materials), and any addition underdrainage works is the responsibility of the pond designer. Where a prefabricated liner is used, the liner manufacturer should be consulted for construction requirements particular to the liner. Further recommendations on the use of a clay liner can be provided as the design progresses.

5.9.6 Additional Comments

The potential for groundwater inflow to the pond (either dry or wet) should be considered. The long-term groundwater levels within the overburden were not measured as part of this investigation.

Ongoing inflow of groundwater to the pond may cause groundwater lowering to occur in the surrounding areas. Please note that a detailed hydrogeological study / model for the site and the surrounding areas has not been prepared. An assessment of the potential effect of the pond on nearby sensitive receivers, water extraction points, and potential sources of contamination that may be mobilised by the operation of the pond may influence the design approach for the pond (in particular if ongoing inflow to the pond is likely to occur).

The design of the pond should consider the provision of a suitable access route and pavements for maintenance works to be carried out over the design life of the pond. This may include for instance provision of a trafficable surface around the pond perimeter, to key infrastructure locations and to the base of the pond. Recommendations can be provided as the design progresses. If the pond base needs to be accessible placement of a rip-rap layer, concrete blocks or similar proprietary system may be required. Geotextile reinforcement may also be required.

6.0 SLOPE STABILITY ANALYSIS

6.1 General

GEMTEC has carried out a site reconnaissance and slope stability analysis 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.

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 which encompasses the area where a factor of safety of less than 1.5 against overall rotational failure is calculated, (2) Toe Erosion Allowance, and (3) Erosion Access Allowance.

The analysis was carried out using Slope/W, a two dimensional limit equilibrium slope stability program. The analysis was 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. Static and simplified seismic (or pseudo-static) analyses were carried out to model long term and seismic loading conditions, respectively. An earthquake with a return period of 2,475 years (i.e., probability of exceedance of 2 percent over a 50-year period) was considered for the simplified seismic analyses.

6.1.1 Required Factor of Safety

For the purposes of this study, for static conditions, a factor of safety of 1.5 or greater, is considered an acceptable factor of safety which allows for a degree of uncertainty.

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 computed factor of safety of between 1.3 and greater than 1.0 is generally not considered an acceptable factor of safety for long term conditions. While a factor of safety of 1.0 (or less) is considered to represent a slope which is potentially unstable.

For seismic/dynamic conditions, a factor of safety of 1.1 or greater is considered to indicate adequate stability under the design earthquake event.

6.2 Site Reconnaissance and Description of Slope

A site reconnaissance was carried out on April 26, 2023 by a member of engineering staff.

At the time of the site visits, the geometry of the slopes along the Northeast Tributary were measured at a total of six locations using precision GPS surveying equipment or hand surveying equipment. The cross sections were positioned at the site by GEMTEC personnel. The locations of the measured cross sections are provided on Figure 2. The geometries of the cross sections considered are summarized below in Table 6.1. Cross sections of the slopes are provided in Appendix G.

In general, Cross Sections AA, BB, CC, and DD of the Northwest Tributary are vegetated with grass and small to large trees and Cross Sections EE and FF are generally vegetated with tall grass with few small trees. Based on historical satellite images, it appears that the Northeast Tributary, south of about cross section EE, is a drainage ditch excavated across the area.

Erosion and undercutting of the shoreline was observed at Cross Sections AA and FF; minor erosion along the waterline was observed at Cross Sections BB, CC, DD, and EE. No signs of overall slope instability (i.e., rotational failures) were observed at the cross sections or along the Northwest Tributary.

Table 6.1 – Slope Cross Section Height and Slope Inclination

	West Side		East Side	
Cross Section	Slope Height (metres)	Overall inclination from horizontal (degrees)	Slope Height (metres)	Overall inclination from horizontal (degrees)
A-A	1.7	45	2.2	45
B-B	3.3	10 to 20	3.0	28
C-C	3.6	10 to 20	3.7	35
D-D	2.5	15 to 20	3.5	20
E-E	3.5	50	4.3	35
F-F	2.4	35 to 70	2.2	20 to 50

6.3 Analysis Inputs

6.3.1 Soil Strength Parameters

The soil conditions used in the stability analyses were based on the results of the boreholes advanced across the site. The slope stability analyses were carried out using silty clay strength parameters based on site specific studies in the area of the site. To assess the factor of safety against overall static rotational failure in long term conditions the slope stability analyses were carried out using drained soil parameters. To assess the factor of safety against overall static rotational failure during seismic conditions (i.e., earthquake loading) undrained parameters were assigned to the sandy silt layer.

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

Table 6.2 – Slope Stability Soil Strength Parameters

Soil Type	Effective Angle of Internal Friction, φ (degrees)	Effective Cohesion, c′ (kilopascals)	Undrained Shear Strength (kilopascals)	Unit Weight, γ (kN/m^3)
Weathered Silty Clay Crust	35	5	75	18.5
Grey Silty Clay	35	7	35, increasing with depth	17.9

The groundwater levels measured during this investigation range from about 0.6 to 5.2 metres below the existing ground surface. However, as a conservative approach, we have assumed full hydrostatic saturation with the groundwater level at ground surface and groundwater flow horizontally towards the slope.

The design earthquake loading is based on an acceleration of about 0.136 g (which corresponds to half the PGA, as per the 2015 National Building Code of Canada).

6.3.2 Analysis Geometry

The slope stability analysis was carried out at two cross section, namely Section ‘B-B’ and Section ‘E-E’. The cross sections were chosen based on the ‘worst case’ scenario of the Northeast Tributary and the drainage ditch, respectively. The results of the slope stability analysis are provided in Appendix G.

6.4 Results of Analysis

6.4.1 Existing Conditions

Based on the results of the analyses, the slopes along the Northwest Tributary, Section B-B is considered stable, and Section ‘E-E’ is considered to be unstable under “worst case” conditions (i.e., full hydrostatic saturation).

Based on the results of the analyses, the slopes along Northwest Tributary are considered to be stable under the design earthquake event.

The slope is not considered to be at risk of retrogressive earth flow slide failures since the height of the slopes are less than 8 metres.

The calculated factors of safety against overall rotational failure are provided in Table 6.3. It should be noted that the results of a stability analysis are highly dependent on the assumed groundwater conditions, and a conservative groundwater level has been assumed in the absence of longer term information.

Table 6.3 – Factor of Safety Against Overall Rotational Failure

Cross Section	Direction	Condition	Existing factor of safety against overall rotational failure	Figure
B-B	West	Static	4.2	D7
B-B	East	Static	2.5	D8
B-B	West	Seismic	2.0	D9
B-B	East	Seismic	2.3	D10
E-E	West	Static	1.1	D11
E-E	East	Static	0.6	D12
E-E	West	Seismic	2.8	D13
E-E	East	Seismic	2.6	D14

6.5 Setback Requirements

At Section ‘E-E’ the Stable Slope Allowance described in the MNR procedures extends about 5 and 8 metres horizontally from the crest of the slope on the west and east side, respectively.

In accordance with the MNR documents, a minimum Toe Erosion Allowance of between 8 to 15 metres is required for soft/firm cohesive soils (clays). Based on the relatively small heights of the slope (up to about 4.3 metres tall) and since minor erosion was observed along the Northeast Tributary at this section, a Toe Erosion Allowance of 8 metres should be used in the absence of a fluvial geomorphology and erosion hazard assessment.

Based on the above information, the Erosion Hazard Limit for the slopes along the Northeast Tributary will be 13 and 16 metres on the west and east sides of the tributary, respectively, as measured from the crest of the slope.

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. The Erosion Access Allowance will be in addition to the Erosion Hazard Limit, as described above. In past experience, the toe erosion allowance can be facilitated within an at grade parking area.

From a geotechnical point of view, as an alternative to providing a setback for the Erosion Hazard Limit, the Northeast Tributary south of about Section EE (i.e., where the tributary appears to be a drainage ditch), could be regraded to a flatter slope. The slope should be sloped at 3 horizontal to 1 vertical, or flatter. After regrading the slope, the slope should be vegetated to provide protections against further erosion of the slope. Erosion protection could also be provided to the ditch.

7.0 ADDITIONAL CONSIDERATIONS

7.1 Winter Construction

If construction is required during freezing temperatures, the soil below the proposed houses should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any open excavations should be opened for as short a time as practicable. The materials on the sides of the excavation 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.

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

7.2 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, 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. The magnitude of the vibrations will be much less than that required to cause damage to the nearby structures or services in good condition.

7.3 Monitoring Well Abandonment

All monitoring wells installed as part of this investigation should be decommissioned by a licensed well technician. The well abandonment could be carried out in advance of or during construction.

7.4 Disposal of Excess Soil and Re-Use of Existing Fill

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. This report does not constitute a Phase II Environmental Site Assessment (ESA) nor does it constitute a contaminated material management plan.

As indicated above, the existing granular base and subbase could be used for grade raise fill below the new parking areas, or depending on the quality of the material, possibly within the new pavement structure or as grade raise material below the floor slabs (other than in areas where the use of clear stone has been specified). The material should be carefully separated and stockpiled for evaluation by GEMTEC at the time of construction. Existing, non-deleterious earth fill could likely be used as grade raise material in soft landscaped areas, subject to approval by GEMTEC at the time of construction.

7.5 Design Review and Construction Observation

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 buildings, services, and access roadway/parking areas 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.

8.0 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.

LICENSED PROFESSIONAL ENGINEER
W. A. MEACOE
100162115
04 Nov, 2025
PROVINCE OF ONTARIO

W. Alex Meacoe

Alex Meacoe, P.Eng.
Senior Geotechnical Engineer

Daire Cummins

Daire Cummins, M.Sc.

PS/WAM/DC

Enclosures
N:\Projects\100000\100011.049\Deliverables\Geotech\100011.049_RPT_GEOT_carp airport_Rev03_2025-11-04.docx

CONDITIONS AND LIMITATIONS OF THIS REPORT

1. **Standard of Care:** GEMTEC has prepared this report in a manner consistent with generally accepted engineering or environmental consulting practice in the jurisdiction in which the services are provided at the time of the report. No other warranty expressed or implied is made.

2. **Copyright:** The contents of this report are subject to copyright owned by GEMTEC, save to the extent that copyright has been legally assigned by us to another party or is used by GEMTEC under license. To the extent that GEMTEC owns the copyright in this report, it may not be copied without our prior written agreement for any purpose other than the purpose indicated in this report. The methodology (if any) contained in this report is provided to the Client in confidence and must not be disclosed or copied to third parties without the prior written agreement of GEMTEC. Disclosure of that information may constitute an actionable breach of confidence or may otherwise prejudice our commercial interests.

3. **Complete Report:** This report is of a summary nature and is not intended to stand alone without reference to the instructions given to GEMTEC by the Client, communications between GEMTEC and the Client and to any other reports prepared by GEMTEC for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. GEMTEC cannot be responsible for use of portions of the report without reference to the entire report.

4. **Basis of Report:** This Report has been prepared for the specific site, development, design objectives and purposes that were described to GEMTEC by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document, subject to the limitations provided herein, are only valid to the extent that this report expressly addresses the proposed development, design objectives and purposes. Any change of site conditions, purpose or development plans may alter the validity of the report and GEMTEC cannot be responsible for use of this report, or portions thereof, unless GEMTEC is requested to review any changes and, if necessary, revise the report.

5. **Time Dependence:** If the proposed project is not undertaken by the Client within 18 months following the issuance of this report, or within the timeframe understood by GEMTEC to be contemplated by the Client, the guidance and recommendations within the report should not be considered valid unless reviewed and amended or validated by GEMTEC in writing.

6. **Use of This Report:** The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without GEMTEC's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, GEMTEC may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process.

   Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety, and equipment capabilities.

7. **No Legal Representations:** GEMTEC makes no representations whatsoever concerning the legal significance of its findings, or as to other legal matters touched on in this report, including but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and change. Such interpretations and regulatory changes should be reviewed with legal counsel.

8. **Decrease in Property Value:** GEMTEC shall not be responsible for any decrease, real or perceived, of the property or site's value or failure to complete a transaction, as a consequence of the information contained in this report.

9. **Reliance on Provided Information:** The evaluation and conclusions contained in this report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations. information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in this report as a result of misstatements, omissions, misrepresentations. or fraudulent acts of the Client or other persons providing information relied on by us. We are entitled to rely on such representations, information

and instructions and are not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

10. **Investigation Limitations:** Site investigation programs are a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions but even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions.

    The data derived from the site investigation program and subsequent laboratory testing are interpreted by trained personnel and extrapolated across the site to form an inferred geological representation and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Conditions between and beyond the borehole/test hole locations may differ from those encountered at the borehole/test hole locations and the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies. Accordingly, GEMTEC does not warrant or guarantee the exactness of the subsurface descriptions.

    Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

    In addition, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

11. **Sample Disposal:** GEMTEC will dispose of all uncontaminated soil and/or rock samples 60 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fill materials or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

12. **Follow-Up and Construction Services:** All details of the design were not known at the time of submission of GEMTEC's report. GEMTEC should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of GEMTEC's report.

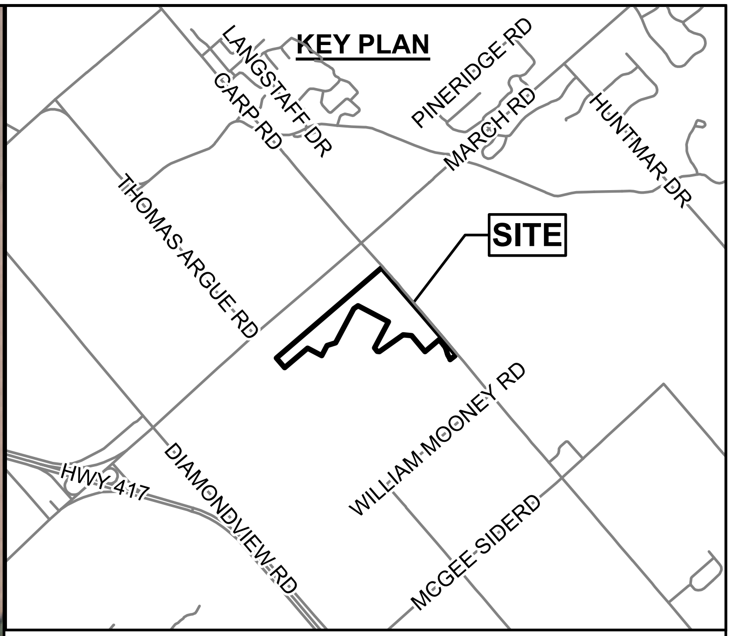
    During construction, GEMTEC should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of GEMTEC's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in GEMTEC's report. Adequate field review, observation and testing during construction are necessary for GEMTEC to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, GEMTEC's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

13. **Changed Conditions:** Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that GEMTEC be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that GEMTEC be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

14. **Drainage:** Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. GEMTEC takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

N:\Projects\100000\100011.049\Drafting\BH R0\100011.049_BH_R0_2024_09.aprx

KEY PLAN



LEGEND

BH # ← BOREHOLE ID
CPT # ← CONE PENETRATION TEST ID
XX.XX ← GROUND SURFACE ELEVATIONS, IN METRES GEODETIC DATUM

⊕	BOREHOLE (GEMTEC, 2023)
⊕	BOREHOLE (PATERSON GROUP, 2011)
⊞	TEST PIT (PATERSON GROUP, 2011)
▭	APPROXIMATE PROJECT LIMITS

DATA SOURCES AND REFERENCES:
1. Coordinate system: NAD83(CSRS) / UTM zone 18N.
2. Geographic dataset source: Ontario GeoHub.
3. Contains information licensed under the Open Government Licence – Ontario.
4. Service Layer Credits: Ottawa 2022 Imagery

Scale
1:7,000
Meters
0 100 200 400 600

Drawing
SITE PLAN

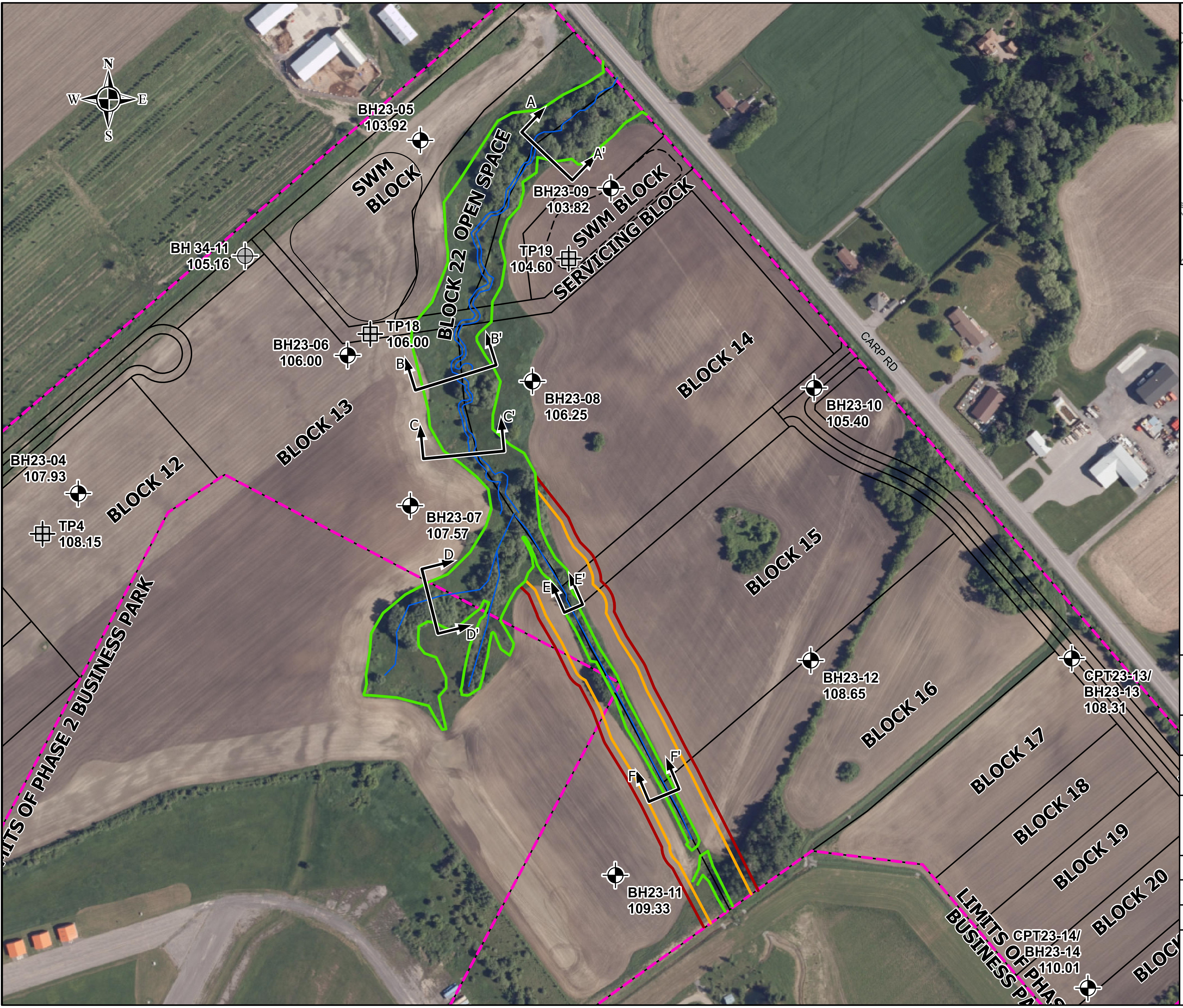
Client
NOVATECH

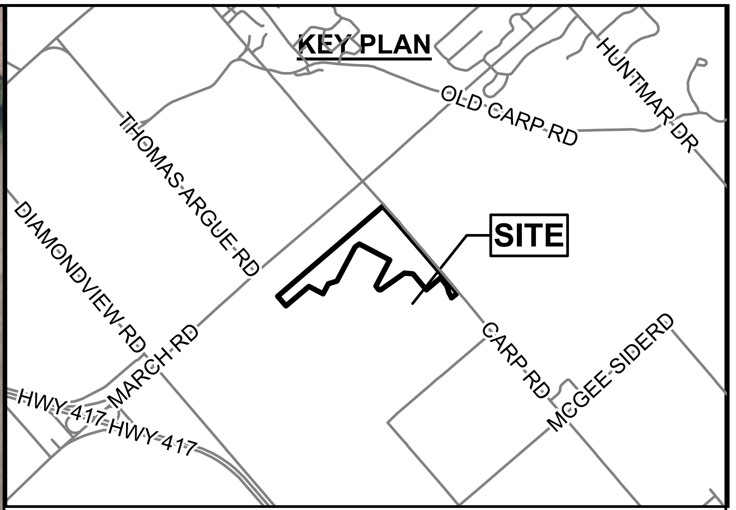
Project
GEOTECHNICAL INVESTIGATION
PHASE 2 BUSINESS PARK
CARP AIRPORT
OTTAWA, ONTARIO

Drwn By	S.L.	Chkd By	W.A.M.
Project No.	100011.049	Revision No.	0
Date	OCTOBER 2024		FIGURE 1

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

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





LEGEND

BH # ← BOREHOLE ID
CPT # ← CONE PENETRATION TEST ID
XX.XX ← GROUND SURFACE ELEVATIONS, IN METRES GEODETIC DATUM

⊕	BOREHOLE (GEMTEC, 2023)
⊞	TEST PIT (PATERSON GROUP, 2011)
—	APPROXIMATE LOCATION OF WATERCOURSE
—	APPROXIMATE TOP OF SLOPE
—	APPROXIMATE LIMIT OF HAZARD LANDS
—	EROSION ACCESS ALLOWANCE
▭	APPROXIMATE PROJECT LIMITS
A ⌊⌋ A'	SLOPE CROSS SECTION LOCATION

DATA SOURCES AND REFERENCES:

1. Coordinate system: NAD83(CSRS) / UTM zone 18N.
2. Geographic dataset source: Ontario GeoHub.
3. Contains information licensed under the Open Government Licence – Ontario.
4. Service Layer Credits: Ottawa 2022 Imagery

Scale
1:3,000
Meters
0 40 80 160 240

Drawing
CROSS SECTION LOCATION PLAN

Client
NOVATECH

Project
GEOTECHNICAL INVESTIGATION
PHASE 2 BUSINESS PARK
CARP AIRPORT
OTTAWA, ONTARIO

Drwn By	S.L.	Chkd By	W.A.M.
Project No.	100011.049	Revision No.	0
Date	OCTOBER 2024		FIGURE 2

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

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

N:\Projects\100000\100011.049\Drafting\BH R0\100011.049_BH_R0_2024_09.aprx

APPENDIX A

Record of Borehole Logs – Current Investigation
List of Abbreviations and Symbols
Boreholes 23-01 to 23-14

ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

SAMPLE TYPES	
AS	Auger sample
CA	Casing sample
CS	Chunk sample
BS	Borros piston sample
GS	Grab sample
MS	Manual sample
RC	Rock core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled open shelby tube
TP	Thin-walled piston shelby tube
WS	Wash sample

SOIL TESTS	
w	Water content
PL, w_p	Plastic limit
LL, w_L	Liquid limit
C	Consolidation (oedometer) test
D_R	Relative density
DS	Direct shear test
G_S	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
UC	Unconfined compression test
γ	Unit weight

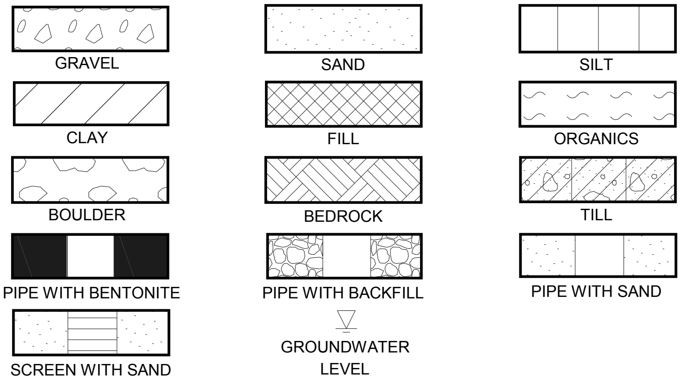
PENETRATION RESISTANCE

Standard Penetration Resistance, N
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

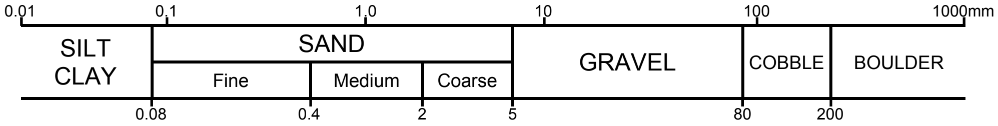
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

WH	Sampler advanced by static weight of hammer and drill rods
WR	Sampler advanced by static weight of drill rods
PH	Sampler advanced by hydraulic pressure from drill rig
PM	Sampler advanced by manual pressure

COHESIONLESS SOIL Compactness		COHESIVE SOIL Consistency	
SPT N-Values	**Description**	**Cu, kPa**	**Description**
0-4	Very Loose	0-12	Very Soft
4-10	Loose	12-25	Soft
10-30	Compact	25-50	Firm
30-50	Dense	50-100	Stiff
>50	Very Dense	100-200	Very Stiff
		>200	Hard



GRAIN SIZE



DESCRIPTIVE TERMINOLOGY

(Based on the CANFEM 4th Edition)

0	10	20	35
TRACE	SOME	ADJECTIVE	noun > 35% and main fraction
trace clay, etc	some gravel, etc.	silty, etc.	sand and gravel, etc.

GEMTEC

descriptive terms.pub
Modified May 2018

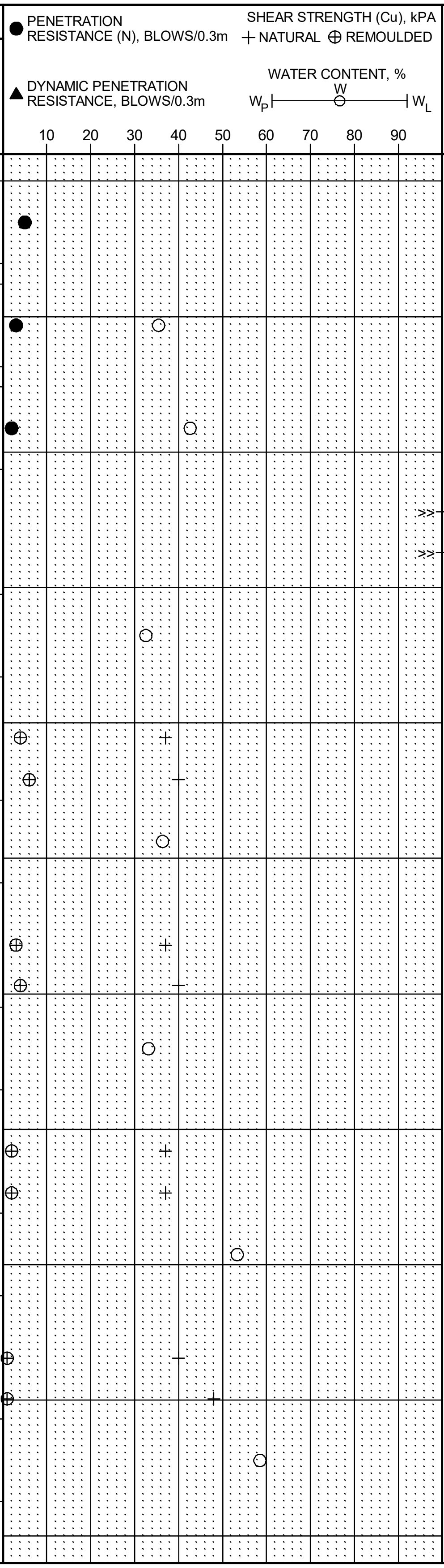
RECORD OF BOREHOLE 23-01

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 2
DATUM: CGVD28
BORING DATE: Apr 3 2023

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	109.21					
TOPSOIL	109.06 0.15					
Stiff to very stiff, grey brown (ML) CLAYEY SILT, trace to some sand, with silty sand seams (WEATHERED CRUST)		1	SS	460	5	
		2	SS	480	3	MH
		3	SS	610	2	
Firm, grey (CL) SILTY CLAY, with silty sand seams	106.16 3.05	4	SS	610	WH	
		5	SS	610	WH	
		6	SS	610	WH	
		7	SS	610	WH	
		8	SS	610	WH	
Stiff, grey (CL) SILTY CLAY, with silty sand seams	99.46 9.75					
	99.21					



PIEZOMETER OR STANDPIPE INSTALLATION

Borehole backfilled with auger cuttings

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

GEMTEC Consulting Engineers and Scientists

LOGGED: A.N.

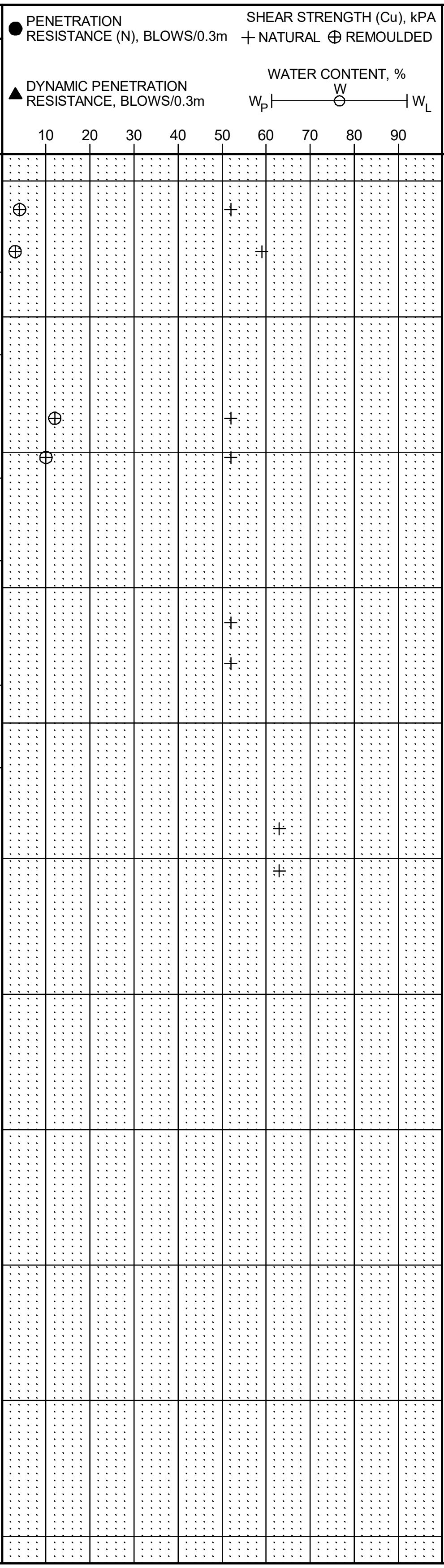
CHECKED: T.M./W.A.M.

RECORD OF BOREHOLE 23-01

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 2 OF 2
DATUM: CGVD28
BORING DATE: Apr 3 2023

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Power Auger Hollow Stem Auger (210mm OD)	Stiff, grey (CL) SILTY CLAY, with silty sand seams	▨	10.00						Borehole backfilled with auger cuttings
11					9	SS	510	WH		
12					10	SS	610	WH		
14					11	SS	610	WH		
15		End of borehole		94.12 15.09						
16										
17										
18										
19										
20										



GEMTEC
Consulting Engineers and Scientists

LOGGED: A.N.

CHECKED: T.M./W.A.M.

RECORD OF BOREHOLE 23-02

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

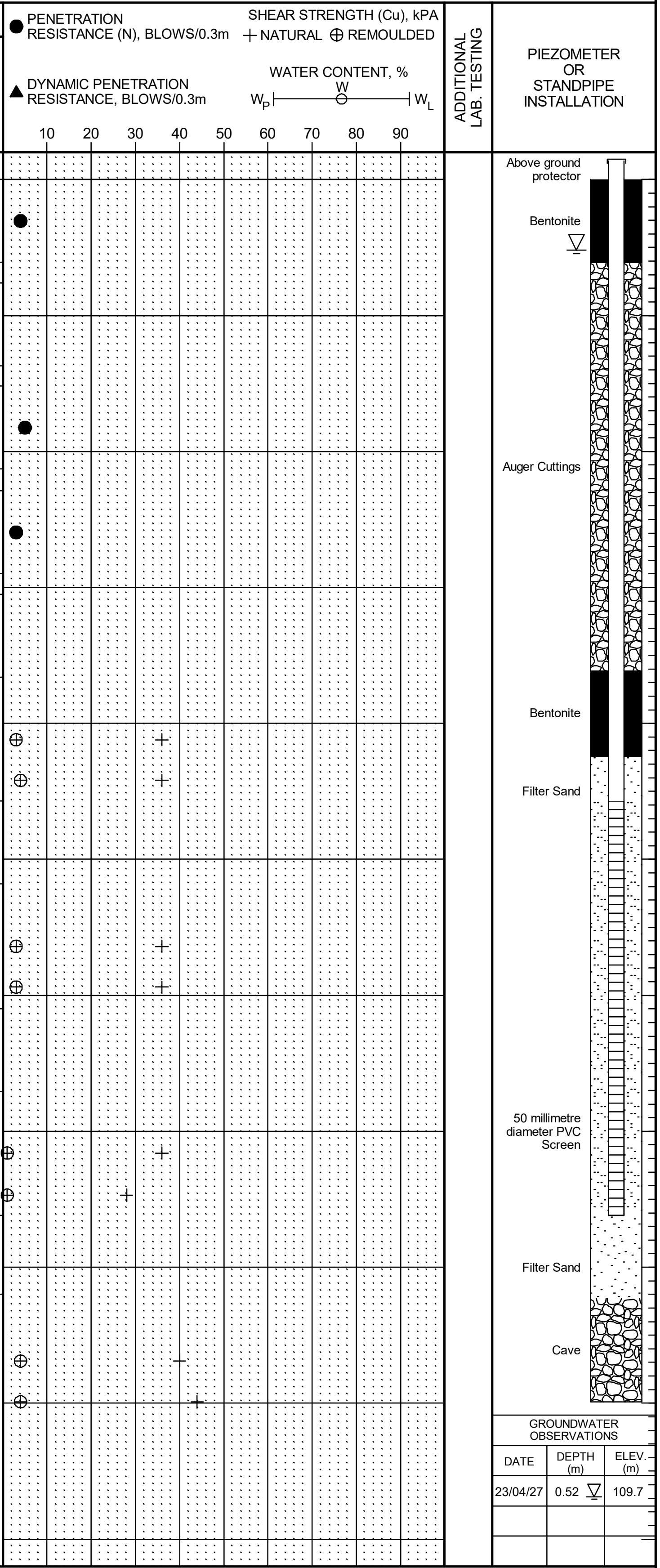
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 4 2023

DEPTH SCALE METRES: 0 – 10

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

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	110.22
TOPSOIL	
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace to some sand, with silty sand seams (WEATHERED CRUST)	109.97 0.25
Firm, grey (CL) SILTY CLAY, with silty sand seams	107.02 3.20
End of borehole	101.23 8.99

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	75	4
2	SS	405	WH
3	SS	610	5
4	SS	610	3
5	SS	610	WH
6	SS	610	WH
7	SS	610	PM
8	TP	585	PH



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/04/27	0.52 ∇	109.7

GEMTEC CONSULTING ENGINEERS AND SCIENTISTS

LOGGED: A.N.

CHECKED: T.M./W.A.M.

RECORD OF BOREHOLE 23-03

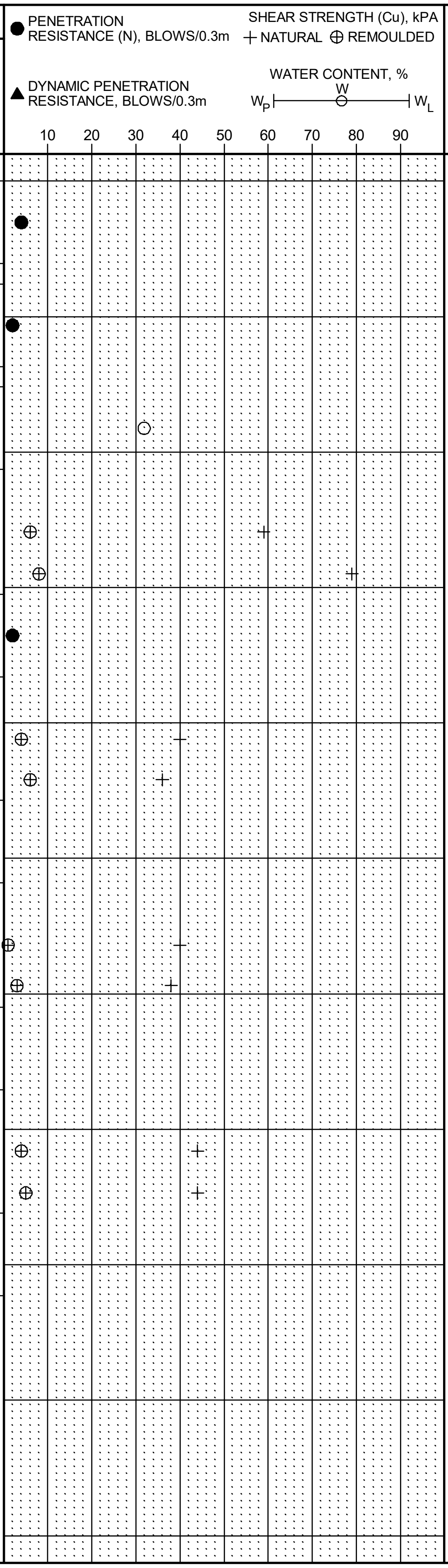
CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 4 2023

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

SOIL PROFILE DESCRIPTION	ELEV. (m)	DEPTH (m)
Ground Surface	109.10	
TOPSOIL	108.95	0.15
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace to some sand, with silty sand seams (WEATHERED CRUST)		
Stiff, grey brown (CL) SILTY CLAY, trace to some sand, with silty sand seams	107.58	1.52
Firm, grey (CL) SILTY CLAY, with silty sand seams	105.80	3.30
End of borehole	100.87	8.23

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	330	4
2	SS	405	2
3	SS	460	WH
4	SS	560	2
5	SS	610	WH
6	SS	610	WH
7	SS	610	WH



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Borehole backfilled with auger cuttings

LOGGED: A.N.

CHECKED: T.M./W.A.M.

RECORD OF BOREHOLE 23-04

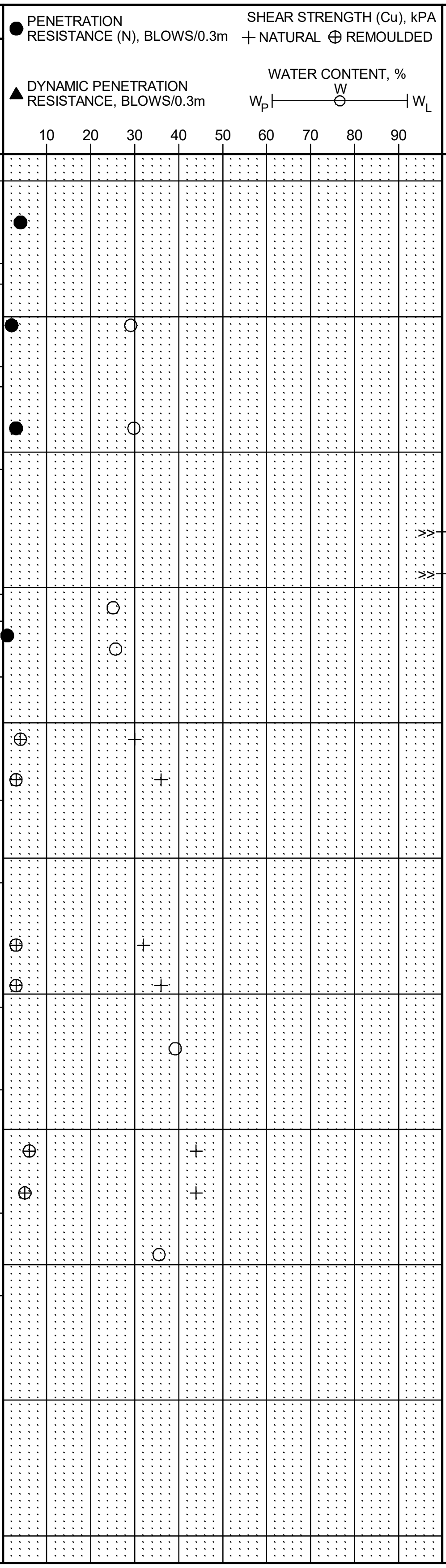
CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 4 2023

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

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	107.93
TOPSOIL	0.08
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace to some sand, with silty sand seams (WEATHERED CRUST)	
Very loose, grey brown (SM-ML) SILTY SAND to sandy SILT, some clay	106.28 1.65
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace sand (WEATHERED CRUST)	105.80 2.13
Firm, grey (CL) SILTY CLAY, with silty sand seams	104.68 3.25
End of borehole	99.70 8.23

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	330	4
2	SS	405	2
3	SS	480	3
4	SS	610	1
5	TP	610	PH
6	SS	610	WH
7	SS	610	WH



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Borehole backfilled with auger cuttings

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

LOGGED: A.N.
CHECKED: T.M./W.A.M.

GEO - BOREHOLE LOG 100011.049_BH LOGS_2023-04-19.GPJ GEMTEC 2018.GDT 11/4/25

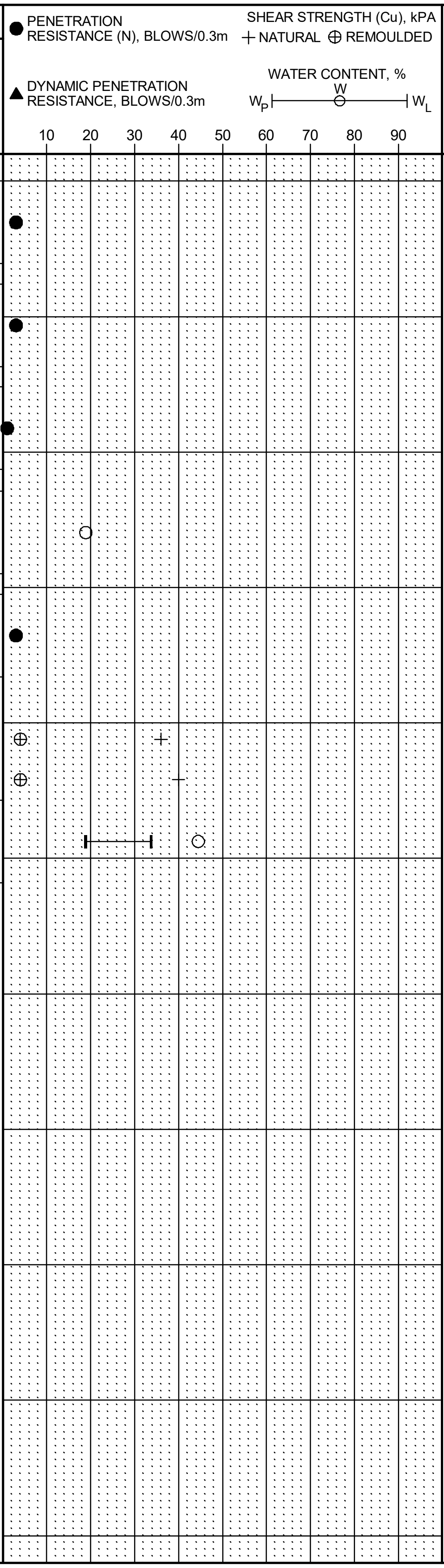
RECORD OF BOREHOLE 23-05

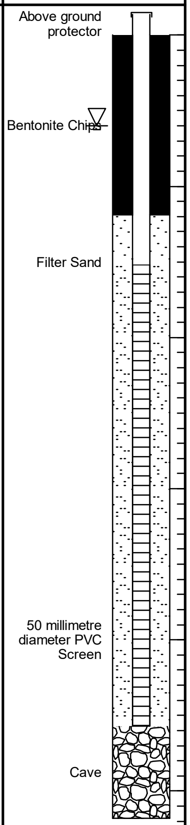
CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 6 2023

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
Ground Surface	103.74				
TOPSOIL					
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace to some sand, with silty sand and sandy silt seams (WEATHERED CRUST)	0.10	1	SS	200	3
		2	SS	480	3
Very loose, grey brown, layered, (SM-ML) SILTY SAND to sandy SILT, trace to some clay	102.22 1.52	3	SS	560	1
		4	SS	610	-
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace sand, with silty sand seams (WEATHERED CRUST)	100.69 3.05	5	SS	610	3
Firm, grey (CL) SILTY CLAY, trace sand	100.29 3.45				
		6	SS	610	WH
End of borehole	98.56 5.18				





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/04/27	0.60 ▽	103.1

LOGGED: A.N.

CHECKED: T.M./W.A.M.

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

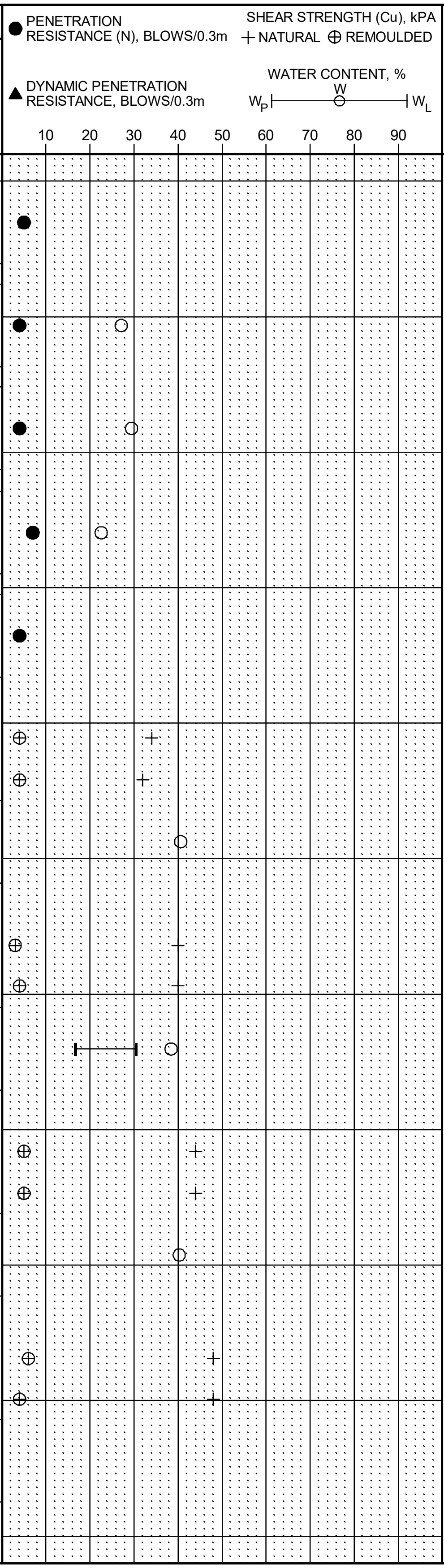
RECORD OF BOREHOLE 23-06

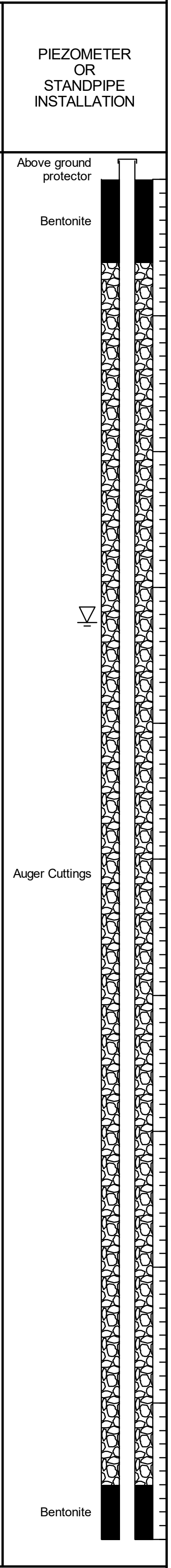
CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 2
DATUM: CGVD28
BORING DATE: Apr 6 2023

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

SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	106.00					
TOPSOIL	105.87 0.13					
Stiff to very stiff, grey brown (ML) CLAYEY SILT, trace sand, with silty sand and sandy silt seams (WEATHERED CRUST)		1	SS	355	5	
		2	SS	100	4	
		3	SS	610	4	NP
Stiff to very stiff, grey brown, layered (CL-ML) SILTY CLAY, trace sand and sandy SILT (WEATHERED CRUST)	103.71 2.29	4	SS	610	7	
		5	SS	610	4	
Firm, grey (CL) SILTY CLAY, trace sand, with silty sand seams	102.57 3.43					
Firm, grey (CL) SILTY CLAY, trace sand	101.43 4.57	6	SS	610	WH	
		7	SS	610	WH	
		8	SS	610	WH	
		9	SS	610	WH	
	96.00					





GEMTEC
Consulting Engineers and Scientists

LOGGED: A.N.
CHECKED: T.M./W.A.M.

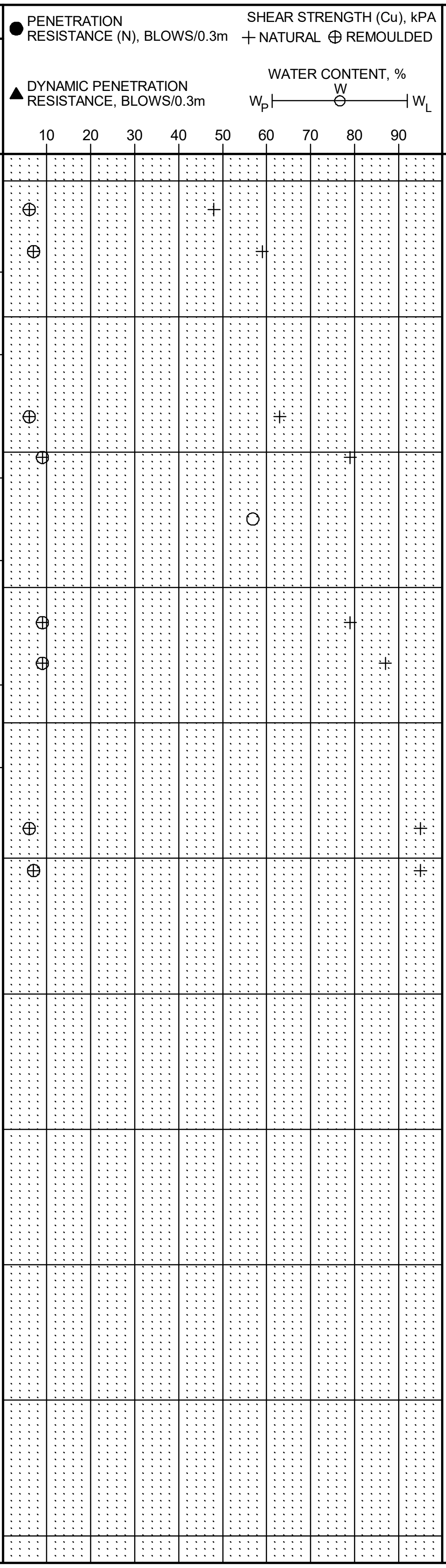
RECORD OF BOREHOLE 23-06

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 2 OF 2
DATUM: CGVD28
BORING DATE: Apr 6 2023

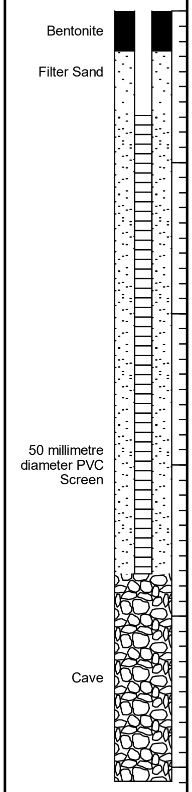
DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
10	Power Auger Hollow Stem Auger (210mm OD)	Stiff, grey (CL) SILTY CLAY, trace sand	▨	10.00
15		End of borehole		90.91 15.09

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
10	SS	610	WH
11	SS	610	WH
12	SS	610	WH



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/04/27	3.26 ∇	102.7

GEMTEC
Consulting Engineers and Scientists

LOGGED: A.N.
CHECKED: T.M./W.A.M.

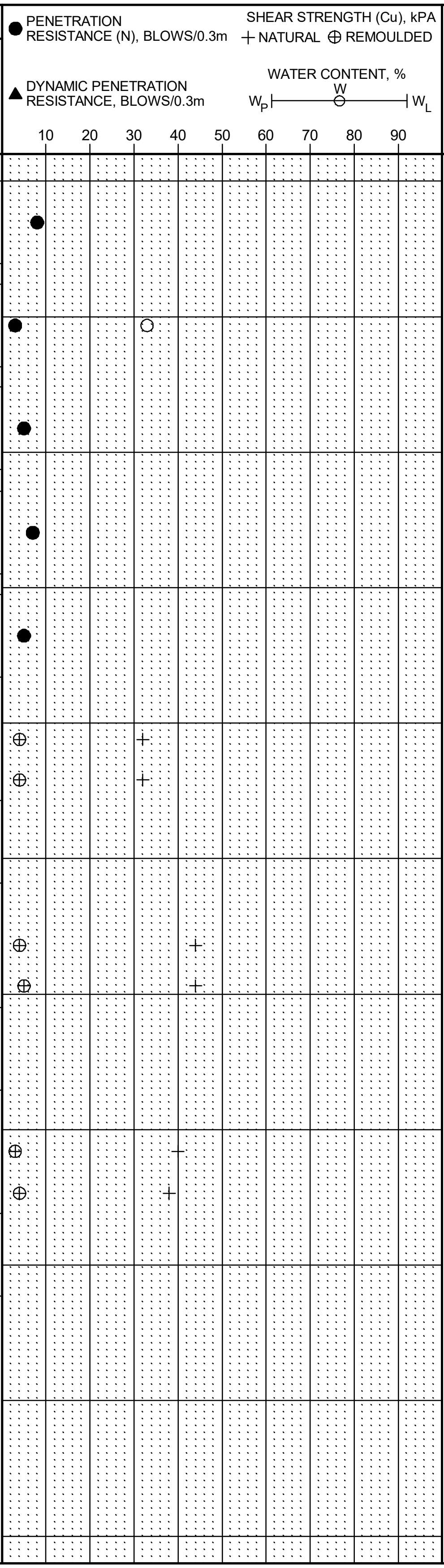
RECORD OF BOREHOLE 23-07

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 6 2023

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	107.57					
TOPSOIL	107.42 0.15					
Stiff to very stiff, grey brown (ML) CLAYEY SILT, some sand to sandy, with silty sand seams (WEATHERED CRUST)		1	SS	460	8	
		2	SS	480	3	MH
		3	SS	610	5	
		4	SS	610	7	
Loose, grey brown, (ML) sandy SILT	104.52 3.05	5	SS	610	5	
Firm, grey (CL) SILTY CLAY, trace to some sand, with silty sand seams	103.99 3.58	6	SS	610	WH	
Firm, grey (CL) SILTY CLAY	101.47 6.10	7	SS	610	WH	
		8	SS	610	WH	
End of borehole	99.34 8.23					



PIEZOMETER OR STANDPIPE INSTALLATION: Borehole backfilled with auger cuttings

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

LOGGED: A.N.
CHECKED: T.M./W.A.M.

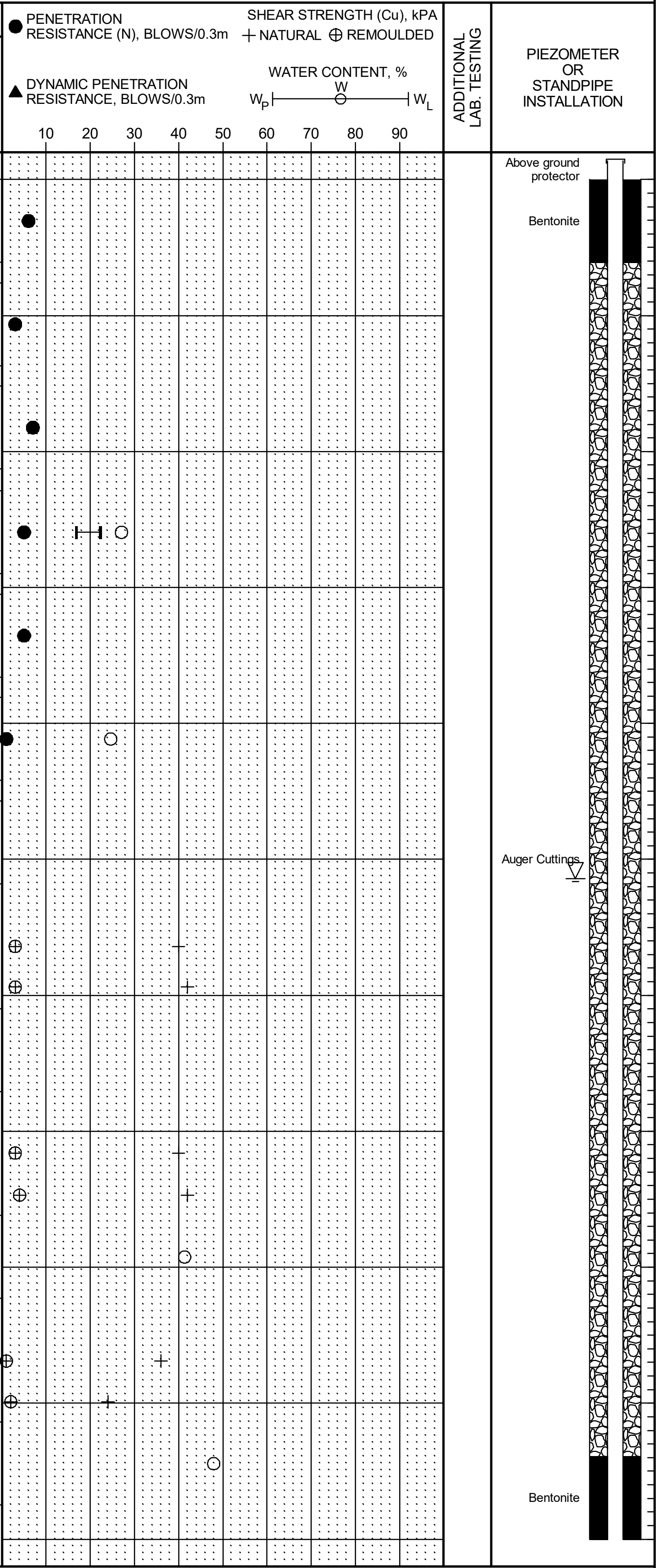
RECORD OF BOREHOLE 23-08

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 2
DATUM: CGVD28
BORING DATE: Apr 11 2023

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
0	Ground Surface	106.25				
	TOPSOIL					
	Stiff to very stiff, grey brown (CL) SILTY CLAY, trace to some sand, with silty sand seams (WEATHERED CRUST)	106.05 0.20	1	SS	460	6
1			2	SS	405	3
	Stiff to very stiff, grey brown, layered (CL-SM) SILTY CLAY and SILTY SAND (WEATHERED CRUST)	104.73 1.52	3	SS	610	7
2			4	SS	610	5
3	Loose, grey brown (SM) SILTY SAND, trace to some clay	103.20 3.05	5	SS	610	5
4	Firm, grey (CL) SILTY CLAY, some sand, with silty sand seams	102.44 3.81	6	SS	610	1
5	Firm to soft, grey (CL) SILTY CLAY, with silty sand seams	101.68 4.57	7	SS	610	WH
6			8	SS	610	WH
7–8			9	SS	610	PM
9	Stiff to firm, grey (CL) SILTY CLAY, with silty sand seams	97.11 9.14	10	SS	610	PM
10		96.25				



GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

LOGGED: A.N.
CHECKED: T.M./W.A.M.

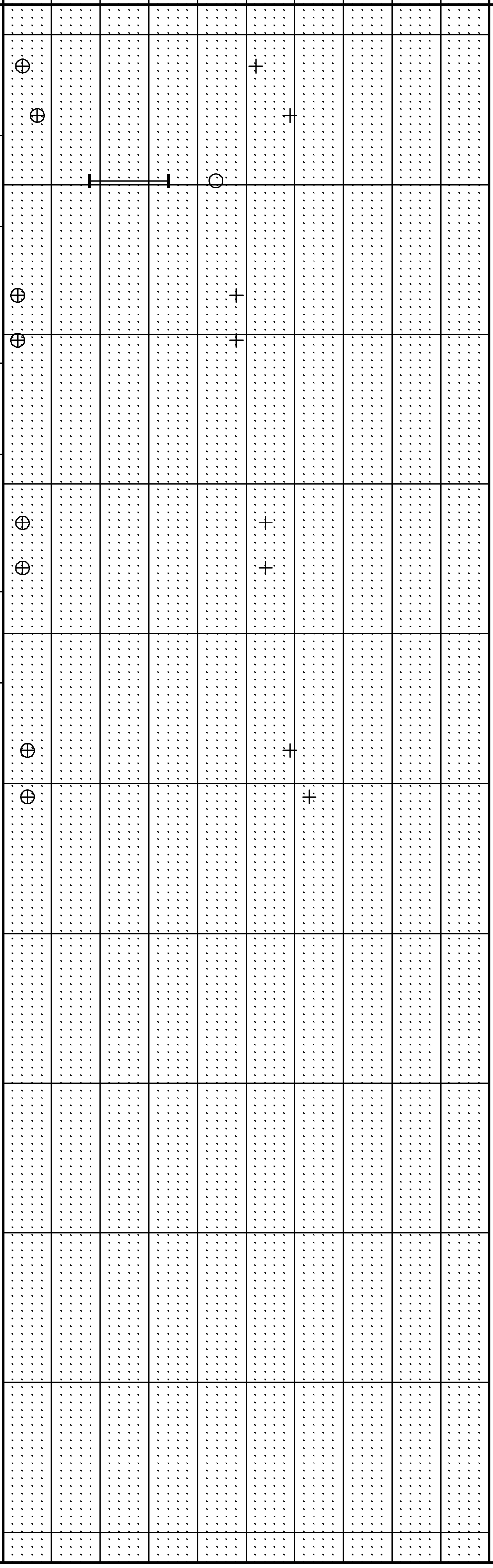
RECORD OF BOREHOLE 23-08

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 2 OF 2
DATUM: CGVD28
BORING DATE: Apr 11 2023

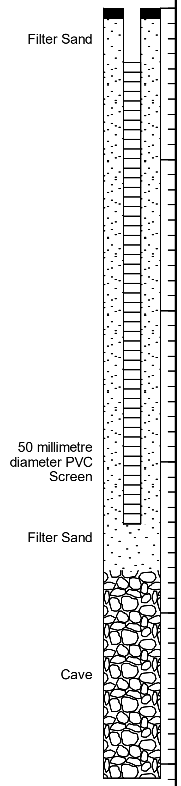
DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
10	Power Auger Hollow Stem Auger (210mm OD)	Stiff to firm, grey (CL) SILTY CLAY, with silty sand seams	▨	10.00				
11					11	SS	610	PM
12					12	SS	610	WH
14					13	SS	610	PM
15		End of borehole		91.16 15.09				

● 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



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/04/27	5.14 ∇	101.1

GEMTEC
Consulting Engineers and Scientists

LOGGED: A.N.
CHECKED: T.M./W.A.M.

RECORD OF BOREHOLE 23-09

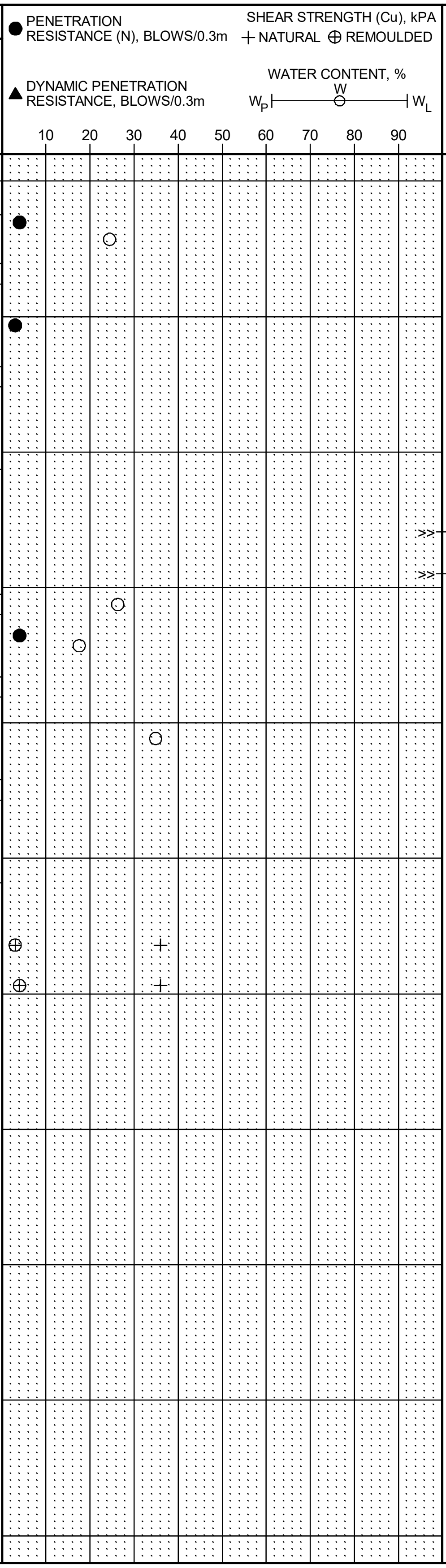
CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

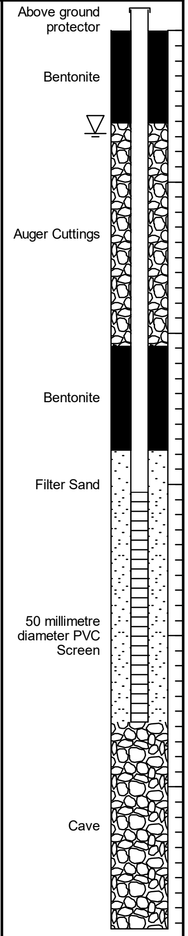
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 11 2023

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

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	103.83
TOPSOIL	103.58 0.25
Loose, grey brown (ML) sandy SILT, some clay	103.07 0.76
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace to some sand, with silty sand seams (WEATHERED CRUST)	100.78 3.05
Grey brown (SM) SILTY SAND	3.20
Very loose to loose, grey (SM) SILTY SAND, trace to some clay, with silty clay seams	100.02 3.81
Firm, grey (CL) SILTY CLAY, with silty sand seams	97.89 5.94
End of borehole	

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	380	4	MH
2	SS	480	3	
3	SS	610	WH	
4	SS	510	4	
5	SS	610	WH	
6	SS	610	WH	





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/04/27	0.68 ∇	103.2

GEMTEC Consulting Engineers and Scientists

LOGGED: A.N.

CHECKED: T.M./W.A.M.

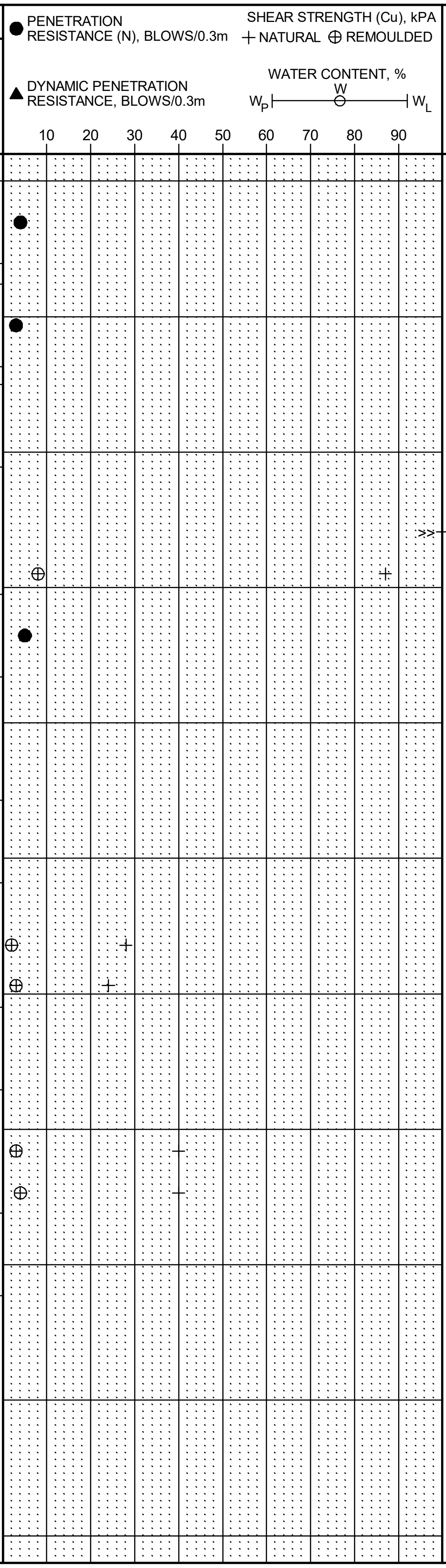
RECORD OF BOREHOLE 23-10

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 10 2023

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
Ground Surface	105.40				
TOPSOIL					
Loose, grey brown (SM) SILTY SAND, trace clay, with silty clay seams	105.15 0.25	1	SS	480	4
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace to some sand, with silty sand seams (WEATHERED CRUST)	104.64 0.76	2	SS	460	3
		3	SS	610	WH
Loose, grey (SM) SILTY SAND, with silty clay seams	102.35 3.05	4	SS	510	5
Firm to soft, grey (CL) SILTY CLAY, with silty sand seams	101.74 3.66	6	SS	610	WH
		7	SS	610	WH
		8	SS	610	WH
End of borehole	97.17 8.23				



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Borehole backfilled with auger cuttings

GEMTEC
Consulting Engineers and Scientists

LOGGED: A.N.

CHECKED: T.M./W.A.M.

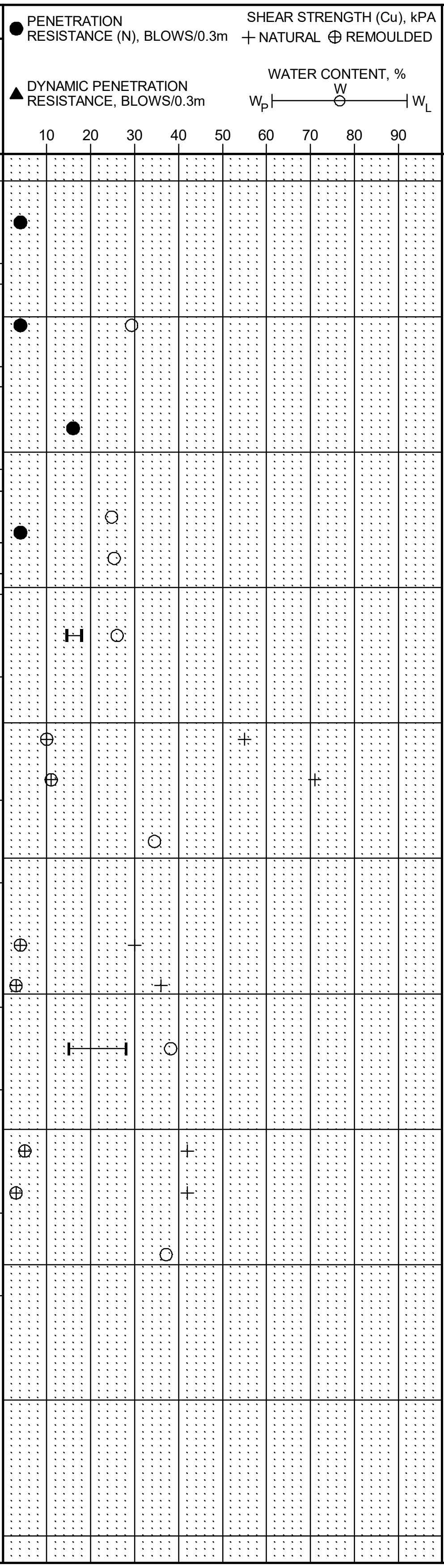
RECORD OF BOREHOLE 23-11

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 10 2023

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	109.33					
TOPSOIL						
Loose to compact, grey brown (ML-SM) sandy SILT to SILTY SAND, trace to some clay	109.08 0.25	1	SS	310	4	
		2	SS	380	4	MH
		3	SS	610	16	
Stiff to very stiff, grey brown (CL) SILTY CLAY, trace sand, with silty sand seams (WEATHERED CRUST)	107.04 2.29	4	SS	610	4	
Stiff, grey (ML) CLAYEY SILT, trace sand	106.66 2.67	5	SS	150	WH	
Firm, grey (CL) SILTY CLAY, trace sand, with silty sand seams	104.76 4.57	6	SS	610	WH	
		7	TP	530	PH	C
		8	SS	610	PM	
End of borehole	101.10 8.23					



DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

PIEZOMETER OR STANDPIPE INSTALLATION: Borehole backfilled with auger cuttings

GEMTEC
Consulting Engineers and Scientists

LOGGED: A.N.

CHECKED: T.M./W.A.M.

RECORD OF BOREHOLE 23-12

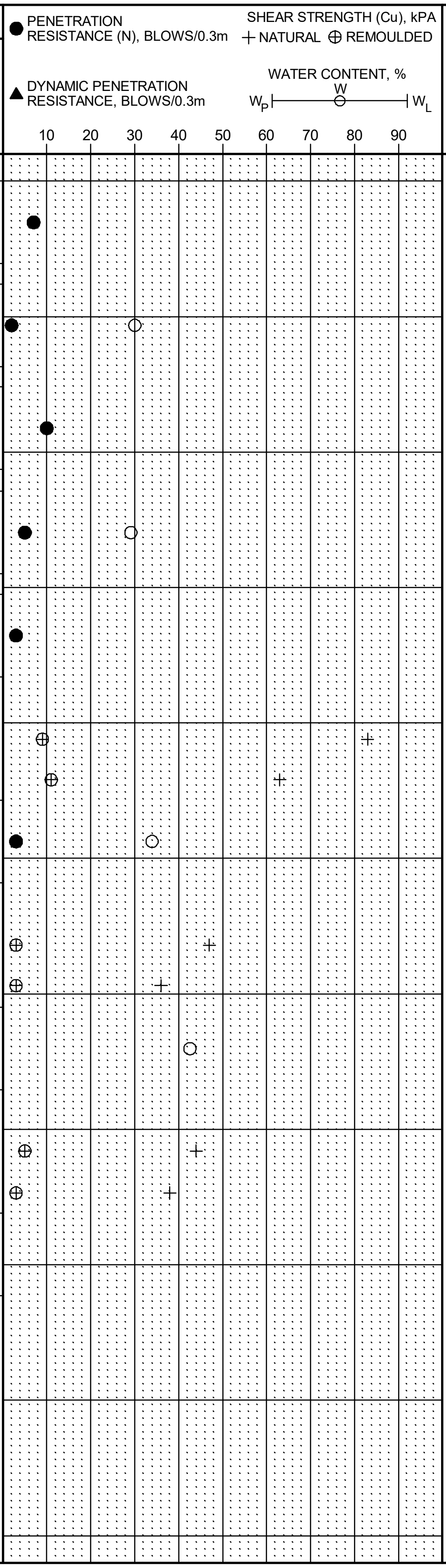
CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

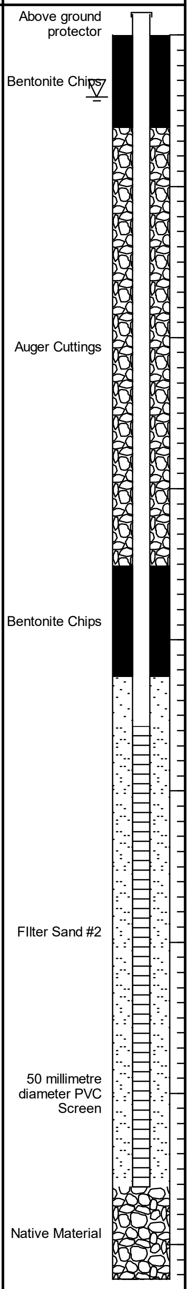
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Apr 10 2023

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	108.65
TOPSOIL	
Loose, grey brown (ML) sandy SILT, trace to some clay	108.40 0.25
Stiff to very stiff, grey brown, layered, (CL-ML) SILTY CLAY and sandy SILT (WEATHERED CRUST)	107.74 0.91
Stiff to very stiff, grey brown (CL) SILTY CLAY, with silty sand seams (WEATHERED CRUST)	107.13 1.52
Stiff, grey (CL) SILTY CLAY, with silty sand seams	105.60 3.05
Firm, grey, layered (CL-SM) SILTY CLAY, trace to some sand, and SILTY SAND	104.08 4.57
Firm, grey (CL) SILTY CLAY, with silty sand seams	103.47 5.18
End of borehole	100.42 8.23

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	460	7
2	SS	460	2
3	SS	610	10
4	SS	460	5
5	SS	610	3
6	SS	610	3
7	SS	610	WH
8	SS	610	WH





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/04/27	0.41 ∇	108.2

ADDITIONAL LAB. TESTING

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

LOGGED: A.N.
CHECKED: T.M./W.A.M.

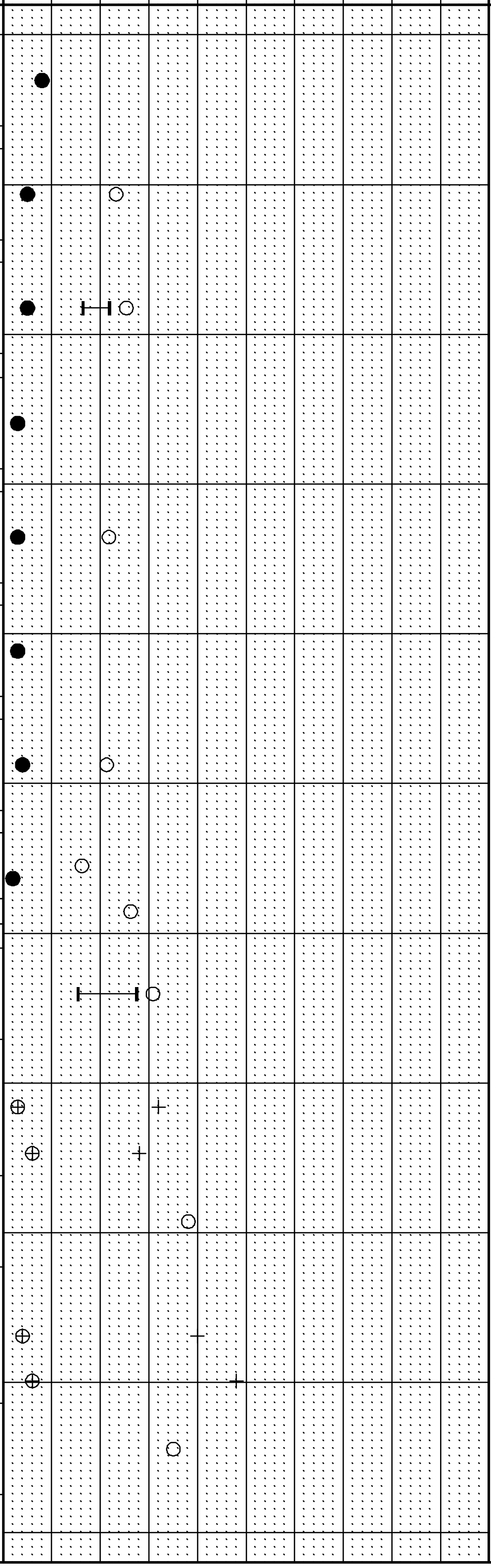
RECORD OF BOREHOLE 23-13

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 1 OF 2
DATUM: CGVD28
BORING DATE: Apr 13 2023

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

SOIL PROFILE		SAMPLES			
DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
Ground Surface	108.31				
TOPSOIL					
Loose, grey brown (SM) SILTY SAND, trace clay	108.11 0.20	1	SS	460	8
Stiff to very stiff, grey brown (CL-ML) SILTY CLAY to CLAYEY SILT, with sand seams (WEATHERED CRUST)	107.27 1.04	2	SS	330	5
		3	SS	460	5
		4	SS	480	3
Very loose, grey, layered (ML-SM) sandy SILT and SILTY SAND, trace to some clay	105.26 3.05	5	SS	610	3
		6	SS	610	3
		7	SS	610	4
Very loose, grey (SM) SILTY SAND, trace to some clay	102.98 5.33	8	SS	610	2
Firm, grey (CL) SILTY CLAY, trace sand, with silty sand seems	102.54 5.77	9	SS	610	PM
		10	SS	610	PM
		11	SS	610	PM
	98.31				



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Borehole backfilled with auger cuttings

GEMTEC
Consulting Engineers and Scientists

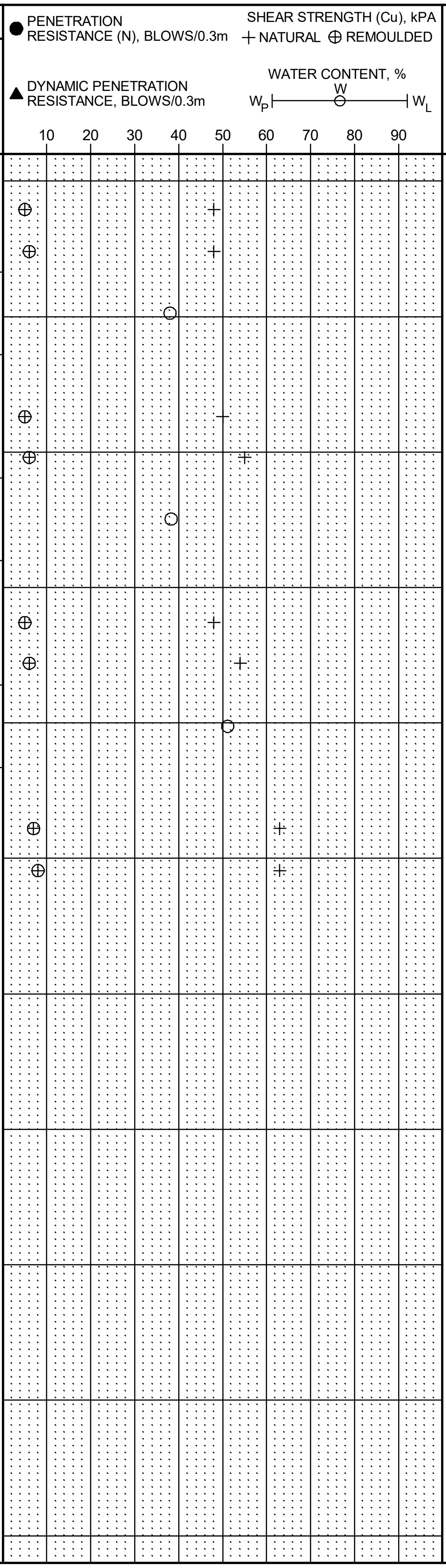
LOGGED: A.N.
CHECKED: T.M./W.A.M.

RECORD OF BOREHOLE 23-13

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

SHEET: 2 OF 2
DATUM: CGVD28
BORING DATE: Apr 13 2023

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Power Auger Hollow Stem Auger (210mm OD)	Firm, grey (CL) SILTY CLAY, trace sand, with silty sand seems	▨	10.00						Borehole backfilled with auger cuttings
11					12	SS	610	PM		
		Firm to stiff, grey (CL) SILTY CLAY	▨	97.03 11.28						
12					13	SS	610	WH		
13										
14					14	SS	610	WH		
15		End of borehole		93.22 15.09						
16										
17										
18										
19										
20										



GEMTEC
Consulting Engineers and Scientists

LOGGED: A.N.
CHECKED: T.M./W.A.M.

RECORD OF BOREHOLE 23-14

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

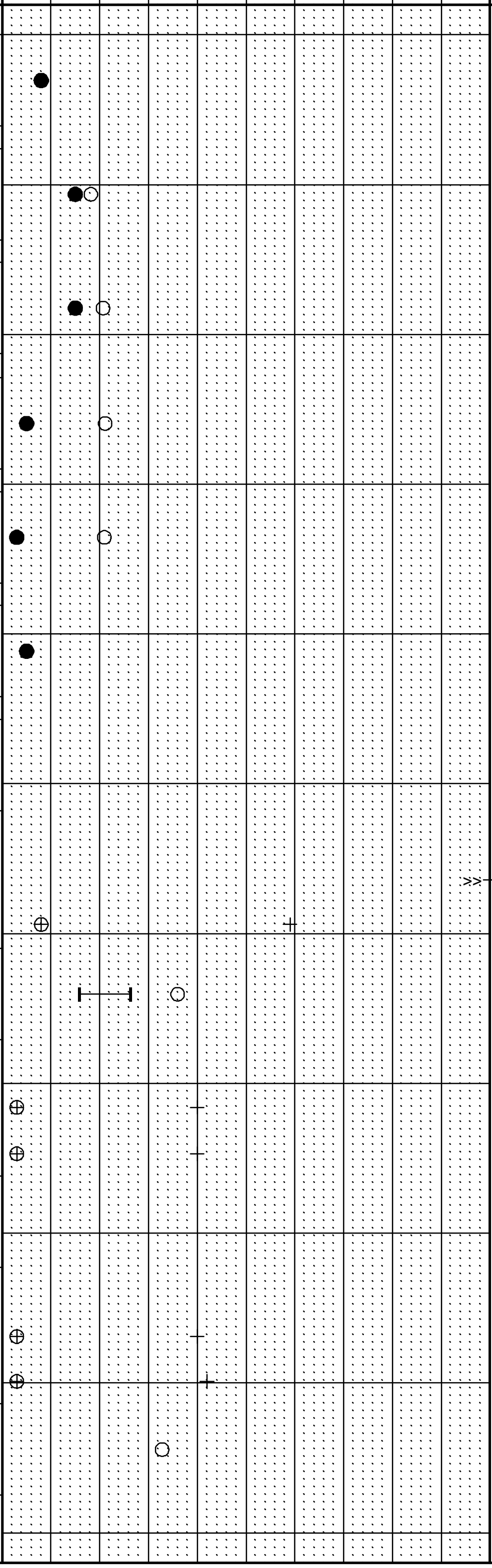
SHEET: 1 OF 2
DATUM: CGVD28
BORING DATE: Apr 14 2023

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

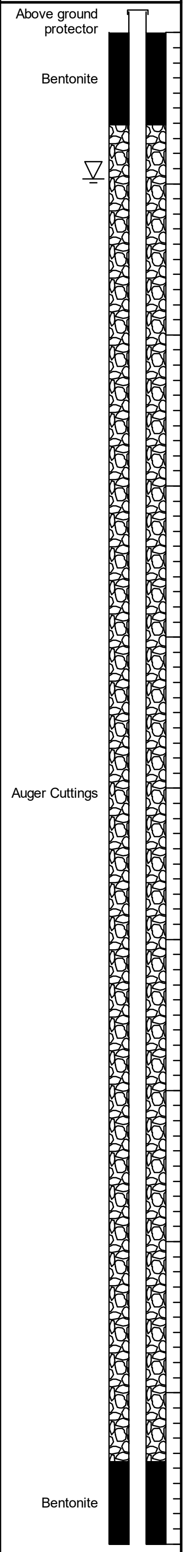
SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	110.01
TOPSOIL	
Compact to loose, grey brown (ML) sandy SILT, some clay	109.76 0.25
Loose to very loose, grey (SM) SILTY SAND	107.42 2.59
Firm, grey, layered (CL-SM) SILTY CLAY and SILTY SAND	106.10 3.91
Stiff to firm, grey (CL) SILTY CLAY, with silty sand seams	104.83 5.18
	100.01

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	430	8	
2	SS	380	15	MH
3	SS	405	15	
4	SS	310	5	
5	SS	25	3	
6	SS	510	5	
7	SS	405	WH	
8	SS	610	PM	
9	SS	610	PM	
10	SS	580	PM	

● PENETRATION RESISTANCE (N), BLOWS/0.3m
▲ DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED
WATER CONTENT, % W_P ⊢○⊣ W_L



PIEZOMETER OR STANDPIPE INSTALLATION



GEMTEC Consulting Engineers and Scientists

LOGGED: A.N.
CHECKED: T.M./W.A.M.

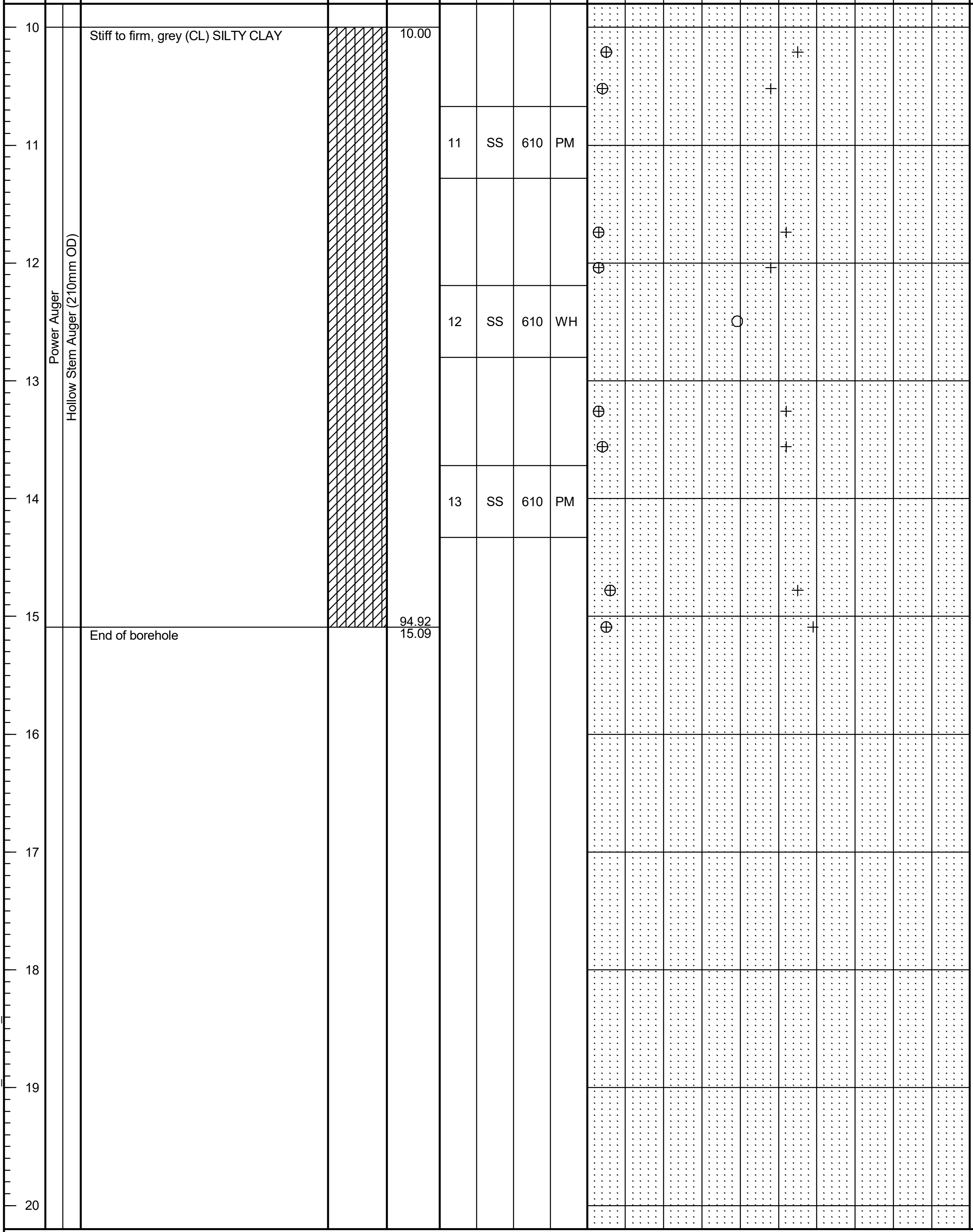
RECORD OF BOREHOLE 23-14

CLIENT: Novatech
PROJECT: Geotechnical Investigation, Proposed Phase 2 Business Park, Carp Aiport, Ottawa Ontario
JOB#: 100011.049
LOCATION: See Site Plan, Figure 1

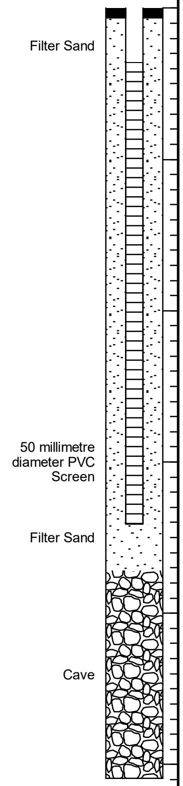
SHEET: 2 OF 2
DATUM: CGVD28
BORING DATE: Apr 14 2023

BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Power Auger Hollow Stem Auger (210mm OD)	Stiff to firm, grey (CL) SILTY CLAY	▨	10.00					
				11	SS	610	PM	
				12	SS	610	WH	
				13	SS	610	PM	
	End of borehole		94.92 15.09					

● 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



PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/04/27	0.97 ∇	109.0

LOGGED: A.N.
CHECKED: T.M./W.A.M.

GEMTEC Consulting Engineers and Scientists

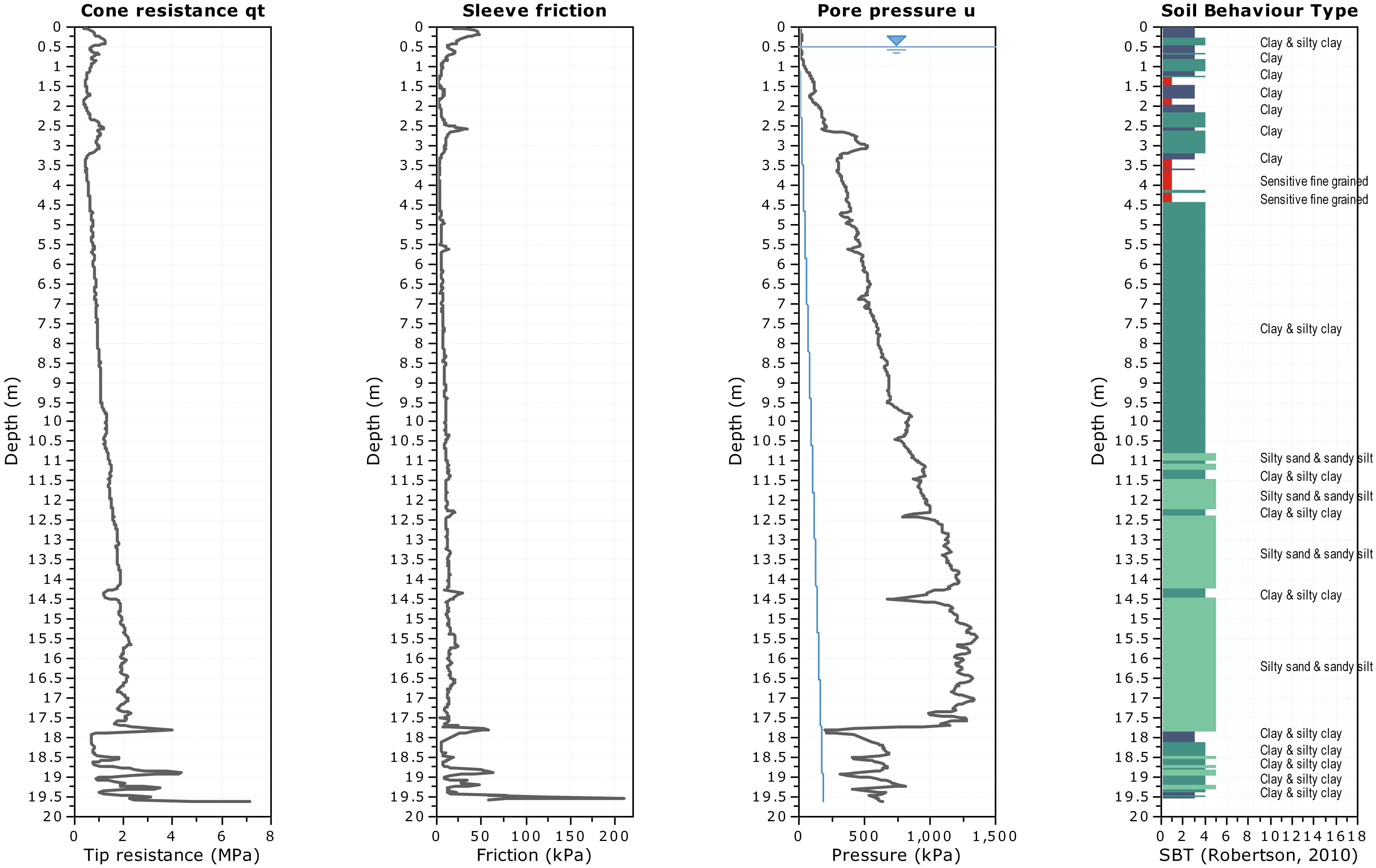
APPENDIX B

Record of Cone Penetration Testing
CPT 23-03, 23-13, and 23-14

CPT: 23-03

Project: **Geotechnical and Hydrogeological Investigation, Proposed Carp Airport Phase 2 Commercial Development**
Total depth: 19.63 m, Date: 2023-04-12

Location: **Part of Lots 13, 14 and 15. Concession 3, Ottawa, Ontario**
Surface Elevation: 109.10 m

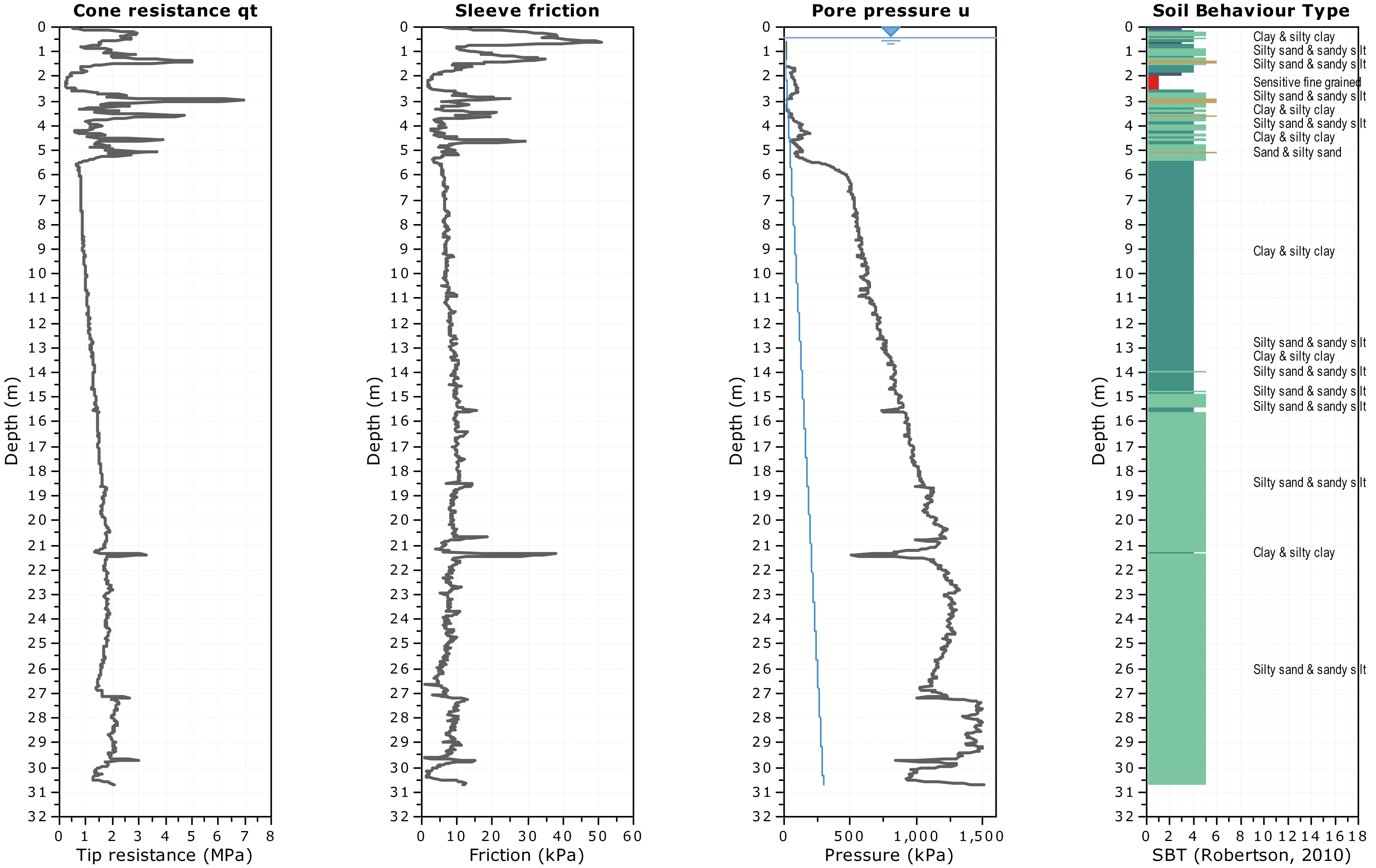


CPT: 23-13

Project: **Geotechnical and Hydrogeological Investigation, Proposed Carp Airport Phase 2 Commercial Development**

Location: **Part of Lots 13, 14 and 15. Concession 3, Ottawa, Ontario**

Total depth: 30.72 m, Date: 2023-04-12
Surface Elevation: 108.31 m



GEMTEC
Consulting Engineers and Scientists
www.gemtec.ca

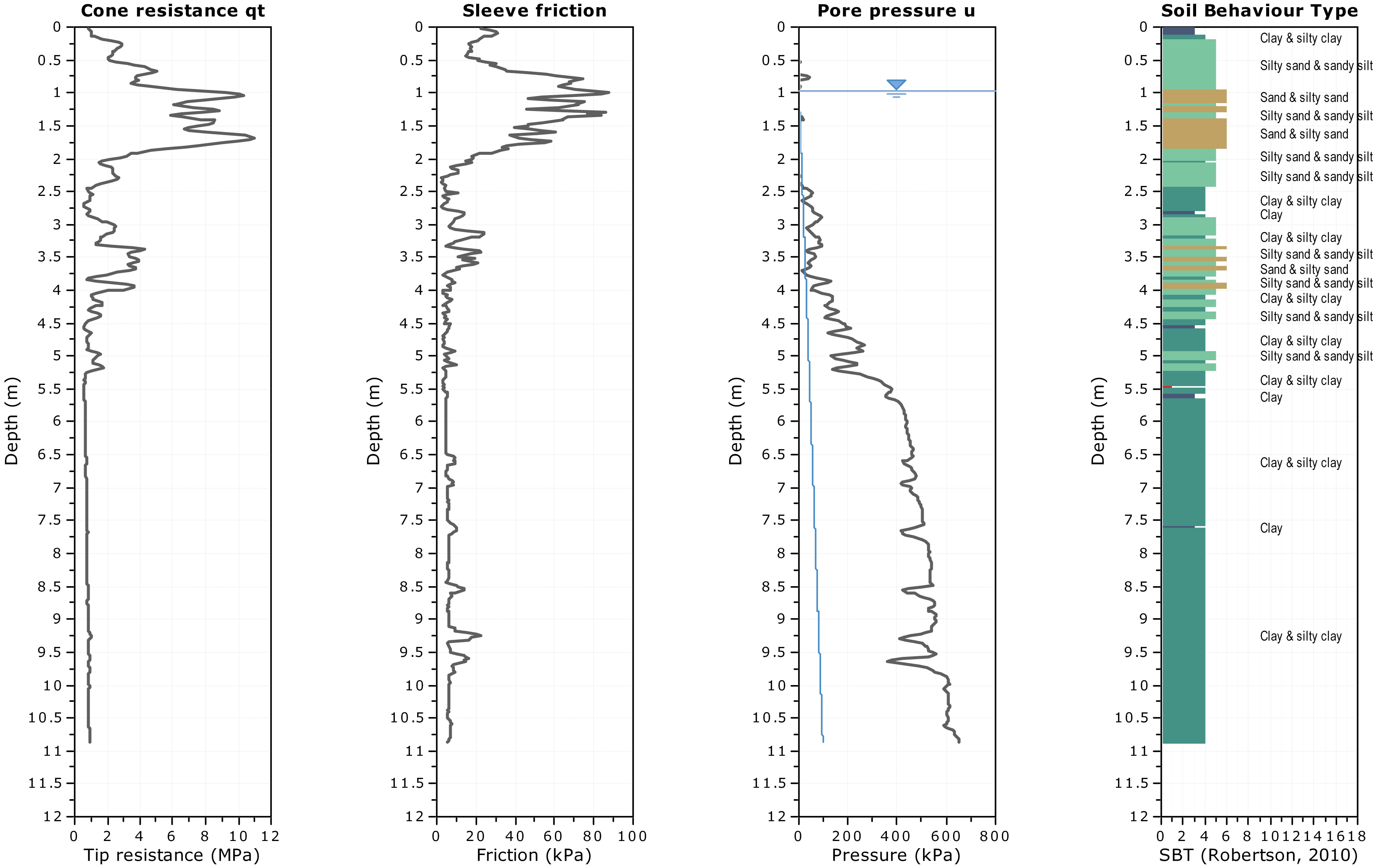
CPT: 23-14

Project: **Geotechnical and Hydrogeological Investigation, Proposed Carp Airport Phase 2 Commercial Development**

Total depth: 10.87 m, Date: 2023-04-13

Location: **Part of Lots 13, 14 and 15. Concession 3, Ottawa, Ontario**

Surface Elevation: 110.05 m



APPENDIX C

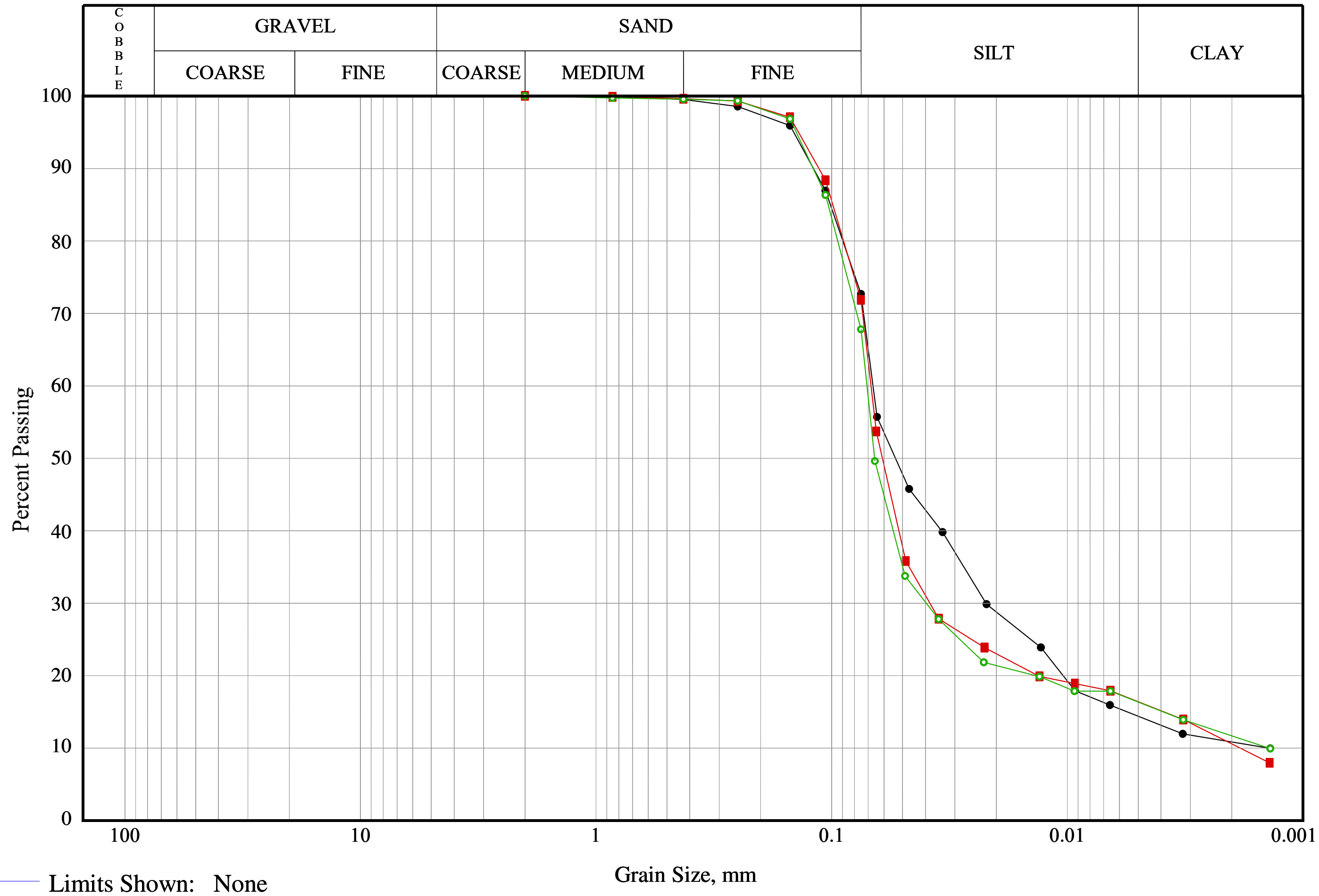
Laboratory Test Results

Report to: Novatech
GEMTEC Project: 100011.049 (November 4, 2025)

GEMTEC
Consulting Engineers
and Scientists

Client: Novatech
Project: Environmental Impact Statement and Slope Stability Ana
Project #: 100011049

Soils Grading Chart (LS-702/ ASTM D-422)

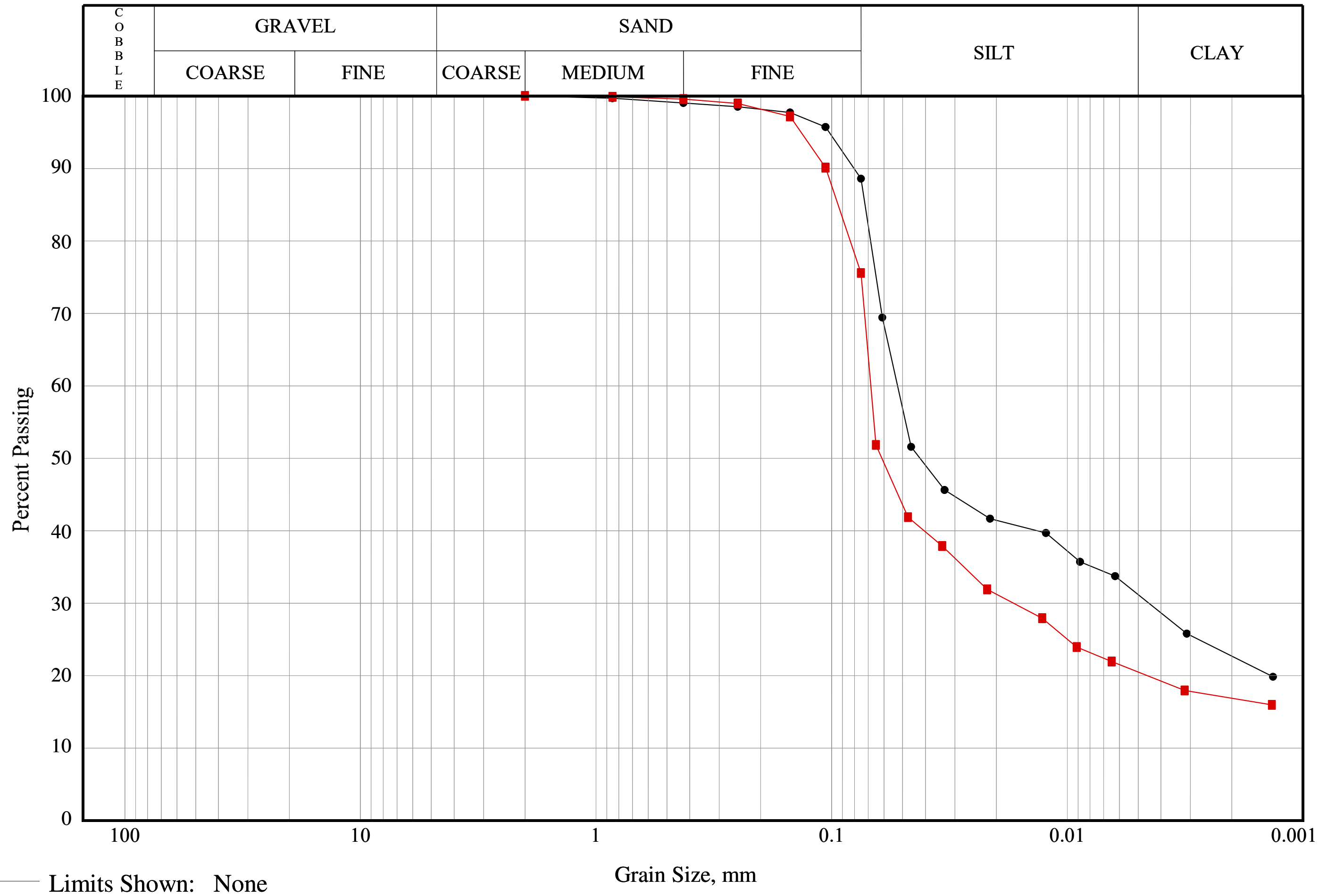


Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●	Sandy SILT, some clay	23-09	SA 01B	0.25-0.61	0.0	27.3	58.3	14.4
■	Sandy SILT, some clay	23-11	SA 02	0.76-1.37	0.0	28.2	55.4	16.4
○	Sandy SILT, some clay	23-14	SA 02	0.76-1.37	0.0	32.2	51.4	16.4

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
●	Sandy silt , some clay	N/A	0.00	0.01	0.02	0.05	0.07	0.10	58.3
■	Sandy silt , some clay	N/A	0.00	0.00	0.04	0.06	0.07	0.10	55.4
○	Sandy silt , some clay	N/A	0.00	0.00	0.04	0.07	0.07	0.10	51.4

GEMTEC Consulting Engineers and Scientists		Soils Grading Chart (LS-702/ ASTM D-422)
	Client: Novatech	
	Project: Environmental Impact Statement and Slope Stability Ana	
	Project #: 100011049	



Limits Shown: None

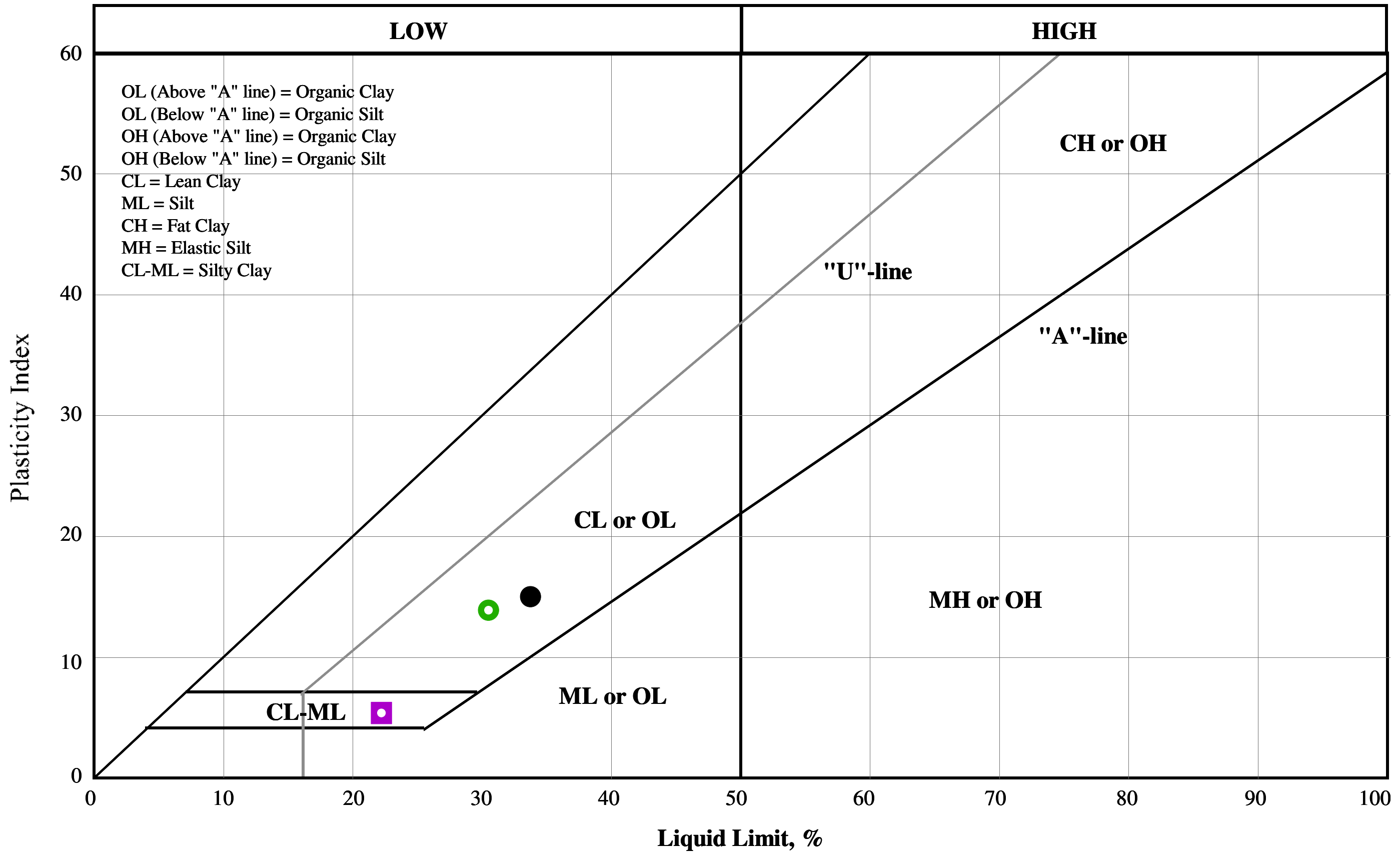
Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	WEATHERED CRUST	23-01	SA 02	0.76-1.37	0.0	11.4	57.4	31.2
—■—	WEATHERED CRUST	23-07	SA 02	0.76-1.37	0.0	24.4	55.1	20.5

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—	Clayey silt , some sand	N/A	---	---	0.00	0.04	0.05	0.07	57.4
—■—	Sandy clayey silt	N/A	---	---	0.02	0.06	0.07	0.09	55.1

GEMTEC
Consulting Engineers and Scientists

Client:	Novatech
Project:	Environmental Impact Statement and Slope Stability Ana
Project #:	100011049

Plasticity Chart (D4318)



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	23-05	SA 06	4.57-5.18	33.7	18.8	15.0	☐	44.48
■	23-06	SA 03	1.52-2.13				☑	29.43
○	23-06	SA 07	6.10-6.70	30.5	16.6	13.9	☐	38.42
▣	23-08	SA 04	2.28-2.89	22.2	16.8	5.4	☐	27.06

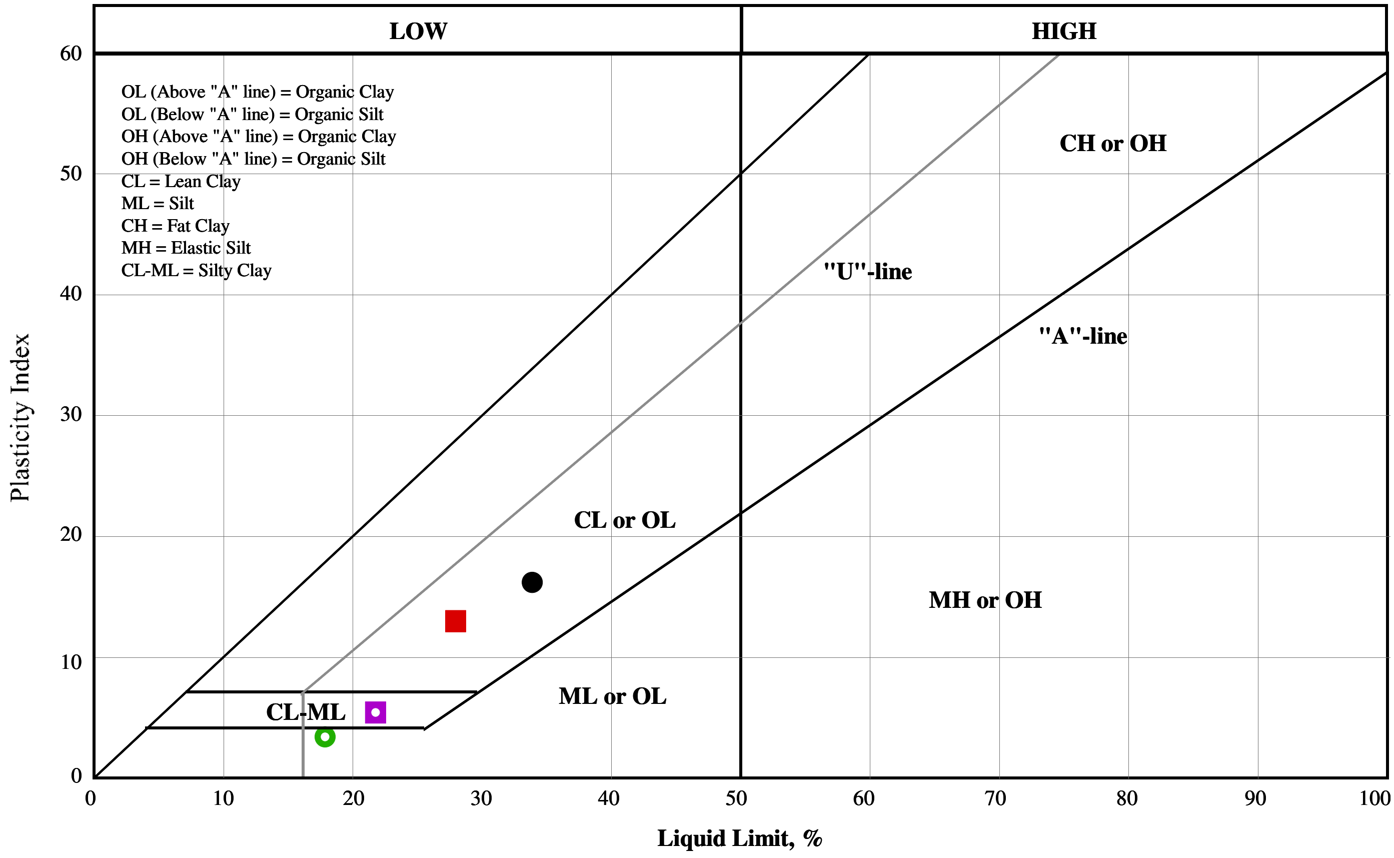
Note: More information available upon request

CCIL

GEMTEC
Consulting Engineers and Scientists

Client: Novatech
Project: Environmental Impact Statement and Slope Stability Ana
Project #: 100011049

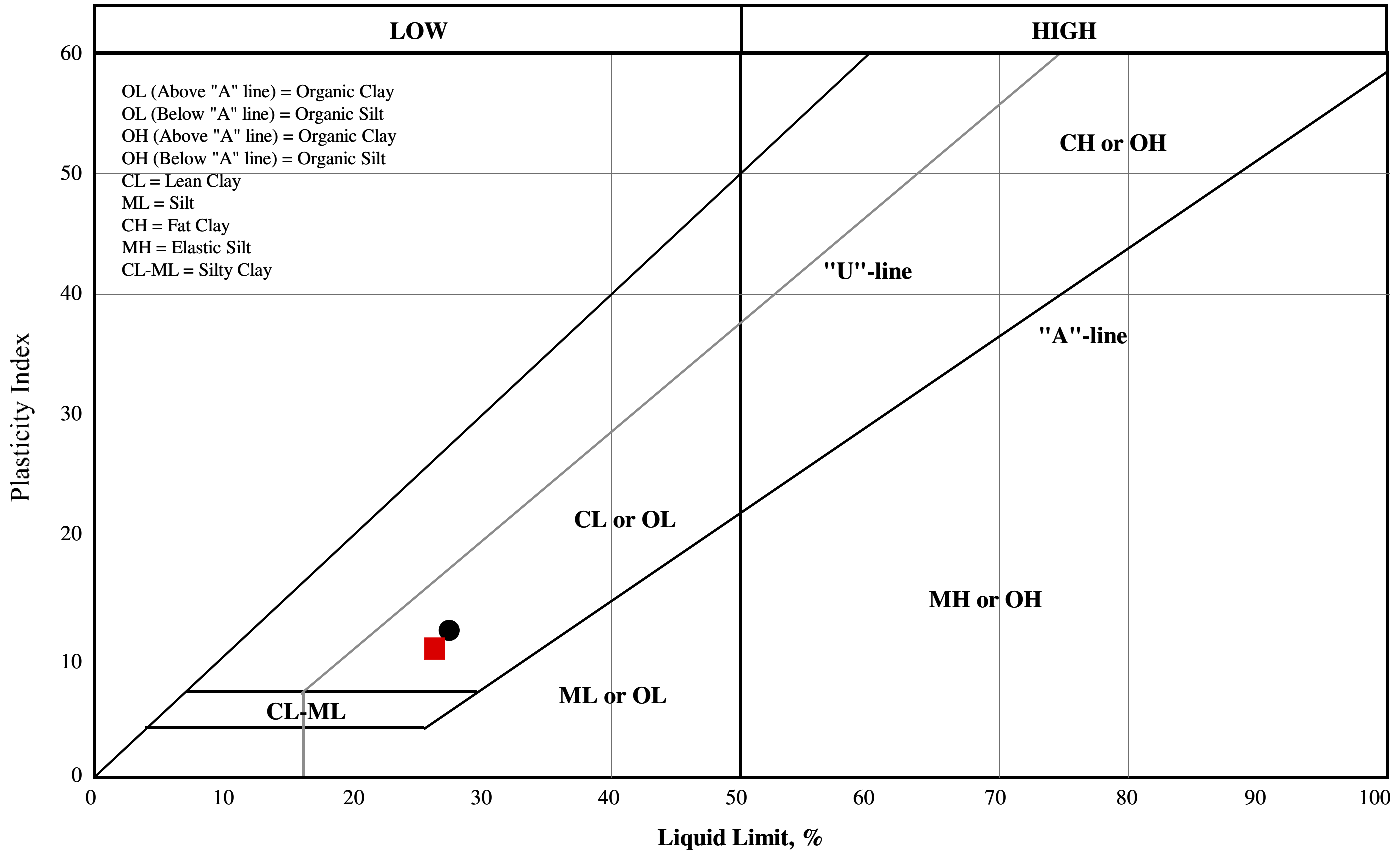
Plasticity Chart (D4318)



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	23-08	SA 11	10.67-11.28	33.9	17.7	16.2	☐	43.75
■	23-11	7 (ST)	6.10-6.70	28.0	15.0	13.0	☐	38.20
○	23-11	SA 05	3.05-3.66	17.9	14.5	3.4	☐	26.01
▣	23-13	SA 03	1.52-2.13	21.8	16.4	5.4	☐	25.37

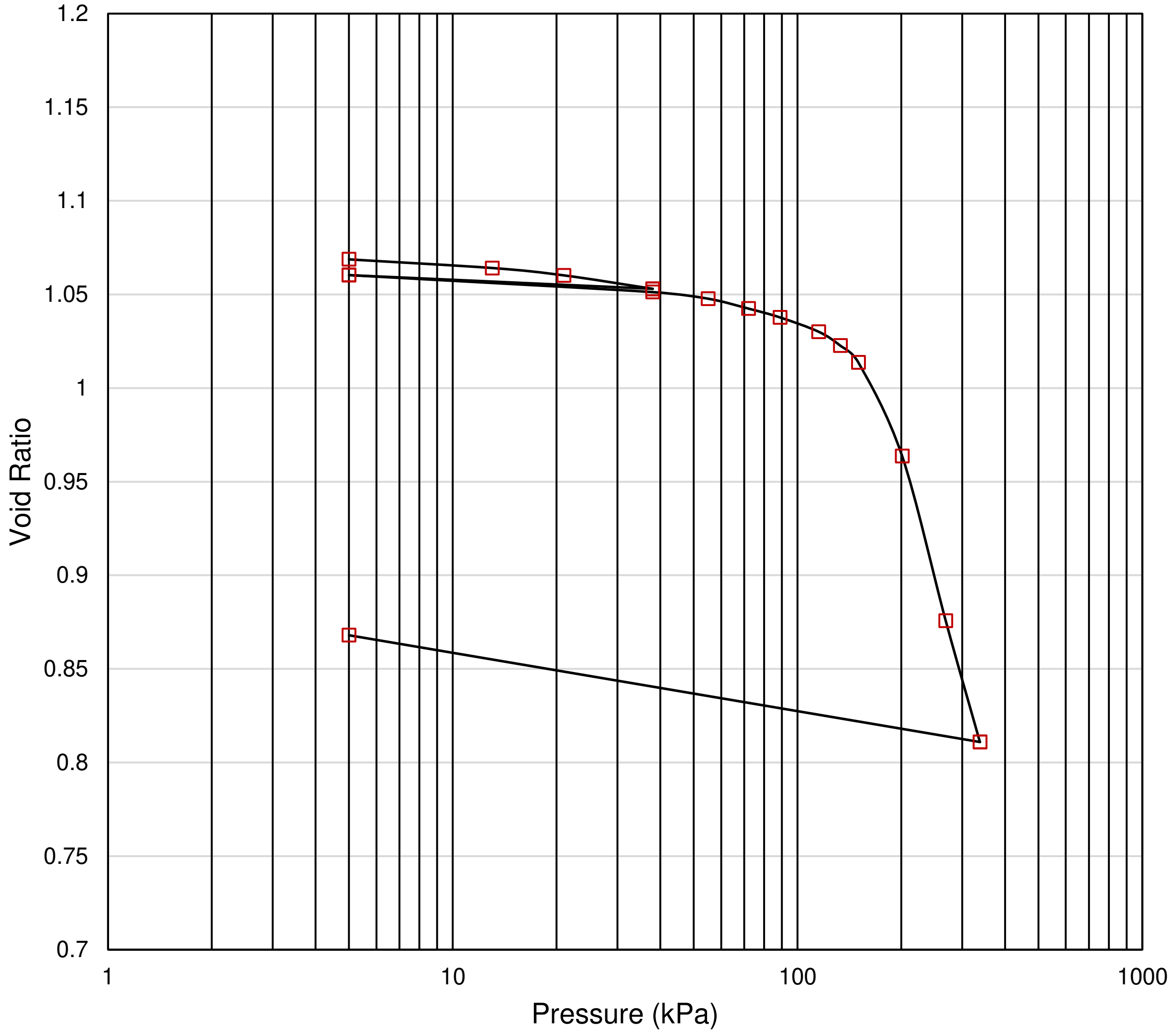
Note: More information available upon request

GEMTEC Consulting Engineers and Scientists		
Client:	Novatech	Plasticity Chart (D4318)
Project:	Environmental Impact Statement and Slope Stability Ana	
Project #:	100011049	



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	23-13	SA 09	6.10-6.70	27.4	15.3	12.2	☐	30.84
🟥	23-14	SA 08	6.10-6.70	26.3	15.7	10.7	☐	35.96

Note: More information available upon request



Borehole/Sample Number	23-11 / 7
Sample Depth (m)	6.10
Initial Water Content (%)	41
Existing Effective Overburden Pressure (kPa)	65
Probable Preconsolidation Pressure (kPa)	170
Compression Index (Cc)	0.68
Recompression Index (Cr)	0.008

CONSOLIDATION TEST RESULTS

GEMTEC
Consulting Engineers
and Scientists

Date:	23/04/04	SILTY CLAY	
Entry:	KN	CARP AIRPORT DEVELOPMENT	
Check:	KS	PROJECT NO.	100011.049
Review:	WAM	FIGURE NO.	

GEMTEC
www.gemtec.ca

Specific Gravity

LS-705, ASTM D854

Specimen No.	**G1**	**G2**	**G3**
Pycnometer Reference No.	173	161	206
Mass of Pycnometer (M_f)	32.72	32.65	32.48
Mass of Dry Specimen + Pycnometer (M_s)	52.72	52.66	52.49
Mass of Dry Soil ($M_s - M_f = M_o$)	20.00	20.01	20.01
Mass of Pycnometer + Water (M_a)	82.6	82.5	82.3
Mass of Pycnometer + Specimen + Water (M_B)	95.24	95.17	94.95
Mass of Water Displaced = $[(M_a + M_o) - M_b]$	7.36	7.34	7.36
Temperature, T_x, of the Content	26.5	26.5	26.5
Specific Gravity, $G = M_o/[M_o + (M_a - M_b)]$	2.717	2.726	2.719
Mean Specific Gravity at Temperature T_x, $G_{avg} = (G1+G2+G3)/3$			2.721
Specific Gravity at 20°C, $G_s = K\ (G_{avg})$		2.716	

Removal of Entrapped Air By

A) Vacuum	
B) Boiling	X

Project No.: 100011.049	**Tested By: K.Neil**
Project Name:	**Checked By: K.Neil**
Date Tested: May 8, 2023	**Sample No: 23-11 (7ST)**
Remarks:	**Depth: 20'-22'**

APPENDIX D

Borehole and Test Pit Logs – Previous Investigation

Boreholes 22-11 to 35-11
Test Pits 2, 4, 6, 11, 18, and 19

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotehnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

FILE NO. **PG2450**

REMARKS

HOLE NO. **BH26-11**

BORINGS BY CME 55 Power Auger **DATE** 19 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	113.14		
25mm Asphaltic concrete **FILL:** Crushed stone 0.60		AU	1						
FILL: Brown silty sand, trace crushed stone 1.22		SS	2	50	9	1	112.14		
FILL: Brown silty clay, trace sand and organics 1.45									
Compact, brown **SILTY SAND,** some clay		SS	3	42	10	2	111.14		▽
- grey and fine by 2.2m depth 2.97		SS	4	50	13				
End of Borehole (GWL @ 1.8m depth based on field observations)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

FILE NO. **PG2450**

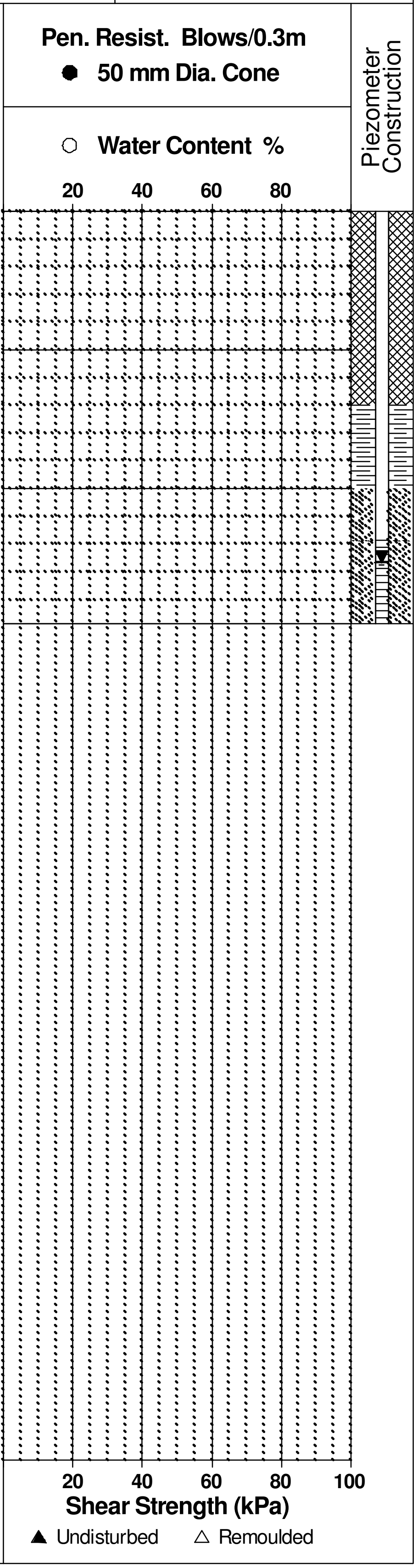
REMARKS

HOLE NO. **BH27-11**

BORINGS BY CME 55 Power Auger

DATE 19 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	112.83
25mm Asphaltic concrete over crushed stone with some sand (to 0.60)		AU	1				
Loose to compact, brown **SILTY FINE SAND** - trace clay from 0.6 to 1.4m depth - grey by 2.2m depth (to 2.97)		SS	2	58	9	1	111.83
		SS	3	42	19	2	110.83
		SS	4	50			
End of Borehole (GWL @ 2.53m-Sept. 21/11)							



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotehnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

FILE NO. **PG2450**

REMARKS

HOLE NO. **BH28-11**

BORINGS BY CME 55 Power Auger

DATE 19 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	113.49	
25mm Asphaltic concrete over brown silty sand with crushed stone — 0.46		AU	1					
Compact, brown **SILTY FINE SAND**		SS	2	58	15	1	112.49	
		SS	3	67	23	2	111.49	∇
- grey by 2.2m depth		SS	4	58	11			
2.90 — End of Borehole (GWL @ 2.1m depth based on field observations)								

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**
20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotehnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

FILE NO. PG2450

REMARKS

HOLE NO. BH29-11

BORINGS BY CME 55 Power Auger

DATE 19 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	112.93		
FILL: Crushed stone with silty sand 0.60		AU	1						
FILL: Brown silty sand, trace clay and gravel 1.22		SS	2	17	5	1	111.93		
Compact, brown **SILTY FINE SAND** - trace clay from 1.2 to 2.2m depth - loose and grey by 2.2m depth 2.97		SS	3	67	11	2	110.93		∇
		SS	4	50	7				
End of Borehole (GWL @ 1.8m depth based on field observations)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

FILE NO. **PG2450**

REMARKS

HOLE NO. **BH30-11**

BORINGS BY CME 55 Power Auger

DATE 19 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	112.00		
FILL: Crushed stone 0.60	▨	AU	1						
Compact, red-brown **SILTY FINE SAND**		SS	2	33	19	1	111.00		
- brown by 1.4m depth		SS	3	67	16	2	110.00		▽
- trace clay by 2.2m depth 2.97		SS	4	50	11				
End of Borehole (GWL @ 1.8m depth based on field observations)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup

Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotehnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

FILE NO. **PG2450**

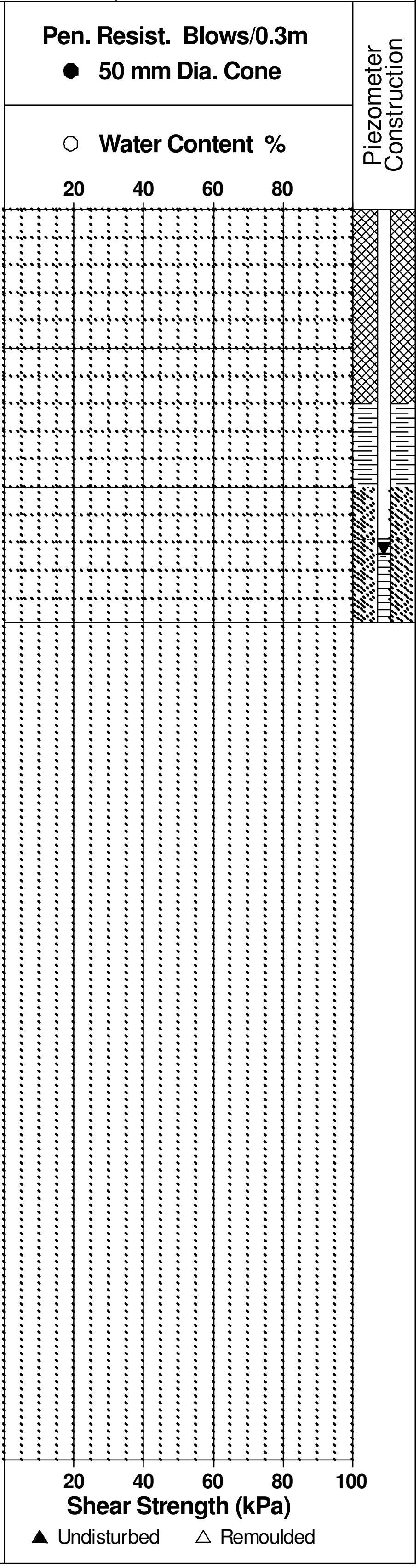
REMARKS

HOLE NO. **BH31-11**

BORINGS BY CME 55 Power Auger

DATE 19 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	110.88
FILL: Brown silty sand with crushed stone 0.56		AU	1				
Compact to loose, red-brown **SILTY FINE SAND**		SS	2	67	22	1	109.88
- brown with trace to some clay by 1.4m depth		SS	3	75	15	2	108.88
2.97		SS	4	75	4		
End of Borehole (GWL @ 2.48m-Sept. 21/11)							



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

FILE NO. **PG2450**

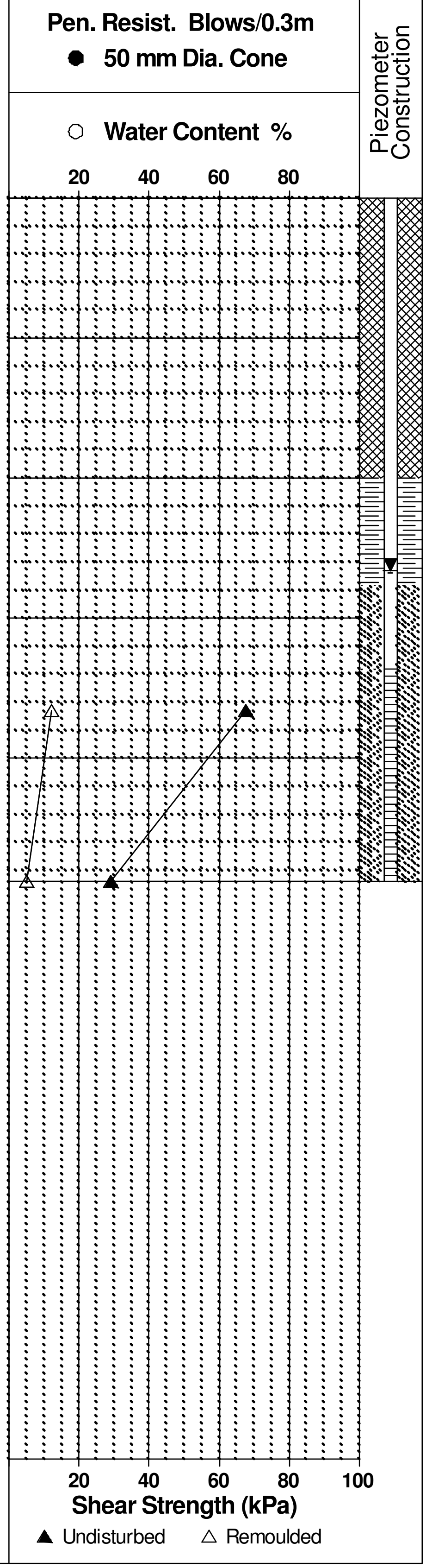
REMARKS

HOLE NO. **BH32-11**

BORINGS BY CME 55 Power Auger

DATE 20 September 2011

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	110.29
TOPSOIL 0.25	SS	1	83	14		
	SS	2	100	9	1	109.29
Stiff, brown **SILTY CLAY** with sand	SS	3	0	5	2	108.29
	SS	4	100	4		
					3	107.29
- firm and grey by 3.7m depth						
	SS	5	42	1	4	106.29
4.88						
End of Borehole (GWL @ 2.66m-Sept. 27/11)						



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

REMARKS

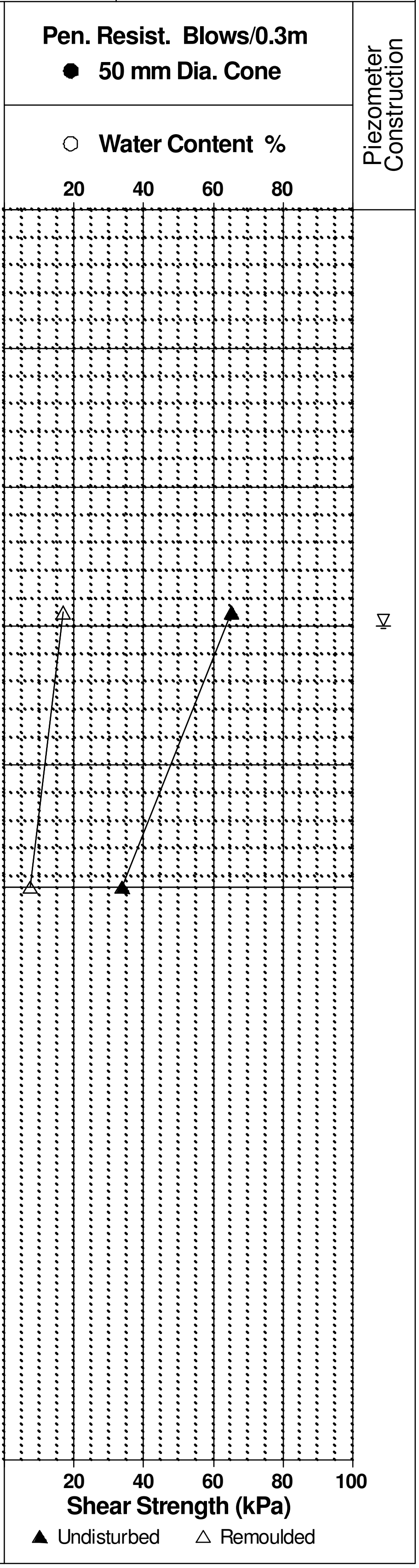
BORINGS BY CME 55 Power Auger

DATE 15 September 2011

FILE NO. PG2450

HOLE NO. BH33-11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	108.46
TOPSOIL 0.25		SS	1	12	9		
Stiff, brown **SILTY CLAY,** some sand - firm and grey by 3.0m depth		SS	2	50	9	1	107.46
		SS	3	42	3	2	106.46
						3	105.46
		SS	4	50	4		
		SS	5	100	2	4	104.46
4.88 End of Borehole (GWL @ 3.0m depth based on field observations)							



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Carp Airport Servicing and Residential Development
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Limited.

FILE NO. PG2450

REMARKS

HOLE NO. BH34-11

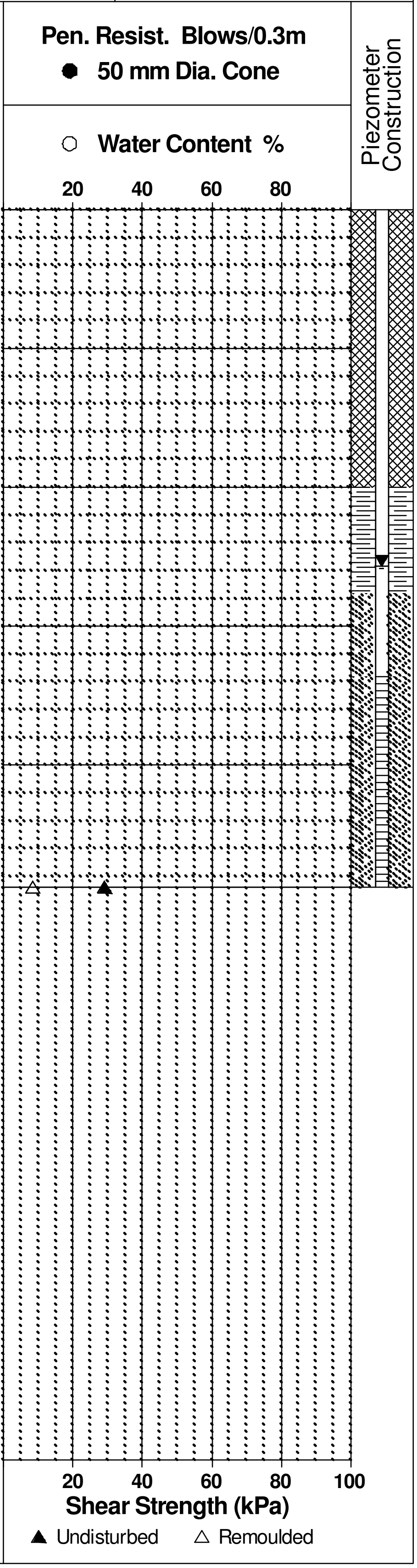
BORINGS BY CME 55 Power Auger

DATE 15 August 2011

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.25
Stiff, brown **SILTY CLAY,** some sand	3.65
Firm, grey **SILTY CLAY**	4.88
End of Borehole (GWL @ 2.57m-Sept. 27/11)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	42	7
SS	2	25	5
SS	3	42	6
SS	4	58	7
SS	5	25	4
SS	6	100	1

DEPTH (m)	ELEV. (m)
0	105.16
1	104.16
2	103.16
3	102.16
4	101.16



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Residential/Business Park Development, Carp Airport
Ottawa, Ontario

DATUM Geodetic, as provided by Novatech Engineering Consultants Ltd.

REMARKS

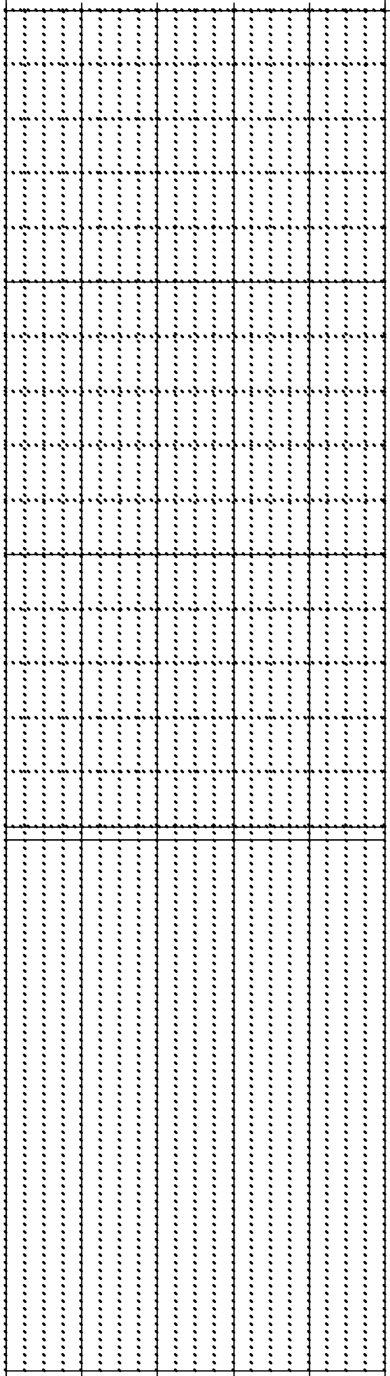
BORINGS BY Backhoe

DATE 8 Dec 05

FILE NO. PG0739

HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	111.50
TOPSOIL 0.38							
Compact, mottled brown and grey **SILTY** fine **SAND**							
- grey by 1.1m depth		G	1			1	110.50
- seashells and occasional gravel and cobbles by 1.7m depth						2	109.50
2.74		G	2				
Loose to compact, dark grey **SANDY SILT/SILTY** fine **SAND** with seashells 3.05						3	108.50
End of Test pit (TP dry upon completion)							



Piezometer Construction

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Residential/Business Park Development, Carp Airport
Ottawa, Ontario

DATUM Geodetic, as provided by Novatech Engineering Consultants Ltd.

FILE NO. PG0739

REMARKS

HOLE NO. TP 4

BORINGS BY Backhoe

DATE 8 Dec 05

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %
20 40 60 80

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	108.15	
TOPSOIL 0.23								
Compact, mottled brown and grey **SILTY** fine **SAND/SANDY SILT** with clay		G	1					
						1	107.15	
- compact to dense by 1.1m depth								▽
		G	2					
- some seashells by 1.5m depth								
						2	106.15	
- grey by 2.7m depth		G	3					
3.05								
						3	105.15	
Loose, dark grey **SANDY SILT** with clay 3.81								
End of Test Pit (Slow water infiltration @ 1.4m depth)								

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Residential/Business Park Development, Carp Airport
Ottawa, Ontario

DATUM Geodetic, as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Backhoe

DATE 8 Dec 05

FILE NO. PG0739

HOLE NO. TP 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	110.65	
TOPSOIL	■							
0.56								
Stiff, mottled brown and grey **SILTY CLAY** with sand - grey by 0.9m depth	▨	G	1			1	109.65	
	▨							∇
- firm by 1.8m depth	▨					2	108.65	
	▨					3	107.65	
	▨	G	2					
3.66								
End of Test Pit (Moderate water infiltration @ 1.4m depth)								

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Residential/Business Park Development, Carp Airport
Ottawa, Ontario

DATUM Geodetic, as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Backhoe

DATE 14 Dec 05

FILE NO. PG0739

HOLE NO. TP11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	115.75		
TOPSOIL 0.28									
Loose to compact, brown medium **SAND** 0.71									▽
Compact, grey **SILTY** fine **SAND/SANDY SILT** with seashells, some clay 2.13		G	1						
						1	114.75		
						2	113.75		
Soft, grey stratified layers of **SILTY CLAY, SILTY** fine **SAND** and **SILT** 3.50						3	112.75		
		G	2						
End of Test Pit (Slow water infiltration @ 0.7m depth)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Residential/Business Park Development, Carp Airport
Ottawa, Ontario

DATUM Geodetic, as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Backhoe

DATE 8 Dec 05

FILE NO. PG0739

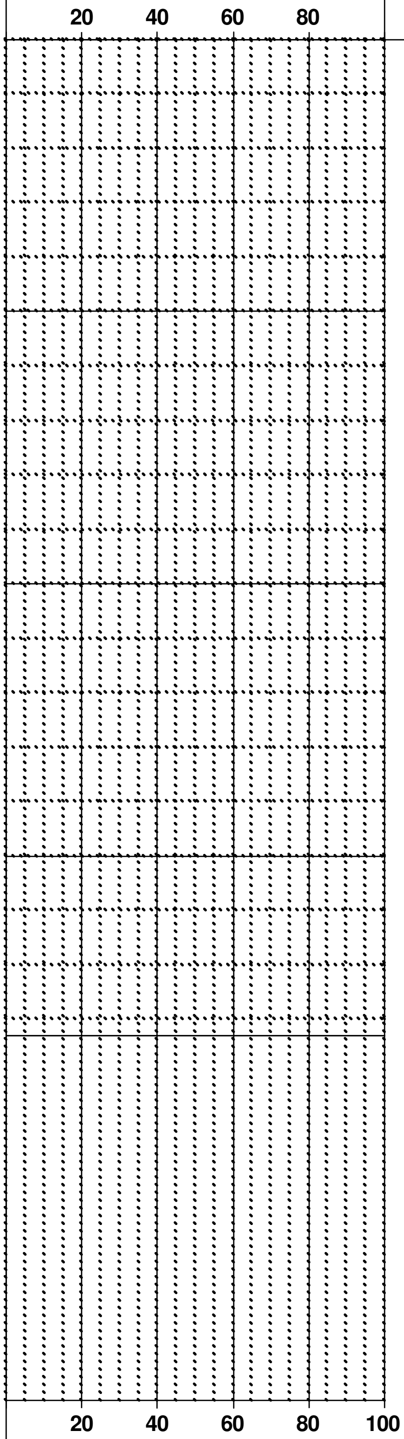
HOLE NO. TP18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	106.00
TOPSOIL 0.22							
						1	105.00
Compact, mottled brown and grey **SILTY** fine **SAND** with clay		G	1				
- grey by 2.0m depth						2	104.00
- some seashells by 2.4m depth							
						3	103.00
3.66							
End of Test Pit (TP dry upon completion)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Residential/Business Park Development, Carp Airport
Ottawa, Ontario

DATUM Geodetic, as provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Backhoe

DATE 8 Dec 05

FILE NO. PG0739

HOLE NO. TP19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	104.60		
TOPSOIL 0.25									
Compact, mottled brown and grey **SANDY SILT/SILTY** fine **SAND** with clay - grey by 0.9m depth						1	103.60		
		G	1						
						2	102.60		
- loose to compact by 2.4m depth									
- some seashells by 3.0m depth						3	101.60		
3.35									
End of Test Pit (TP dry upon completion)									

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

APPENDIX E

Chemical Analysis of Soil Samples
Samples Relating to Corrosion
(Paracel Laboratories Ltd. Order No. 2319294)

Certificate of Analysis

Client: GEMTEC Consulting Engineers and Scientists Limited

Client PO:

Report Date: 28-Apr-2023

Order Date: 26-Apr-2023

Project Description: 100011.049

	Client ID:	BH23-03/SA3 - Carp Airport	BH23-08/SA9 - Carp Airport	-	-
	Sample Date:	04-Apr-23 09:00	11-Apr-23 09:00	-	-
	Sample ID:	2317229-01	2317229-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	75.1	68.2	-	-
General Inorganics					
Conductivity	5 uS/cm	106	452	-	-
pH	0.05 pH Units	7.53	7.57	-	-
Resistivity	0.1 Ohm.m	94.6	22.1	-	-
Anions					
Chloride	10 ug/g dry	<10	<10	-	-
Sulphate	10 ug/g dry	16	174	-	-

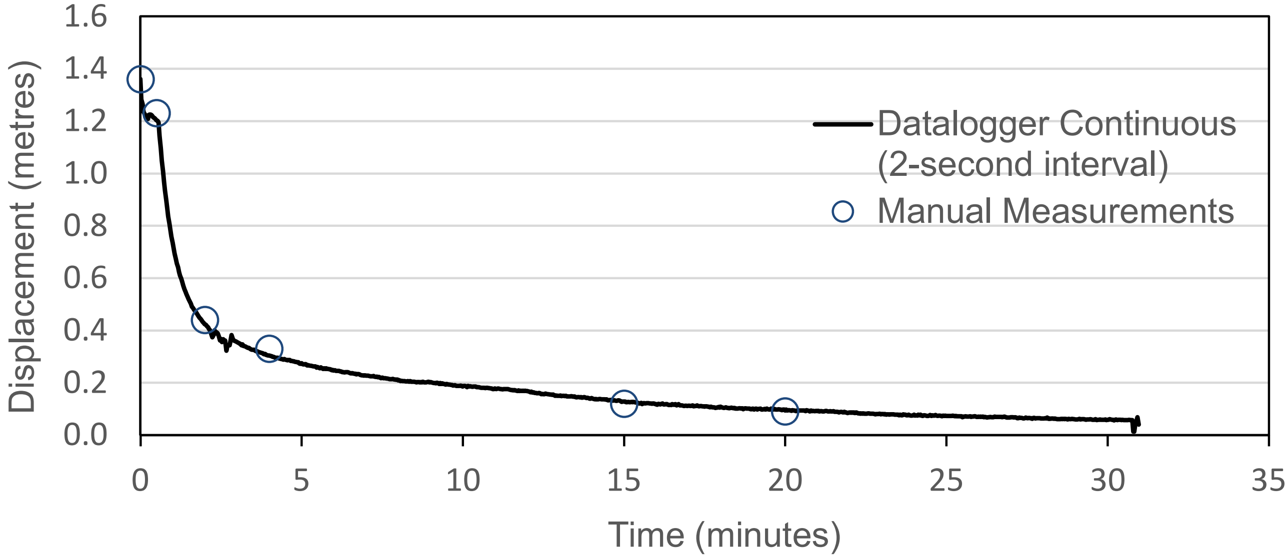
APPENDIX F

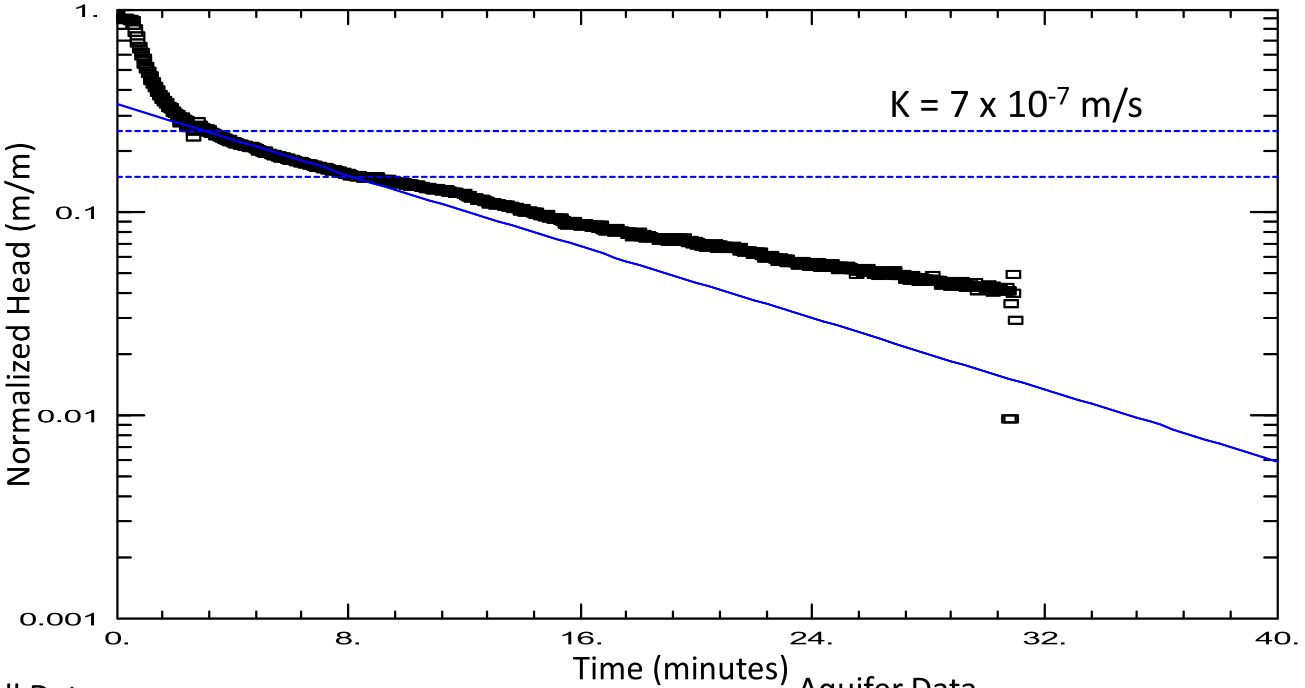
Hydraulic Conductivity Testing Results
Figure 1

Report to: Novatech
GEMTEC Project: 100011.049 (November 4, 2025)

Recovery Test Data

FIGURE 1


BH23-05 Recovery Test


BH23-05 Recovery Test: Hvorslev Analysis

Well Data:

Displacement observed: 1.36 metres

Well Depth: 4.6 metres

Screen Length: 3.05 metres

Well Radius: 0.025 metres

Aquifer Data

Saturated Thickness: 3.43 metres

Anisotropy Ratio (Kz/Kr): 1

Aquifer Model: Unconfined, Hvorslev

Static Water Level: 0.04 metres bgs

GEMTEC

CONSULTING ENGINEERS AND SCIENTISTS

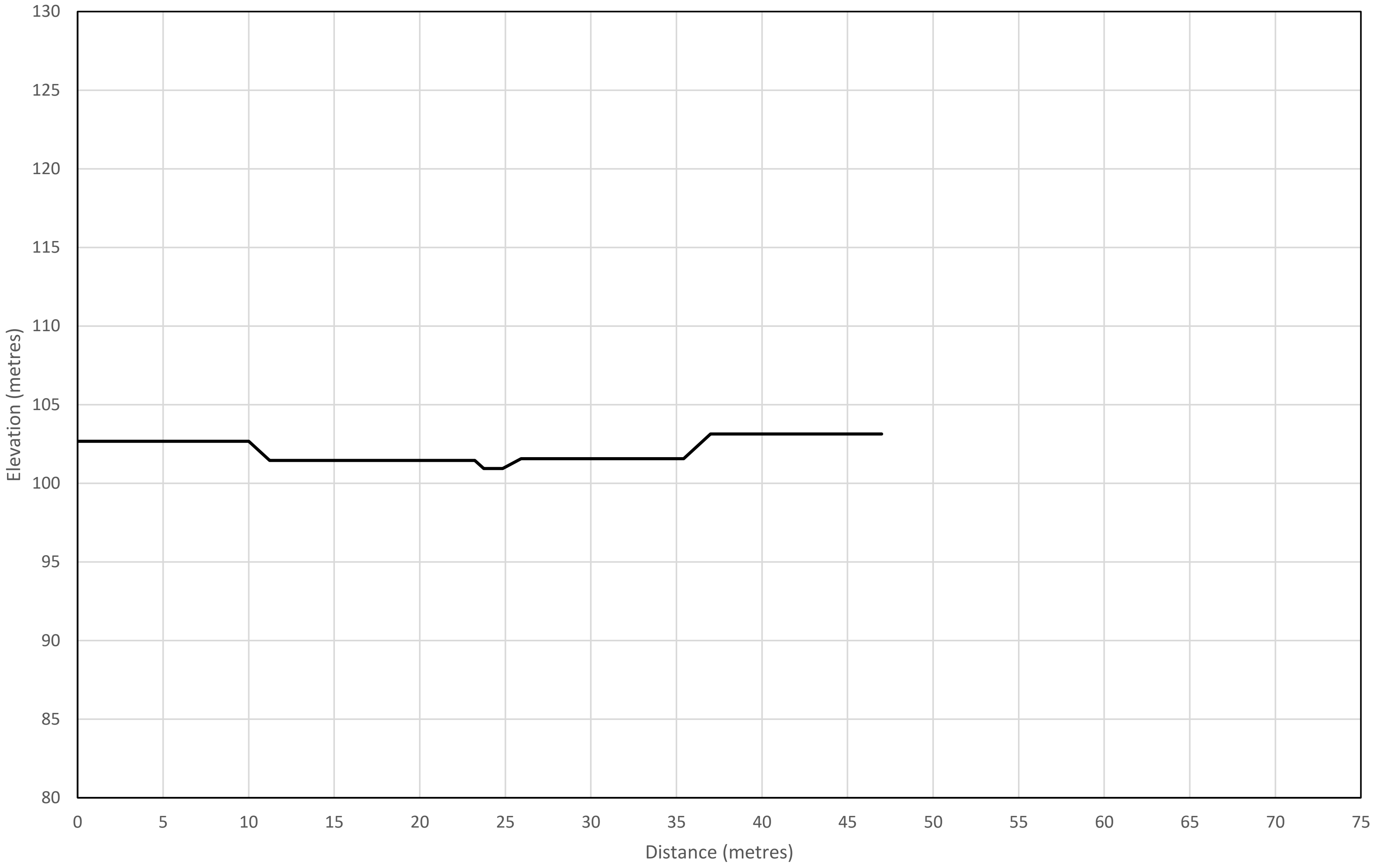
Date: May 2023

Project: 100011.049

APPENDIX G

Slope Stability Analysis
Figure G1 to G14

Report to: Novatech
GEMTEC Project: 100011.049 (November 4, 2025)



GEMTEC
Consulting Engineers and Scientists

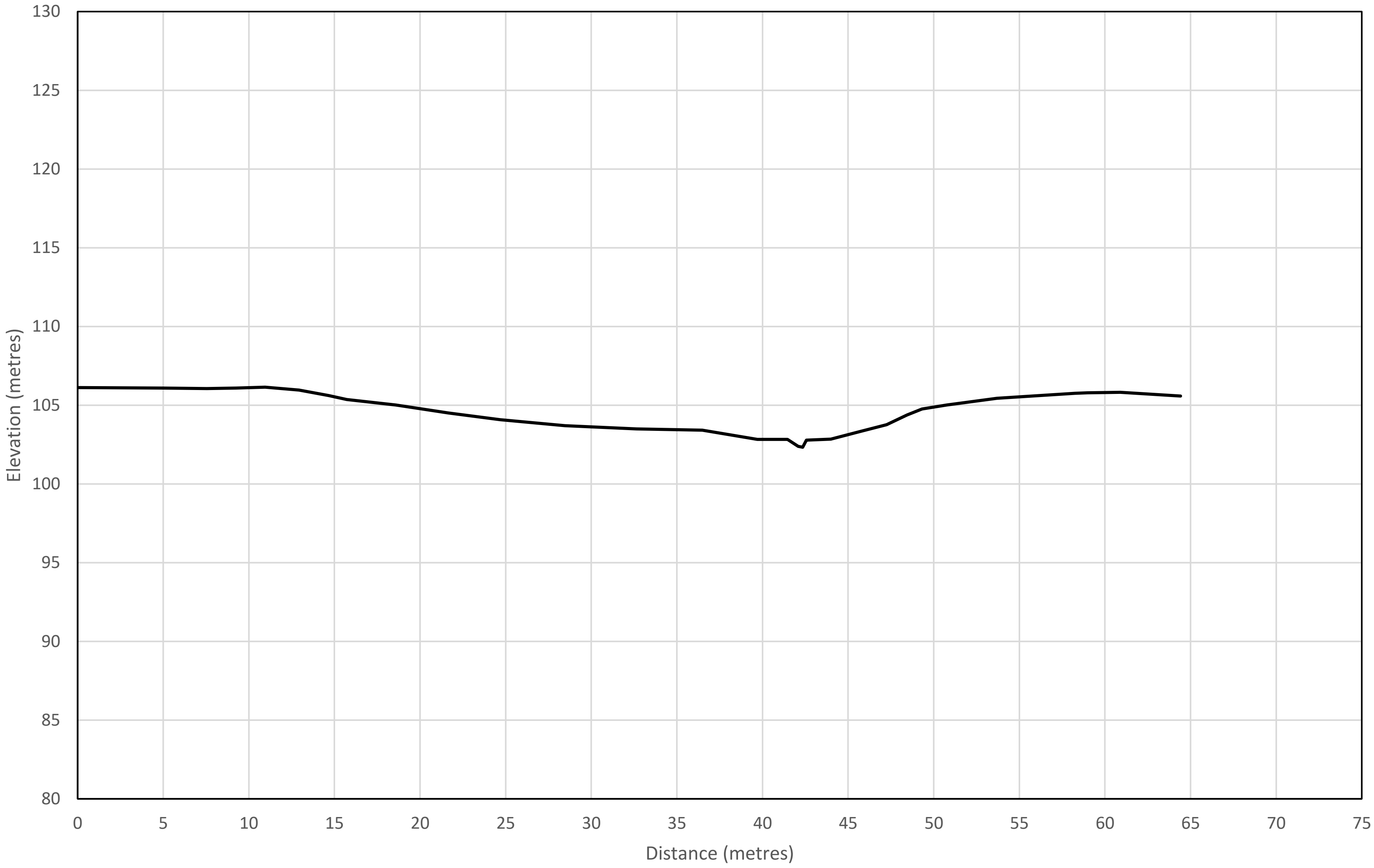
Slope Cross Section A-A

Carp Airport Commercial Development Phase 2

Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G1



GEMTEC
Consulting Engineers and Scientists

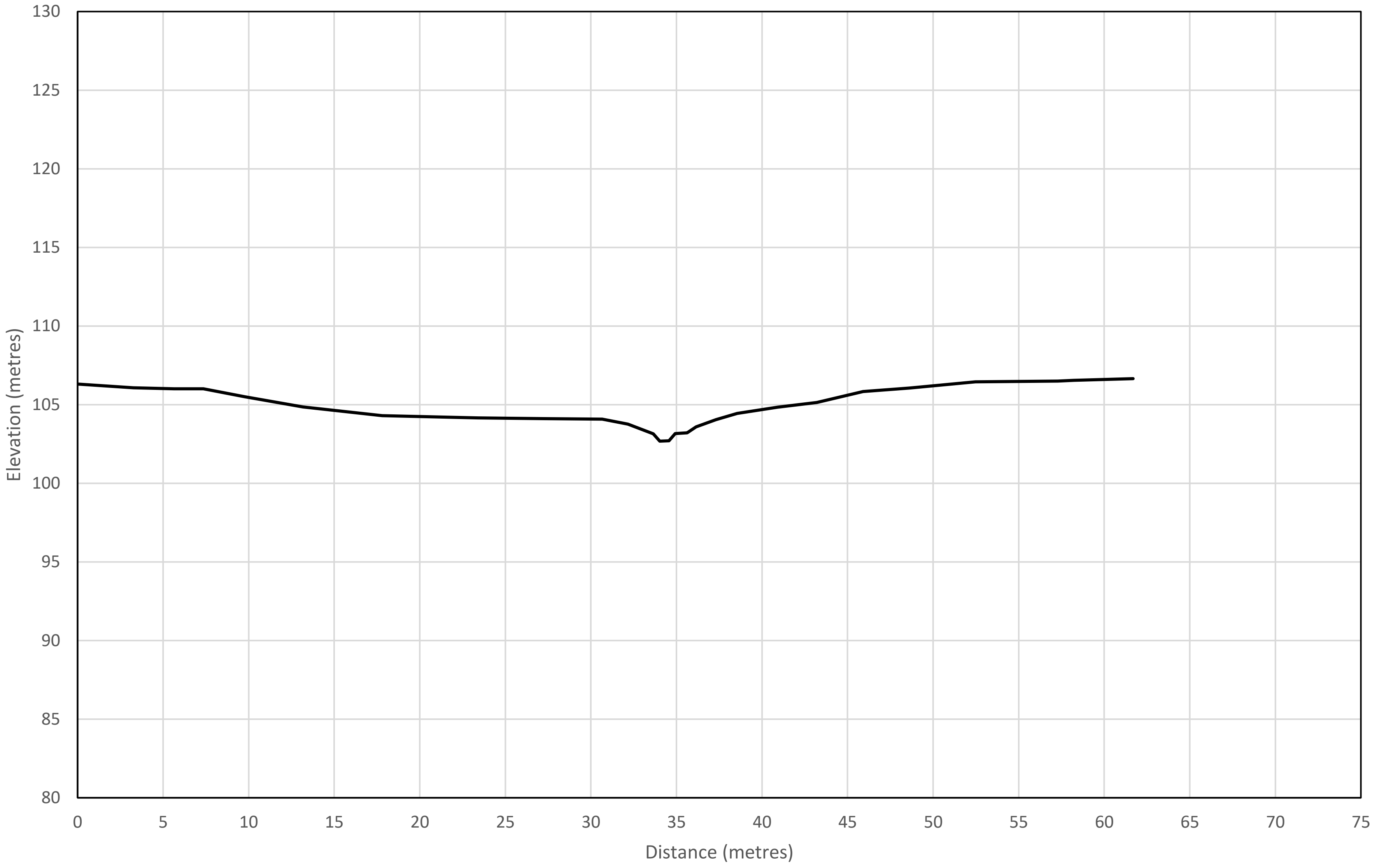
Slope Cross Section B-B

Carp Airport Commercial Development Phase 2

Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G2



GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

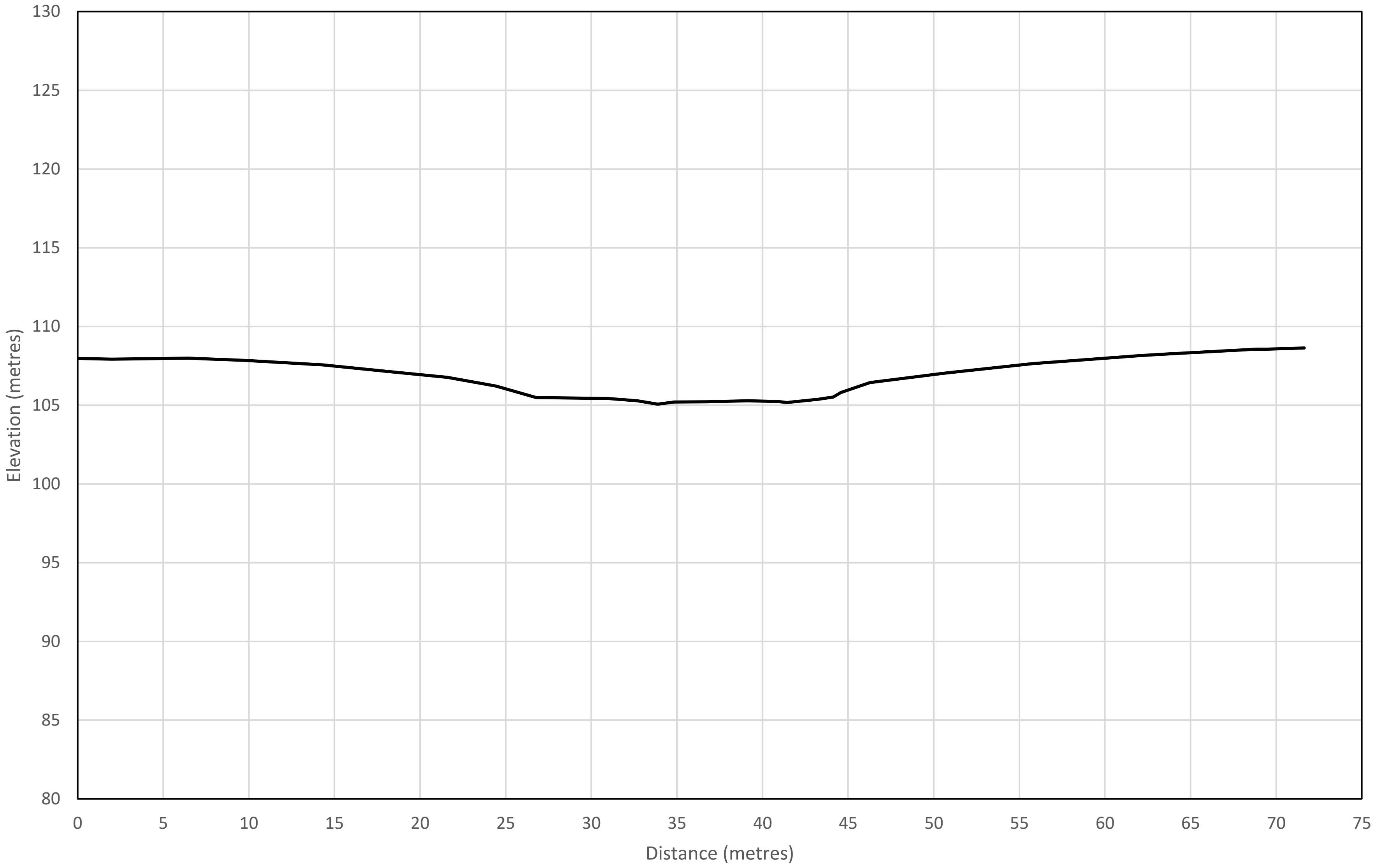
Slope Cross Section C-C

Carp Airport Commercial Development Phase 2

Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G3



GEMTEC
CONSULTING ENGINEERS
AND SCIENTISTS

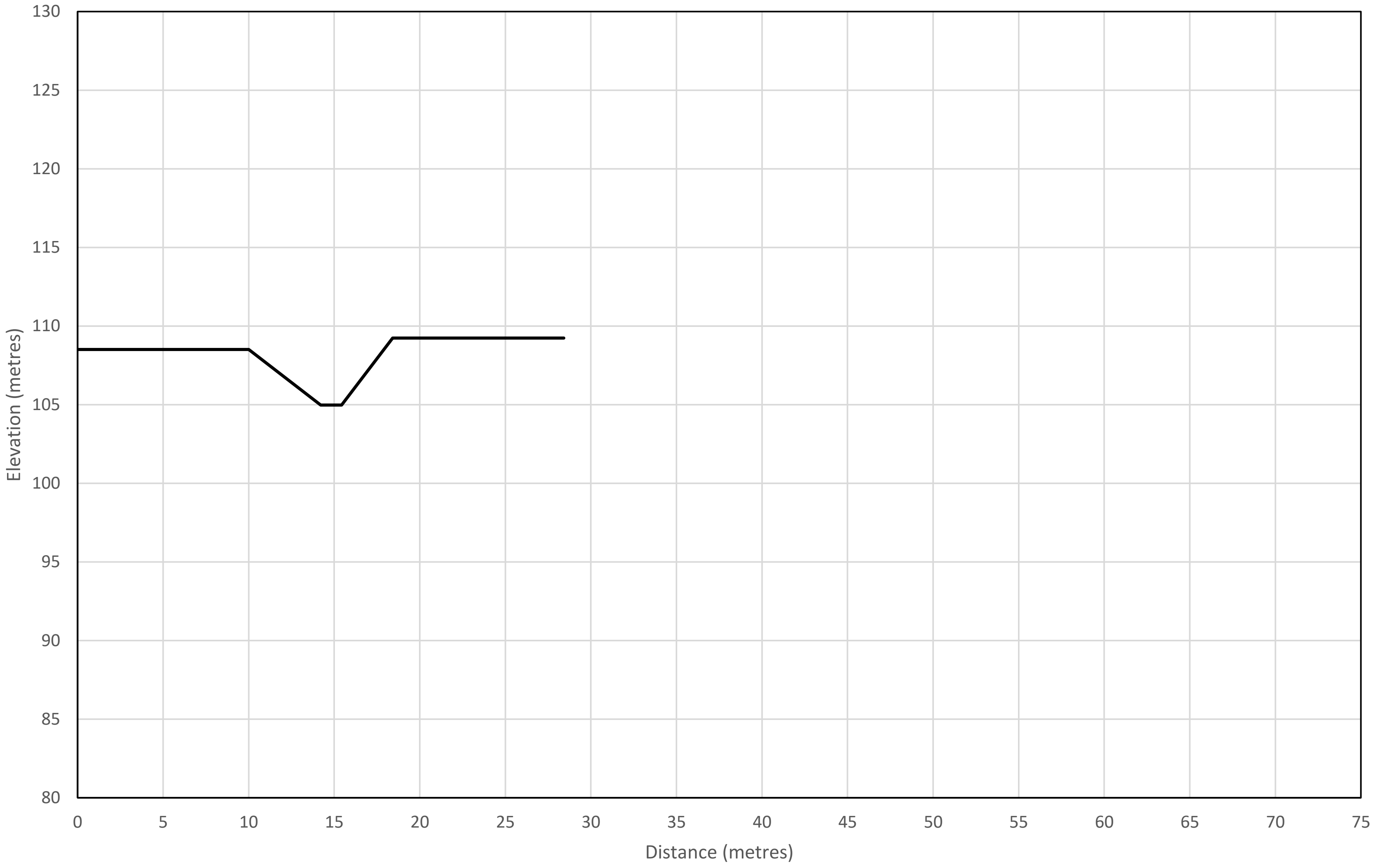
Slope Cross Section D-D

Carp Airport Commercial Development Phase 2

Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G4



GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

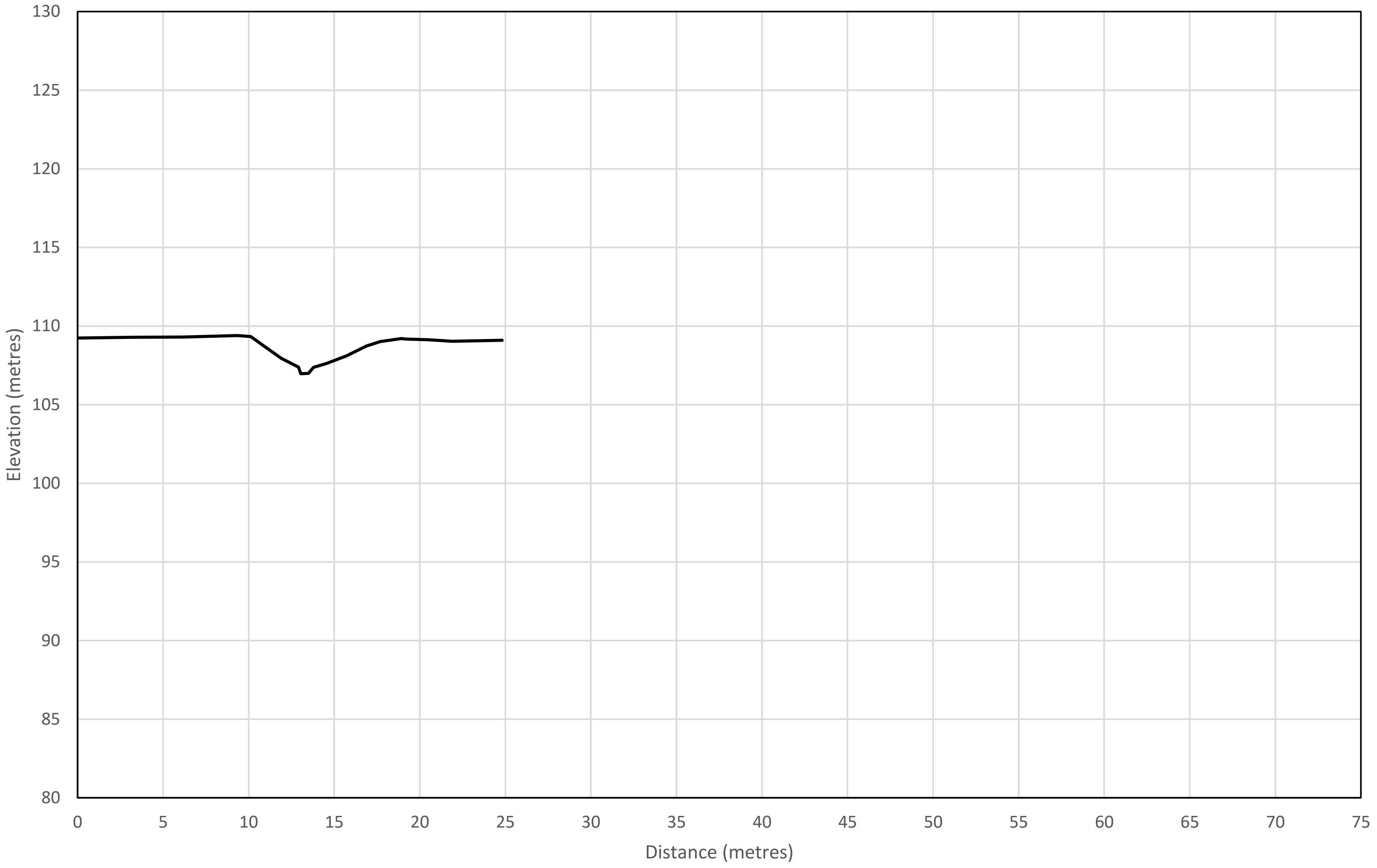
Slope Cross Section E-E

Carp Airport Commercial Development Phase 2

Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G5



GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

Slope Cross Section F-F

Carp Airport Commercial Development Phase 2

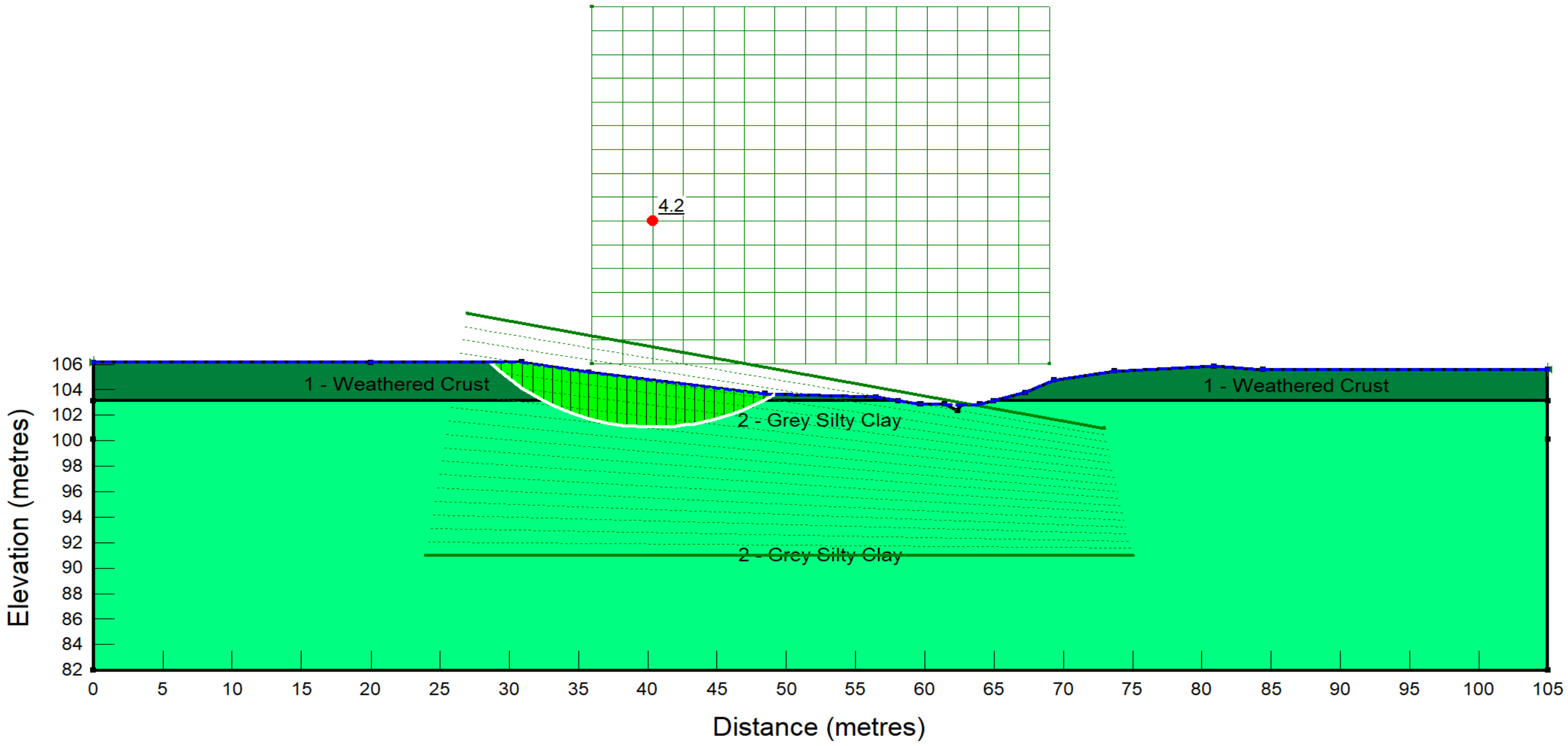
Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G6

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟩	1 - Weathered Crust	Mohr-Coulomb	18.5	5	35
🟩	2 - Grey Silty Clay	Mohr-Coulomb	17.9	7	35

Name: 100011.049_Section B-B Static (L-R)
Method: Morgenstern-Price
Direction of movement: Left to Right
Horz Seismic Coef.: 0



GEMTEC
Consulting Engineers
and Scientists

Static Slope Assessment - Cross Section B-B

Carp Airport Commercial Development Phase 2

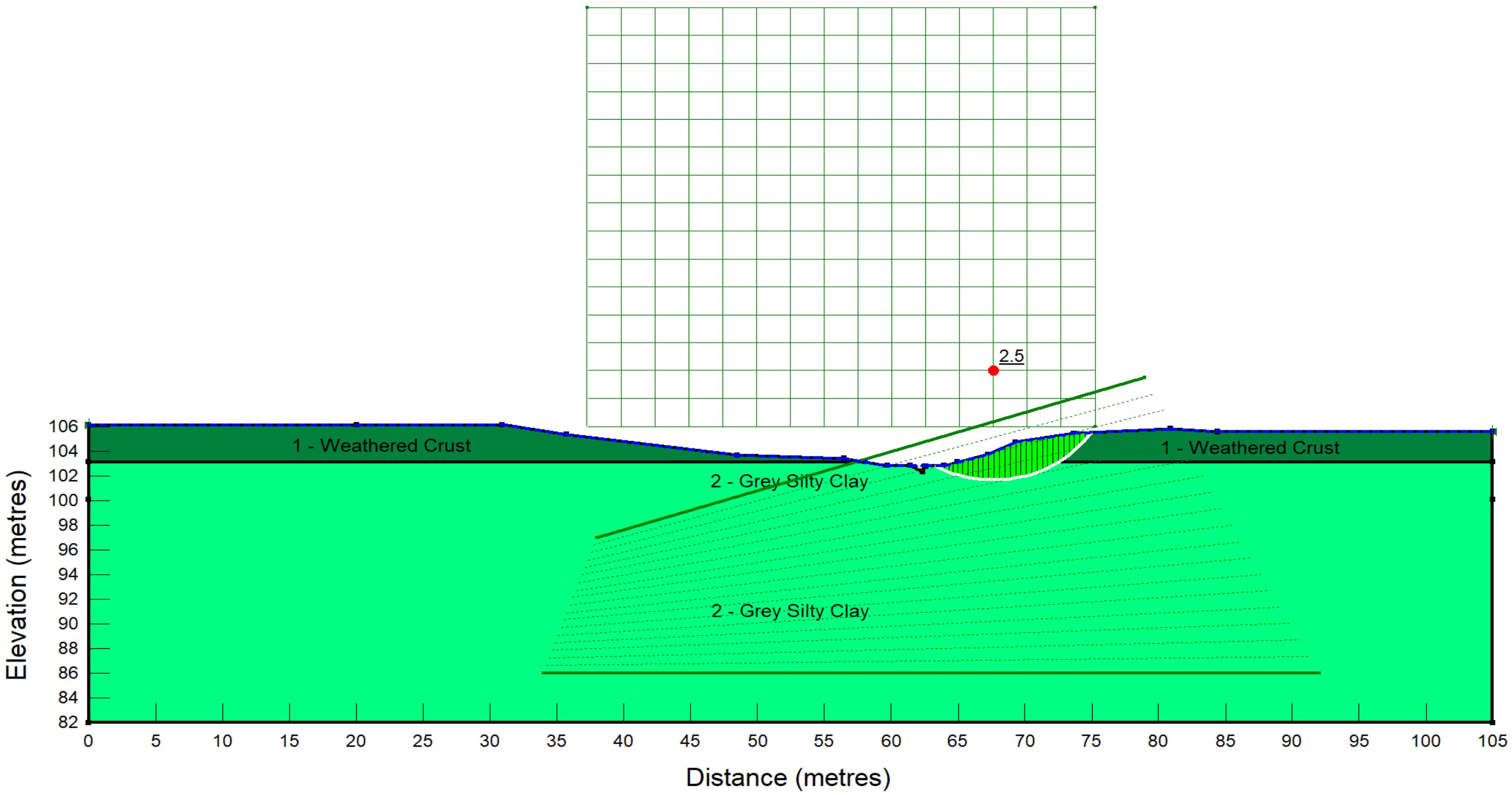
Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G7

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟩	1 - Weathered Crust	Mohr-Coulomb	18.5	5	35
🟩	2 - Grey Silty Clay	Mohr-Coulomb	17.9	7	35

Name: 100011.049_Section B-B Static (R-L)
Method: Morgenstern-Price
Direction of movement: Right to Left
Horz Seismic Coef.: 0



GEMTEC
Consulting Engineers and Scientists

Static Slope Assessment - Cross Section B-B

Carp Airport Commercial Development Phase 2

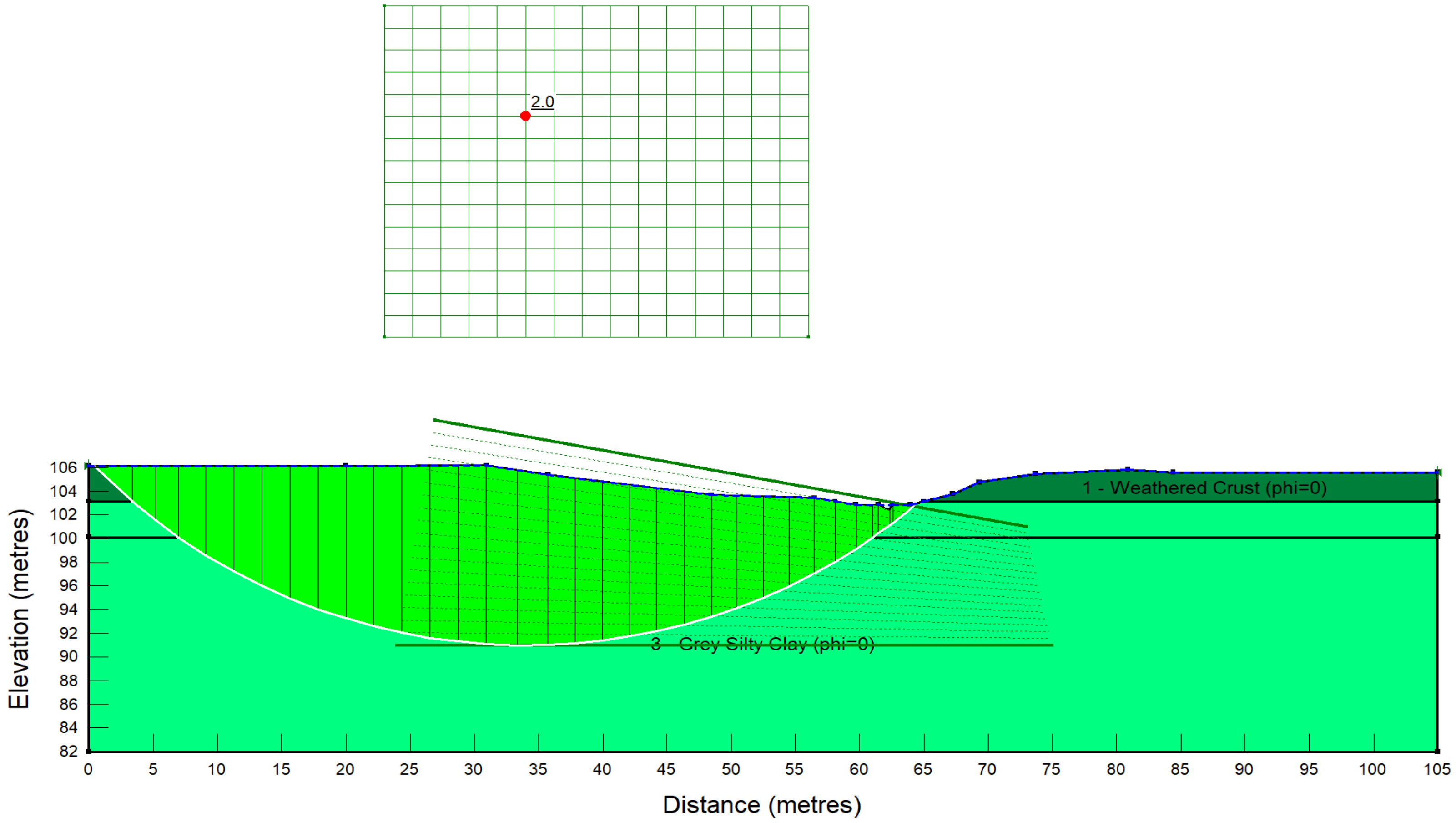
Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G8

Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	Cohesion (kPa)
🟩	1 - Weathered Crust (phi=0)	Undrained (Phi=0)	18.5			75
🟩	2 - Grey Silty Clay (phi=0)	Undrained (Phi=0)	17.9			35
🟩	3 - Grey Silty Clay (phi=0)	S=f(depth)	17.9	35	3	

Name: 100011.049_Section B-B Seismic (L-R)
Method: Morgenstern-Price
Direction of movement: Left to Right
Horz Seismic Coef.: 0.136



GEMTEC
Consulting Engineers and Scientists

Seismic Slope Assessment - Cross Section B-B

Carp Airport Commercial Development Phase 2

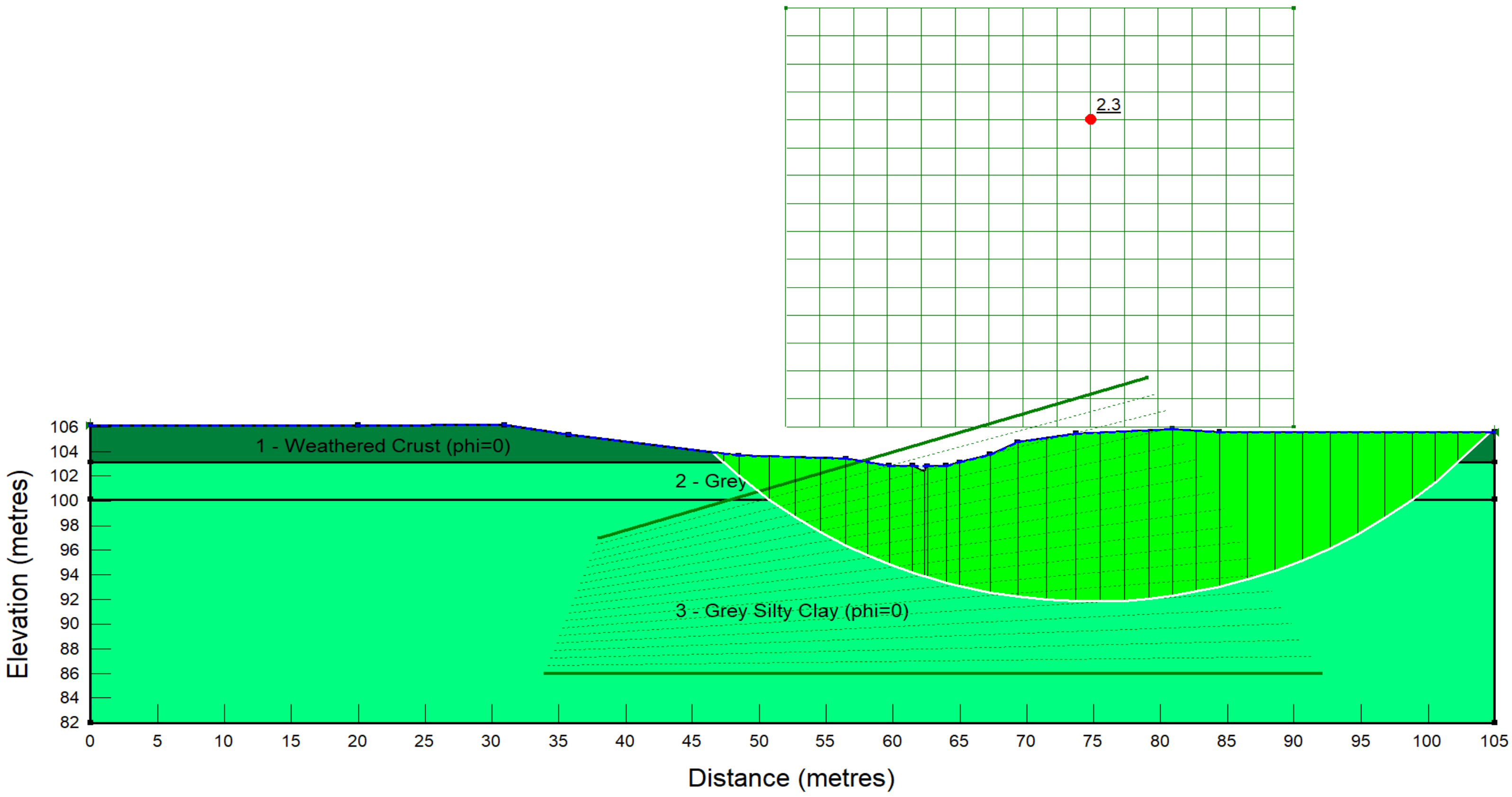
Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G9

Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	Cohesion (kPa)
🟩	1 - Weathered Crust (phi=0)	Undrained (Phi=0)	18.5			75
🟩	2 - Grey Silty Clay (phi=0)	Undrained (Phi=0)	17.9			35
🟩	3 - Grey Silty Clay (phi=0)	S=f(depth)	17.9	35	3	

Name: 100011.049_Section B-B Seismic (R-L)
Method: Morgenstern-Price
Direction of movement: Right to Left
Horz Seismic Coef.: 0.136



GEMTEC
Consulting Engineers
and Scientists

Seismic Slope Assessment - Cross Section B-B

Carp Airport Commercial Development Phase 2

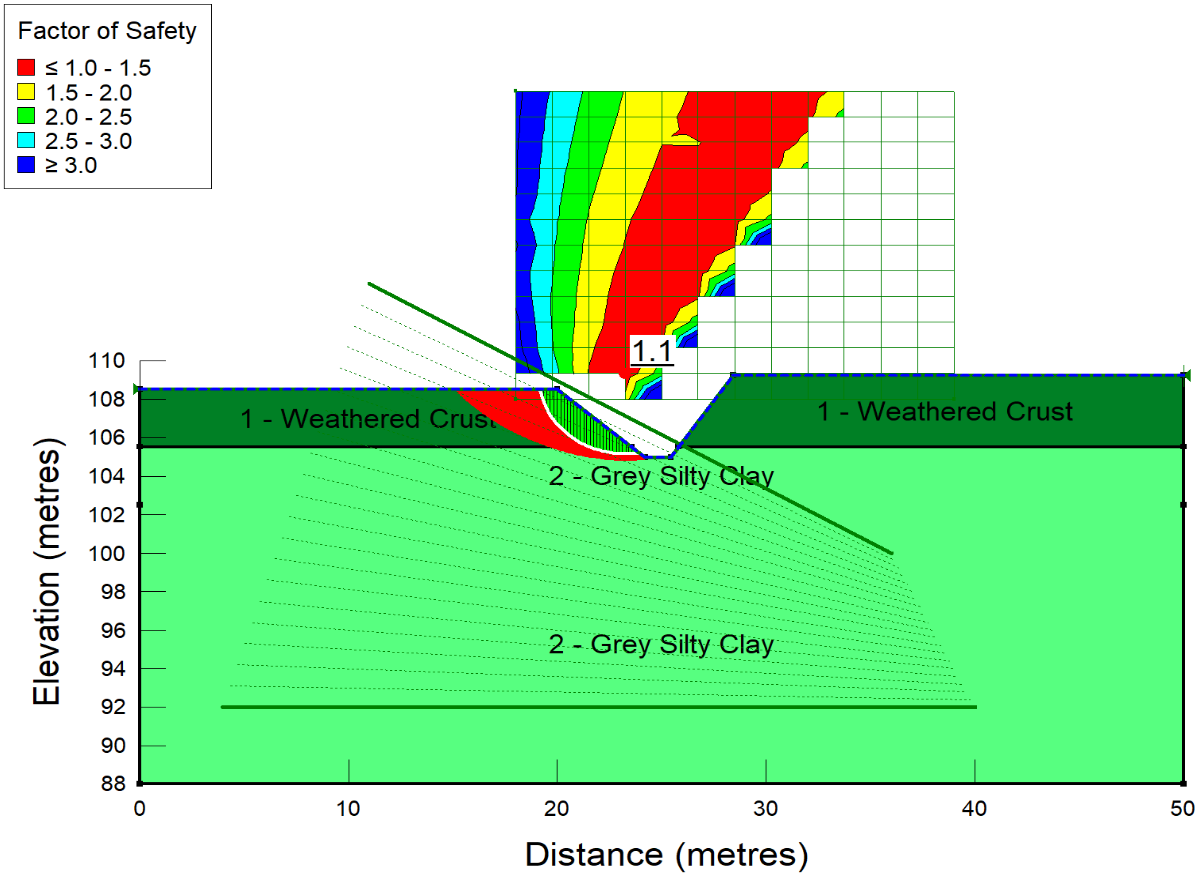
Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G10

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
🟩	1 - Weathered Crust	Mohr-Coulomb	18.5	5	35	1
🟩	2 - Grey Silty Clay	Mohr-Coulomb	17.9	7	35	1

Name: 100011.049_Section E-E Static (L-R)
Method: Morgenstern-Price
Direction of movement: Left to Right
Horz Seismic Coef.: 0



GEMTEC
Consulting Engineers and Scientists

Static Slope Assessment - Cross Section E-E

Carp Airport Commercial Development Phase 2

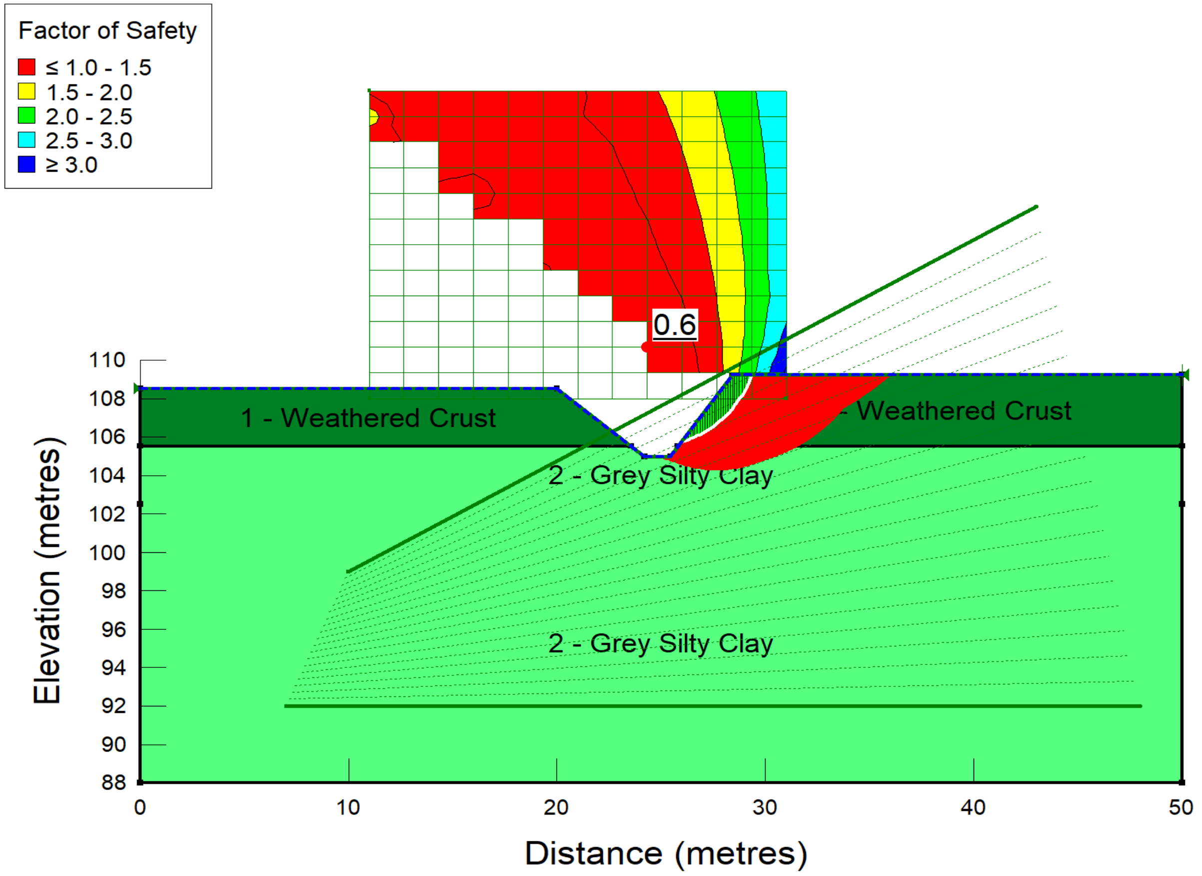
Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G11

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
🟩	1 - Weathered Crust	Mohr-Coulomb	18.5	5	35	1
🟩	2 - Grey Silty Clay	Mohr-Coulomb	17.9	7	35	1

Name: 100011.049_Section E-E Static (R-L)
Method: Morgenstern-Price
Direction of movement: Right to Left
Horz Seismic Coef.: 0



GEMTEC
Consulting Engineers and Scientists

Static Slope Assessment - Cross Section E-E

Carp Airport Commercial Development Phase 2

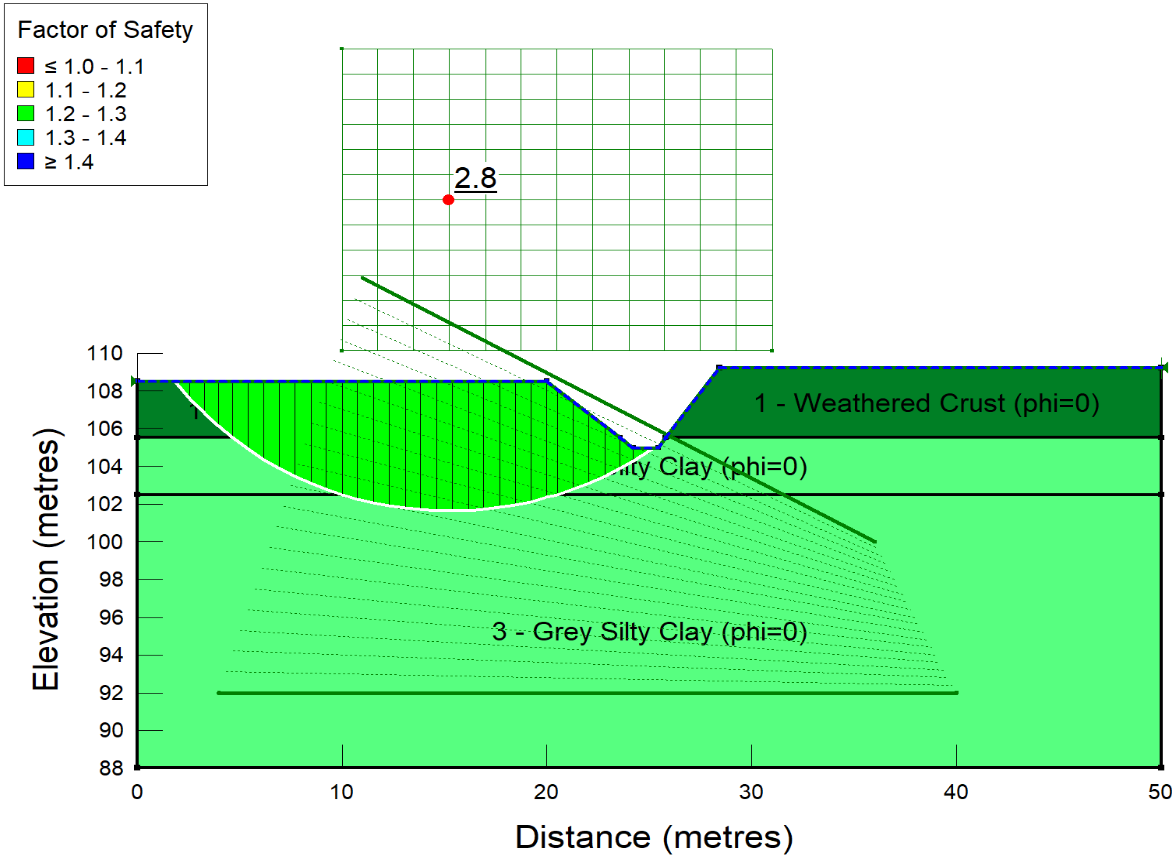
Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G12

Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m³)/m)	Cohesion (kPa)	Piezometric Line
🟩	1 - Weathered Crust (phi=0)	Undrained (Phi=0)	18.5			75	1
🟩	2 - Grey Silty Clay (phi=0)	Undrained (Phi=0)	17.9			35	1
🟩	3 - Grey Silty Clay (phi=0)	S=f(depth)	17.9	35	3		1

Name: 100011.049_Section E-E Seismic (L-R)
Method: Morgenstern-Price
Direction of movement: Left to Right
Horz Seismic Coef.: 0.136



GEMTEC
Consulting Engineers and Scientists

Seismic Slope Assessment - Cross Section E-E

Carp Airport Commercial Development Phase 2

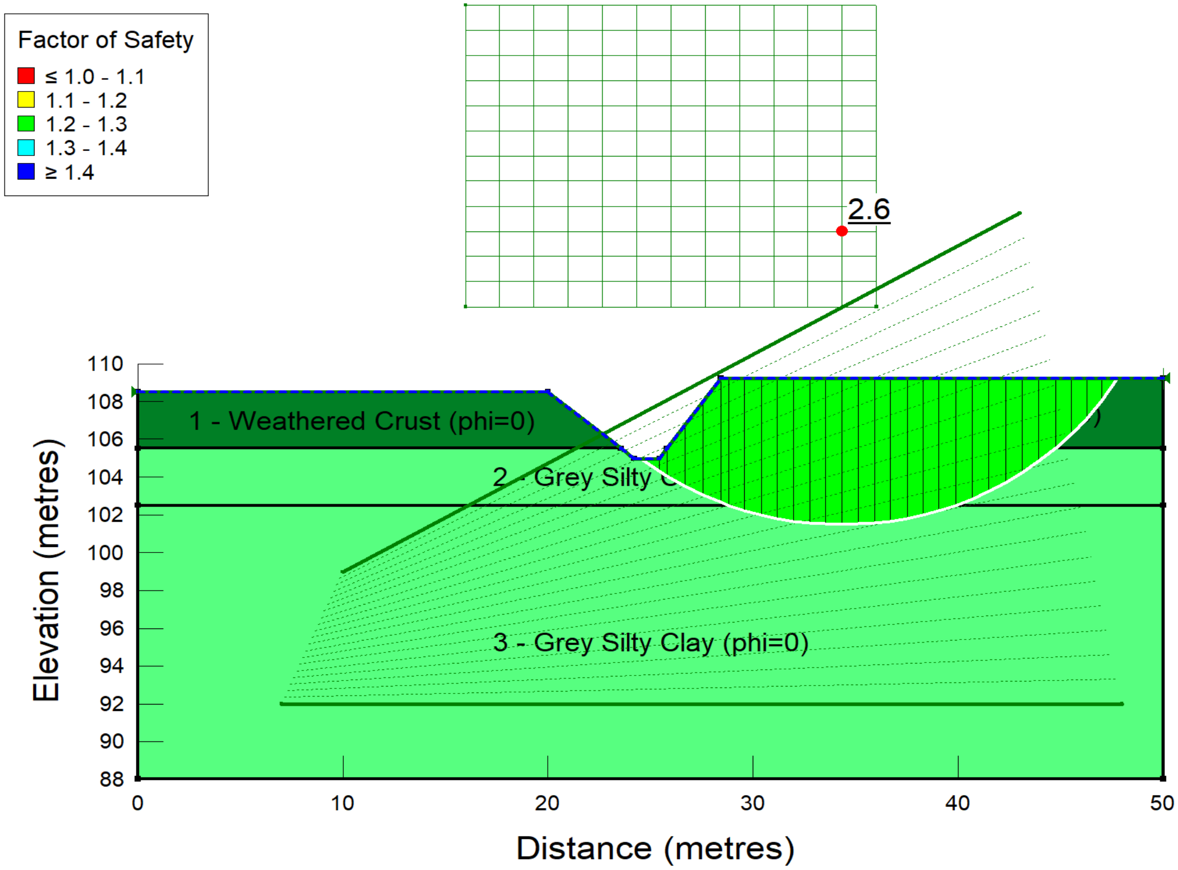
Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G13

Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	Cohesion (kPa)	Piezometric Line
🟩	1 - Weathered Crust (phi=0)	Undrained (Phi=0)	18.5			75	1
🟩	2 - Grey Silty Clay (phi=0)	Undrained (Phi=0)	17.9			35	1
🟩	3 - Grey Silty Clay (phi=0)	S=f(depth)	17.9	35	3		1

Name: 100011.049_Section E-E Seismic (R-L)
Method: Morgenstern-Price
Direction of movement: Right to Left
Horz Seismic Coef.: 0.136



GEMTEC
Consulting Engineers and Scientists

Seismic Slope Assessment - Cross Section E-E

Carp Airport Commercial Development Phase 2

Ottawa, Ontario

Project No.	100011.049
Drawn:	WAM
Date:	13/09/2024

Figure G14

experience • knowledge • integrity

civil	civil
geotechnical	géotechnique
environmental	environnement
structural	structures
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

expérience • connaissance • intégrité