REPORT

FOUNDATION INVESTIGATION AND DESIGN REPORT

Gananoque South Commercial Vehicle Inspection Facility
Town of Gananoque, Leeds and Grenville County, Ontario
MTO Assignment No. 4017-E-0003, G.W.P. 4009-14-00

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MTO ASSIGNMENT NO. 4017-E-0003, G.W.P. 4009-14-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Gananoque South Commercial Vehicle Inspection Facility (CVIF), in the Town of Gananoque, Leeds and Grenville County, Ontario. The proposed CVIF is to replace the existing Gananoque South Truck Inspection Station (TIS) and includes a new facility building, triage canopy, inspection canopy and bays, garage building, static scale, and tri-chord overhead sign (OHS) and breakaway sign supports, including a breakaway sign about 1 km to the west of the existing TIS.

This report addresses the results of the foundation investigation carried out for the proposed CVIF, as shown on the key plan on Drawing 1. The Terms of Reference and Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal, dated April 2017, which forms part of the Consultant Agreement (Assignment No. 4017-E-0003) for this project, and are summarized in Golder's Proposal document (Project No. 1780055 Assignment #4), dated April 20, 2018.

2.0 SITE DESCRIPTION

The existing MTO Gananoque South TIS is located along the eastbound Highway 401, approximately 7 km east of the Town of Gananoque. Overall the site consists of a flat asphalt surface with several structures present at the western portion of the existing station. The ground surface elevation across the existing TIS generally varies between Elev. 94.5 m and Elev. 95.5 m, sloping gently to the south, and is at Elev. 85.9 m at the proposed breakaway sign about 1 km to the west of the existing TIS. Immediately south of the asphalt along the entrance ramp to the TIS, the ground surface at the toe of the embankment is approximately 1 m lower than the top of asphalt. The Highway 401 alignment in the project area is oriented generally in a southwest-northeast orientation; however, for the purposes of this report, the Highway 401 alignment is described as being in a west – east orientation.

The adjacent land to the south of the MTO Gananoque South TIS is actively used for agricultural purposes. East, north and west of the station, the land is occupied by Highway 401.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigations

A foundation investigation was carried out at the existing TIS location in 1991 by the Department of Highways Ontario to assess the subsurface conditions at the site for the replacement of weigh scales. The results of the 1991 investigation are contained in the report titled:

- "Foundation Investigation Report, Hwy 401 South and North Sides, Weigh Scale at the Gananoque Truck Inspection Stations, W.P. 2501-91-01/02", dated July 24, 1991, GEOCRES No. 31C-150.

In 2016, Golder completed a Preliminary Foundation Investigation and Design Report for the site and two boreholes, designated as Boreholes 16-1 and 16-2, were advanced as part of the investigation. The results of the previous Golder investigation are contained in the report titled:

- "Preliminary Foundation Investigation and Design Report, Commercial Vehicle Inspection Facility 1.3 km East of Cliffe Road on Highway 401 Gananoque, W.P. 4046-10-01, Agreement No. 4010-E-0034", dated November 2016, GEOCRES No. 31C-254.

The current investigation is supplemented by relevant information contained within the above-mentioned reports. The locations of the previous boreholes relevant to the current investigation are shown on Drawing 1 and the previous borehole records are presented in Appendix A.

3.2 Current Investigation

Field work for the current investigation was carried out between April 22 and April 25, 2019. During this time a total of seven Standard Penetration Test (SPT) boreholes, designated as Boreholes 19-1 to 19-7, four Dynamic Cone Penetration Tests (DCPTs), designated as DCPTs 19-8, 19-9A and 19-9B plus one DCPT adjacent to Borehole 19-7, and five Cone Penetration Tests (CPTs), designated as CPT 19-10, 19-10B, 19-10C, 19-11 and 19-11B were advanced at the approximate locations shown on Drawing 1. An SPT borehole, originally designated as Borehole 19-9C, was attempted approximately 1 m south of DCPT 19-9B and during augering, the drill string was damaged; as a result, 5.2 m of augers were abandoned in the ground at this location.

The subsurface investigation was carried out using a track-mounted CME 75 drill rig supplied and operated by Pontil Drilling of Mount Albert, Ontario. The boreholes were advanced using 216 mm outside diameter continuous flight hollow-stem augers through the overburden, and HW-size casing and an HQ core barrel (64 mm inside diameter and 96 mm outside diameter) through the bedrock in Boreholes 19-4 and 19-6 using coring techniques.

Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter and 35 mm inside diameter split-spoon sampler driven by an automatic hammer in accordance with SPT procedures (ASTM D1586). The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions.

The SPT and DCPT boreholes were advanced to depths ranging from 5.5 m to 15.2 m below existing ground surface, including coring of bedrock for core lengths of 3.1 m in Boreholes 19-4 and 19-6. Boreholes 19-1, 19-5, 19-7, 19-8, 19-9A and 19-9B were terminated on refusal to further auger, sampler advancement and/or resistance to dynamic cone penetration. These depths to refusal do not confirm bedrock surface elevations but may be inferred to indicate proximity to bedrock surface.

The groundwater conditions were noted during drilling and immediately following drilling operations. A standpipe piezometer was installed in Borehole 19-4 to permit the monitoring of groundwater level at the borehole location. The piezometer consists of a 50 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. Above and below the well screen, the annulus surrounding the pipe was grouted to the surface with bentonite grout. The standpipe piezometer installation details are shown on the borehole record in Appendix B. The remaining boreholes were backfilled upon completion of drilling in accordance with Ontario Regulation 903 (as amended).

The CPTs were carried out using portable equipment supplied and operated by ConeTec Investigations Ltd. (ConeTec) of Richmond Hill, Ontario. The penetration tests used a 15 cm^2 tip base area probe, with an equal end area friction sleeve, and tip and sleeve capacities of 1,500 bar and 15 bar, respectively. The CPT holes were advanced to depths ranging from 2.3 m to 9.6 m below ground surface to refusal.

A total of nine dissipation tests were completed in the CPTs. The groundwater levels at the CPT locations were inferred based on the pore water pressure measurements and dissipation tests taken during advancement. The summary and plots of the CPTs and the pore pressure dissipation tests are included in the ConeTec report provided in Appendix D.

The field work was observed on a full-time basis by members of Golder's engineering staff, who located the boreholes and CPT holes in the field, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes and examined the soil and rock samples. The soil samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Whitby laboratory where the samples underwent further visual examination and geotechnical laboratory testing. Classification testing (water content, grain size distribution and Atterberg limits) was carried out on selected soil samples, to MTO LS and/or ASTM Standards, as appropriate.

The Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD), weathering and strength indices, discontinuity characteristics such as type, shape and surface roughness and classification data of the retrieved core samples were recorded in the field based on visual observation. The bedrock was sequentially photographed, packed and transported to Golder's Mississauga laboratory for further visual examination. Laboratory testing consisting of Unconfined Compression (UC) testing (including assessment of core density), was carried out on selected specimens of the bedrock core samples.

The as-drilled borehole, DCPT and CPT locations (in plan) were established by Tulloch Engineering in MTM NAD 83 Zone 9 northing and easting coordinates. The ground surface elevations are referenced to Geodetic datum. The as-drilled borehole coordinates were converted from MTM NAD 83 Zone 9 coordinates to corresponding latitudes and longitudes. The borehole coordinates together with latitudes and longitudes, as provided on the borehole and drillhole records and on Drawing 1, ground surface elevations and drilled depths are summarized below.

Test Hole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
Borehole 19-1	4,912,966.4 (44.355985)	337,304.0 (-79.092262)	85.9	7.7
Borehole 19-2	4,913,402.8 (44.359878)	338,065.9 (-79.082677)	95.2	8.2
Borehole 19-3	4,913,378.0 (44.359654)	338,080.0 (-79.082501)	94.3	8.2
Borehole 19-4	4,913,440.9 (44.360214)	338,218.3 (-79.080763)	94.8	15.2[1]
Borehole 19-5	4,913,392.8 (44.359779)	338,262.7 (-79.080209)	94.7	7.9
Borehole 19-6	4,913,469.8 (44.360470)	338,298.4 (-79.079755)	94.4	10.2[1]
Borehole/DCPT 19-7	4,913,461.3 (44.360398)	338,200.0 (-79.080990)	95.3	12.2 (SPT) 14.1 (DCPT)

Test Hole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
DCPT 19-8	4,913,454.6 (44.360333)	338,304.6 (-79.079679)	94.5	7.1
DCPT 19-9A	4,913,468.9 (44.360461)	338,325.2 (-79.079420)	94.6	5.5
DCPT 19-9B	4,913,467.9 (44.360452)	338,325.2 (-79.079420)	94.6	5.6
CPT 19-10C[2]	4,913,448.3 (44.360281)	338,205.7 (-76.080919)	94.9	9.6
CPT 19-11B[3]	4,913,462.7 (44.360408)	338,268.5 (-76.080130)	94.9	4.2

Notes:

1. Includes 3.1 m of bedrock coring.
2. CPT 19-10 and 19-10B were attempted near the location of CPT 19-10C; however, CPT 19-10 and 19-10B were terminated due to refusal at 2.3 m and 5.0 m, respectively.
3. CPT 19-11 was attempted near the location of CPT19-11B; however, it was terminated due to refusal at 4.1 m.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The project area is located within the Leeds Knobs and Flats physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putman, 1984).

The Leeds Knobs and Flats is in an area consisting of Precambrian rock knobs and channels which were filled with clay flats by the waters of Lake Iroquois during the Pleistocene Age. Surficial deposits of clay or sand and gravel and/or glacial till generally overlie the bedrock. The bedrock generally consists of strong to very strong granitic gneiss as part of the Central Metasedimentary Belt of the Grenville Province.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes and CPTs advanced during the previous and current investigations, together with the results of the laboratory tests and in situ testing carried out, are presented on the borehole and drillhole records, CPT report and laboratory test sheets in Appendices A to D.

The stratigraphic boundaries shown on the borehole records and on the cross-sections on Drawing 2 are inferred from non-continuous sampling, observations of drilling progress and the results of SPT, DCPT and CPT results. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change and moreover, the interpreted stratigraphy shown on Drawing 2 represent a simplification of the subsurface conditions. Furthermore, subsurface conditions will vary between and beyond the borehole and CPT locations.

A detailed description of the subsurface conditions encountered at the site is provided in the following sub-sections.

4.2.1 Asphalt

Asphalt was encountered at ground surface in Boreholes 19-2, 19-4, and 19-7 and measured approximately 125 mm to 150 mm thick.

In Boreholes 1 and 2 from the 1991 investigation, approximately 0.6 m of asphalt was encountered immediately below ground surface.

4.2.2 Topsoil

An approximately 100 mm thick layer of topsoil was encountered at ground surface in Borehole 19-5 which was drilled within the agricultural field, south of the existing TIS.

In Boreholes 16-1 and 16-2 from the previous 2016 investigation, approximately 100 mm to 200 mm of topsoil was encountered at ground surface.

4.2.3 Gravel (GW) (FILL)

An approximately 25 mm thick layer of surficial gravel fill was encountered at ground surface in Borehole 19-6 which was drilled on the gravel shoulder of the existing TIS.

4.2.4 Gravelly Sand (SW), Sand (SW) and Gravel, and Sand (SP) (FILL)

A layer of non-cohesive gravelly sand, sand and gravel and/or sand fill was encountered at ground surface in Borehole 19-1 and beneath the asphalt in Boreholes 19-2, 19-4, and 19-7. The thickness of the non-cohesive fill layer ranges from 0.4 m to 2.1 m, extending to depths ranging from 0.5 m to 2.2 m below ground surface (Elevation 85.2 m in Borehole 19-1 and Elevations 94.3 m to 93.0 m in Boreholes 19-2, 19-4 and 19-7).

In Boreholes 1 and 2 from the 1991 investigation, approximately 1.3 m to 1.5 m of sand fill containing some silt was encountered below the asphalt.

The SPT "N"-values measured within the non-cohesive fill layer range from 8 to 58 blows per 0.3 m of penetration, indicating a loose to very dense state of compactness.

In Borehole 1 from the 1991 investigation, a natural water content measured on a sample of the sand fill was 12 per cent and a grain size distribution was also completed on this sample, as presented on the borehole record.

4.2.5 Silty Clay (CI) (FILL)

A layer of cohesive silty clay fill was encountered beneath the non-cohesive fill in Boreholes 19-1, 19-4, 19-6, and 19-7. The thickness of the cohesive fill layer ranges from 0.7 m to 1.7 m, extending to depths of 0.7 m and 2.2 m below ground surface (Elevation 83.7 m in Borehole 19-1 and Elevations 93.7 m to 92.6 m in Boreholes 19-4, 19-6 and 19-7).

The SPT "N"-values measured within the cohesive fill layer range from 6 to 17 blows per 0.3 m of penetration, indicating a firm to very stiff consistency.

4.2.6 Clayey Silt (CL) to Clay (CH)

A 5.5 m and 0.8 m thick cohesive clayey silt to clay deposit, containing trace to some sand, was encountered beneath the cohesive fill in Borehole 19-1 and beneath the silt deposit (discussed below) in Borehole 19-4. The surface of the clayey silt to clay was encountered at depths of 2.2 m and 3.7 m below ground surface in Boreholes 19-1 and 19-4, respectively (Elevations 83.7 m and 91.1 m, respectively).

In Boreholes 1 and 2 from the previous 1991 investigation, 1.5 m to 1.9 m of silty clay, containing some sand was encountered below the sand fill. In Boreholes 16-1 and 16-2 from the previous 2016 investigation, a 0.6 m and 1.3 m thick deposit of silty clay was encountered, respectively.

The SPT "N"-values measured within the clayey silt to clay deposit range from 4 to 17 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency.

The result of four grain size distribution tests carried out on samples from the clayey silt to clay deposit from the current investigation are shown on Figure C1 in Appendix C. The results of Atterberg limits testing on the deposit measured liquid limits ranging from about 23 to 52 per cent, plastic limits ranging from about 16 to 30 per cent, and plasticity indices ranging from about 11 to 29 per cent. These results, which are plotted on a plasticity chart on Figure C2 in Appendix C, indicate the deposit is a clayey silt of low plasticity to a clay of high plasticity.

In Boreholes 1 and 2 from the previous 1991 investigation, the results of Atterberg limits testing and grain size distribution testing on two samples of the deposit are shown on the borehole record; the test results are generally consistent with the results from the current investigation. The natural water contents measured on samples of the clayey silt to clay deposit range from 23 to 37 per cent.

4.2.7 Silt to Sandy Silt (ML)

A 1.5 m thick non-cohesive silt to sandy silt deposit was encountered underlying the cohesive fill in Boreholes 19-4, 19-6, and 19-7. The surface of the deposit was encountered at a depth of 0.7 m and 2.2 m below ground surface (Elevations 93.7 m to 92.6 m).

In Borehole 1 from the previous investigation, a 1.6 m thick deposit of sandy silt to silty sand was encountered below the silty clay. In Boreholes 16-1 and 16-2 from the previous investigation, a 1.9 m and 0.8 m thick deposit of silt to sandy silt was encountered below the silty clay.

The SPT "N"-values measured within the silt to sandy silt deposit range from 4 to 28 blows per 0.3 m of penetration, indicating a very loose to compact state of compactness.

The results of grain size distribution tests carried out on three samples from the silt to sandy silt deposit are shown on Figure C3 in Appendix C. The results of two grain size distribution tests on samples from the previous investigation are shown on the borehole records for Borehole 1 and Borehole 16-2; the test results are generally consistent with the results from the current investigation. The natural water content measured on samples of the silt to sandy silt deposit from the current investigation ranges from about 16 to 25 per cent.

4.2.8 Silty Sand (SM) to Sand (SP/SP-SM)

A non-cohesive deposit ranging in composition from silty sand to gravelly silty sand to sand to gravelly sand was encountered underlying the non-cohesive fill in Borehole 19-2, at ground surface in Boreholes 19-3, underlying the clayey silt in Borehole 19-4, underlying the topsoil in Borehole 19-5, and underlying the silt to sandy silt deposit in Boreholes 19-6 and 19-7. The surface of the deposit was encountered at ground surface to a depth of

4.5 m below ground surface (Elevations 94.6 m to 90.3 m). The thickness of the deposit ranges between 4.9 m and 7.0 m, where it was fully penetrated in Boreholes 19-4, 19-5 and 19-6. Boreholes 19-2, 19-3, and 19-7 were terminated within the deposit after exploring the deposit between 8.0 m and 8.5 m of depth (Borehole 19-7 encountered refusal potentially at bottom of deposit). In Borehole 1 from the previous investigation, a 4 m thick deposit of sand was encountered below the sandy silt to silty sand with the bottom of the deposit likely defined by probable bedrock. In Boreholes 16-1 and 16-2 from the previous investigation, the boreholes did not fully penetrate the sand deposit after exploring for 2.6 m and 2.9 m.

The SPT "N"-values measured within the silty sand to sand deposit range from 2 to 125 blows per 0.3 m of penetration, with three values of 100 blows for 0.1 m of penetration, indicating a very loose to very dense state of compactness.

The results of grain size distribution tests carried out on eleven samples from the silty sand to sand deposit are shown on Figures C4 and C5 in Appendix C. The results of two grain size distribution tests on samples from the previous investigation are shown on the borehole records for Borehole 1 and Borehole 16-2; the test results are generally consistent with the results from the current investigation. The natural water content measured on samples of the silty sand to sand deposit ranges from about 2 to 26 per cent.

4.2.9 Gravel (GP-GM) and Sand

A non-cohesive deposit of gravel and sand was encountered underlying the silty sand to sand deposit in Boreholes 19-4 and 19-5. The surface of the gravel and sand deposit was encountered at depths of 7.2 m and 10.1 m (Elevations 87.5 m and 84.7 m). The thickness of the deposit is 2.0 m at Borehole 19-4 and 0.7 m at Borehole 19-5, however Borehole 19-5 was terminated on auger and spoon refusal.

The SPT "N"-values measured within the gravel and sand deposit are 91 blows for 0.3 m of penetration and 100 blows for 0.1 m of penetration, indicating a very dense state of compactness.

The results of a grain size distribution test carried out on one sample from the gravel and sand deposit are shown on Figure C6 in Appendix C. The natural water content measured on a sample of the gravel and sand deposit is about 7 per cent.

4.2.10 Granitic Gneiss Bedrock

Bedrock coring was carried out in Boreholes 19-4 and 19-6 and the depths to the bedrock surface, corresponding elevations and the cored lengths are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Cored Length (m)	Bottom of Borehole Elevation (m)
19-4	94.8	12.1	82.7	3.1	79.6
19-6	94.4	7.1	87.3	3.1	84.2

Based on a review of the bedrock core samples, the bedrock generally consists of slightly weathered to fresh, slightly foliated, pink to red, coarse grained, faintly porous, very strong granitic gneiss. Details of the bedrock coring and core descriptions are presented on the Record of Drillhole sheets in Appendix B. Photographs of the recovered rock core samples are presented on Figures C7 and C8 in Appendix C.

The degree of weathering of the bedrock samples (i.e. fresh to slightly weathered – W1 to W2), and the strength classification of the intact rock mass based on field identification (i.e. strong – R4) are described in accordance with the International Society for Rock Mechanics (ISRM) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples obtained from the investigation ranges from about 45 per cent to 100 per cent, indicating a rock mass of poor to excellent quality, but mostly between 86 per cent and 100 per cent, indicating good to excellent rock mass quality, as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 97 per cent and 100 per cent and between 36 per cent and 97 per cent, respectively.

Unconfined Compression (UC) tests (ASTM D7012) were carried out on selected core samples of the granitic gneiss bedrock. The uniaxial compressive strength (UCS) of the intact samples is summarized below and the details are presented on the Rock Laboratory Test Results in Appendix C. Based on the UCS test results and in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as very strong (R5, 100 MPa < UCS < 250 MPa).

Borehole No.	Run No.	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)
19-6	1	7.6 - 7.9	86.8 – 86.5	179.1
19-6	1	8.3 - 8.6	86.1 – 85.8	141.3

4.2.11 Groundwater Conditions

The overburden samples obtained from the boreholes during the current investigation were generally moist to wet. Boreholes 19-3, 19-5 and 19-7 were dry upon completion of drilling. The unstabilized groundwater conditions were observed in the open Boreholes 19-1, 19-2, 19-4, and 19-6 immediately following the overburden drilling operations (augering) at depths ranging from 1.5 m to 2.7 m below ground surface (Elevation 84.4 m in Borehole 19-1 and Elevations 93.4 m to 91.7 m in Boreholes 19-2, 19-4 and 19-6). A standpipe piezometer was installed in Borehole 19-4 to permit the monitoring of groundwater level at this location and the depth to the groundwater level and elevation measured in the piezometer are summarized below.

Borehole No.	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Comment
19-4	94.8	1.8	93.0	April 25, 2019
		1.7	93.1	May 9, 2019

The water level measured in Borehole 1 during the 1991 investigation was 2.9 m (Elevation 92.4 m). The water level measured in Boreholes 16-1 and 16-2 during the 2016 investigation was 3.2 m and 3.4 m, respectively (Elevations 91.4 m and 91.3 m).

It should be noted that the groundwater levels in the area are subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

4.2.12 Analytical Testing Results

Analytical testing was carried out on three selected soil samples recovered from Boreholes 19-3, 19-4 and 19-6. The soil samples were submitted to AGAT Laboratories of Mississauga, Ontario for testing a suite of parameters associated with potential corrosion to steel and deterioration of concrete. The analytical laboratory test results are summarized below, and the detailed analytical laboratory test report is included in Appendix C.

Borehole No.	Sample No.	Depth (m) (Elev. m)	Parameters				
			Resistivity (ohm-cm)	Electrical Conductivity (mS/cm)	Soluble Sulphate (SO4) Content (µg/g)	Chloride (Cl) Content (µg/g)	pH
19-3	6	4.1 m (90.2 m)	5620	0.18	5	49	8.8
19-4	4	2.6 m (92.2 m)	833	1.20	28	575	8.2
19-6	4	2.6 m (91.9 m)	370	2.70	20	1430	8.0

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Yusuf Soliman, B.A.Sc., E.I.T., a geotechnical engineering intern with Golder and the technical aspects were reviewed by Ms. Sarah E. M. Poot, P.Eng., Associate of Golder and the Senior Foundation Engineer for this project. Ms. Lisa C. Coyne, P.Eng., a Principal of Golder and MTO Foundations Designated Contact, conducted an independent technical and quality control review of the report.

Signature Page

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PART B

FOUNDATION DESIGN REPORT
GANANOQUE SOUTH COMMERCIAL VEHICLE INSPECTION FACILITY
TOWN OF GANANOQUE, LEEDS AND GRENVILLE COUNTY, ONTARIO
MTO ASSIGNMENT NO. 4017-E-0003, G.W.P. 4009-14-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical recommendations for the design of foundations for a new Commercial Vehicle Inspection Facility (CVIF) which includes the following structures: a new facility building, triage canopy, inspection canopy and bays, garage building, static scale, tri-chord overhead sign (OHS) and breakaway sign supports. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation at the site along Highway 401, approximately 7 km west of Gananoque, Ontario.

The discussion and recommendations contained in this report are intended to provide the designers with sufficient information to complete the detail design of the CVIF foundations. The Foundation Investigation Report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor undertaking the work must make their own interpretation based on the factual data in Part A (Foundation Investigation) of this report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Frost Protection

All foundation elements should be provided with a minimum of 1.5 m of conventional soil cover for frost protection, in accordance with OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario),* or equivalent thickness of insulation below the foundation and extending beyond the edge of foundation, as applicable. As a guide, the MTO has adopted a 25 mm thickness of rigid polystyrene foam insulation as equivalent to a 0.3 m reduction in conventional soil cover.

6.3 Design of Commercial Vehicle Inspection Facility (CVIF) Foundations

Based on the 60% submission contract drawings, dated August 28, 2019 provided by Dillon, it is understood that the new structures for the CVIF will include a facility building, a triage canopy, a static scale, inspection canopy and bays, and a garage building. Several foundation options including spread and/or strip footings founded on native soils or on engineered fill, and steel H-piles or pipe piles driven into the very dense “100-blow” cohesionless deposits or onto the granitic gneiss bedrock surface, have been considered and evaluated for support of the new CVIF structures.

Shallow foundations are suitable for supporting the new CVIF structures as the native shallow deposits (i.e., compact sandy silt/silt, compact to very dense sand, firm to very stiff silty clay) underlying engineered compacted granular fill will provide sufficient geotechnical resistance to support the structure loads. Deep foundations driven into the very dense “100-blow” cohesionless deposits or onto the granitic gneiss bedrock surface are also technically feasible from a foundations perspective, but they are not considered necessary or economical for the relatively low design loads that will be imposed by the new building structures.

A grade raise is proposed in the vicinity of some of the facilities, and it is recommended that consideration be given to a brief preloading period within these areas, and other areas of grade raises at the site, if applicable, to minimize any potential differential settlement that may arise due to varying thicknesses of loose/firm soils across the site, as discussed further in Section 6.3.4.

6.3.1 Founding Elevations

Based on the results of the subsurface investigations, the proposed structures can be founded on conventional spread and/or strip foundations bearing on the native soils or on compacted granular fill, following removal of topsoil, existing fill materials, and loose soil, at the foundation elevations given in the table below. The new compacted granular fill should consist of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II, extending at least 1 m beyond the edges of the footing(s), then outward and downward at 1 horizontal to 1 vertical (1H:1V). The granular fill should be placed in accordance with OPSS.PROV 501 *(Compacting).* Alternatively, lean concrete having a minimum 28-day compressive strength of 10 MPa may be placed below the footings, extending at least 0.5 m beyond the edges of the footings.

Structure	Reference Boreholes	Approximate Average Finished Grade Elevation (m)(1)	Proposed Founding Elevation (m)(1)	Proposed Sub-Excavation Elevation (to extend below existing fill/weak soils) (m)	Anticipated Subgrade Soils
CVIF building	19-4	96.1	94.4	92.6	1.8 m thick new compacted granular fill or lean concrete over compact native silt
Triage canopy	19-7 & 1	95.5	93.3	93.1	0.2 m thick new compacted granular fill or lean concrete over loose to compact native sandy silt or firm to very stiff native silty clay
Static scale	19-5	N/A(2)	93.2(2)	92.5	0.7 m thick new compacted granular fill or lean concrete over compact to very dense native sand
Inspection canopy and bays	19-6, 19-8, 19-9A, 19-9B, 16-1 & 16-2	96.0	93.7	93.7	Varies: very stiff native silty clay or loose to compact native silt to sandy silt
Garage	16-1	96.1	94.4	94.4	Very stiff native silty clay

Notes:

1) Elevations are based on drawings received from Dillon on September 16, 2019; these founding levels place the footings below surficial loose or softened soils as encountered in the boreholes.
2) It is understood from Dillon that the static scale will be a propriety item obtained by the Contractor and as such, the exact geometry/foundation details are unknown at this time; founding elevation is assumed to be 1.5 m below ground surface for frost protection, and assumes removal of 0.7 m of loose sandy silt.
3) Inspection canopy and bays will be supported on footings with proposed founding elevation at 1.5 m below founding grade.

6.3.2 Axial Geotechnical Resistances

Foundations constructed on the properly prepared subgrade should be designed using the factored ultimate axial geotechnical resistance and the factored serviceability geotechnical resistance (for 25 mm of settlement) as outlined in the table below.

Structure	Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement) (kPa)
CVIF building	0.6 to 1.2	200	150
Triage canopy	1.8	250	140
Static scale	3.7 (assumed based on information from Dillon)	300	150
Inspection canopy and bays	2.0	300	240
	2.5	310	200
	3.0	320	160
Garage	0.6	200	150

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances should be reviewed if the footing width is greater than that specified above or if the founding elevation differs from that given in Section 6.3.1. The factored ultimate geotechnical resistances provided are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, eccentricity and inclination of the load should be considered.

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill or other unsuitable material have been removed.

The loose to compact sandy silt subgrade could be susceptible to disturbance and degradation on exposure to water and construction traffic. If the concrete footings will not be poured within the working shift after excavation to the founding level, it is recommended that a working slab of 100 mm thickness, having a minimum 28-day compressive strength of 20 MPa be placed within four hours following inspection and approval of the subgrade, to protect the subgrade from softening/loosening.

6.3.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the concrete spread footings and the subsoil should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). The following friction factor (tan δ) values may be used from CFEM Table 24.4 for cast-in-place concrete placed on inspected and approved subgrade:

Material	Coefficient of Friction, tan δ
Cast-in-place footing or working slab on compacted granular fill	0.6
Cast-in-place footing or working slab on compact to very dense sand	0.5
Cast-in-place footing or working slab on firm to very stiff silty clay	0.4

6.3.4 Preloading for Mitigation of Differential Settlement

A grade raise of approximately 0.7 m, 1.4 m, 1.6 m and 1.8 m is proposed at the triage canopy, inspection canopy/bay, garage and the CVIF building locations, respectively. To mitigate total and differential settlement across the proposed building and canopies as a result of the presence of varying thicknesses of relatively loose silts/sands with sporadic firm clay, it is recommended that a preload, extending to the full height of the proposed grade raise, be placed in the footprint of the triage canopy, inspection canopy/bay, garage and the CVIF building (and any other areas of grade raises if applicable) with the preload remaining in place for one month prior to construction of the facility foundations.

It is recommended that fill for construction of the preload consist of granular fill or Select Subgrade Material (SSM). Where granular fill is used, it should consist of OPSS.PROV 1010 (*Aggregates*) Granular 'B' Type I or II or Granular 'A'; SSM should meet the requirements set out in OPSS.PROV 1010. Fill materials should be placed and compacted in accordance with OPSS.PROV 501 *(Compacting)* and OPSS.PROV 206 *(Grading).*

6.4 Seismic Considerations

6.4.1 General

The 2012 Ontario Building Code (2012 OBC) came into effect on January 1, 2014 and contains updated seismic analysis and design methodology. Seismic hazard is defined for an earthquake with a 2% probability of exceedance in 50 years (i.e. a return period of 2,400 years) which encompasses a larger earthquake hazard than in prior editions of the OBC. Design earthquakes are commonly defined by an earthquake magnitude, distance, and peak ground acceleration (PGA). The 2012 OBC uses the uniform hazard spectra (UHS) to define the response of the structure to the design earthquake and also considers the effects of the localized site conditions on the structural response. The 2012 OBC also uses a refined site classification system defined by the average soil/bedrock properties in the top 30 m of the subsurface profile beneath the structure(s). There are six site classes designated as A to F related to decreasing ground stiffness from A for hard rock to E for soft soil and site class F for problematic soils (e.g. sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain acceleration and velocity-based site coefficients, Fa and Fv, respectively, used to modify the reference UHS to account for the effects of site-specific soil conditions in design.

Depending on the structural design requirements for structures that fall under the OBC 2012 jurisdiction, significant structural design and construction costs can apply. Significant cost savings may be realized by adopting a more accurate site classification method which can only be determined based on actual physical testing extending to a depth of at least 30 m below the structure.

6.4.2 Conservative Approach

The conservative site classification is based on physical borehole information obtained at depths of less than 30 m and based on general knowledge of the local geology and physiography. The SPT "N"-values measured in the soil layers and the interpreted shear wave velocity of soils up to 30 m below founding level are used to define the seismic site classification.

Based on this methodology, it is considered that a Site Class D ($15 < N_{60} < 50$) would be applicable for the design of the CVIF structures in accordance with Table 4.1.8.4A of the Ontario Building Code, OBC (2012) and in the absence of any geophysical testing.

6.4.3 Geophysical Method to Refine Seismic Site Class

To determine the actual site classification based on physical on-site measurements of shear wave velocity as required by OBC 2012, the Multichannel Analysis of Surface Waves (MASW) can be utilized. It is noted that a higher (improved) Site Class is not necessarily guaranteed.

6.4.4 Liquefaction Assessment

Liquefaction is a phenomenon whereby seismically induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as "lateral spreading" or under certain conditions even catastrophic failure of the slope often referred to as "flow slides". Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The liquefaction susceptibility of the soils at the proposed CVIF facility was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required. The methodology used to assess liquefaction potential at the site is consistent with the approach outlined in the CHBDC and by Idriss and Boulanger (2008). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil. The CRR values with depth were calculated using the CPT data collected as part of the 2019 investigation. Where available, the data collected from CPTs is typically more reliable for assessment of liquefaction in loose granular deposits, as it significantly reduces the effects of sample disturbance possible during advancement of SPTs.

The analysis considered a design groundwater level ranging from Elevation 93.6 to 93.0 metres, based on the groundwater levels encountered in the open boreholes and standpipe piezometer. The CRR with depth was estimated at each CPT location as outlined in the Commentary to the CHBDC using the parameter, q_{c1Ns}, and the fines content, based on the results of particle size distribution testing carried out on samples obtained from the adjacent boreholes.

The results of the liquefaction assessment using the approach described above indicate that the site soils have a low potential for liquefaction and may be considered non-liquefiable for design.

6.5 Slab-on-Grade Floor

The proposed slab-on-grade floors for the CVIF and garage buildings are anticipated to be founded on compacted granular fill over the native subgrade soil or existing fill. Prior to the placement of the compacted granular fill, all topsoil, organic material, or loosened soil should be stripped from below the proposed slab-on-grade in accordance with OPSS 206 *(Grading)*; this should be reflected in the Contract Documents. The exposed subgrade should be inspected by the Foundation Engineering Specialist, and remedial work (e.g., further sub-excavation and replacement) should be carried out on disturbed zones as directed by the Foundation Engineering Specialist.

The floor areas should be brought to within 200 mm of the underside of the floor slab, as required, using OPSS.PROV 1010 Granular ‘B’, Type I material, placed in maximum 200 mm loose lifts and uniformly compacted to at least 98 % of the material’s Standard Proctor Maximum Dry Density (SPMDD). The final lift directly beneath conventionally loaded floor slabs should consist of a minimum of 200 mm of OPSS.PROV 1010 Granular ‘A’ material, compacted in accordance with OPSS.PROV 501 (*Compacting*).

A polyethylene vapour barrier is recommended between the Granular A layer and the concrete, unless uncontrolled migration of water vapour through the slab is acceptable. It is recommended that the floor slab be designed and constructed to be structurally separate from the foundation walls and columns, and that sawcut control joints be provided at regular intervals and along column lines to minimize shrinkage cracking and to allow for differential settlement of the floor slab.

6.6 Design of Sign Support Foundations

6.6.1 General

Construction of the footing or caisson foundation(s) for sign support structures should be in accordance with OPSS.PROV 915 (*Sign Support Structures*) and OPSS.PROV 903 (*Deep Foundations*).

6.6.2 Static Overhead Sign

The proposed tri-chord static overhead sign support at Station 11+500 will be supported on concrete caisson foundations. The standard foundation design for tri-chord static sign supports is outlined in Division 4 of MTO’s *Sign Support Manual* (2019) and on Standard Drawings SS118-3, SS118-4 and SS118-5 (*Static Sign Support – Footing Details*).

In the standard caisson foundation design, depending on the sign class and corresponding caisson diameter, the caisson is extended 5 m to 6.5 m below the design frost depth, which for this site is 1.5 m as interpreted from OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*), resulting in a total caisson length of 6.5 m to 8.0 m below the final grade. The standard sign foundation designs presented in MTO’s Sign Support Manual have been developed based on the minimum soil conditions given below:

- **Case 1 (Non-Cohesive Soils):** Sand with a friction angle of 28 degrees surrounding the upper two-thirds of the portion of the caisson foundation below the frost depth, and sand with a friction angle of 30 degrees surrounding the lower third of the portion of the caisson below the design frost depth.
- **Case 2 (Cohesive Soils):** Soft clay with an undrained shear strength of 25 kPa surrounding the upper two-thirds of the portion of the caisson foundation below the frost depth, and “soft” clay with an undrained shear strength of 50 kPa surrounding the lower third of the portion of the caisson below the design frost depth.

Based on a review of the subsurface conditions encountered in Boreholes 19-2 and 19-3 advanced at the approximate location of the proposed overhead sign support, the founding conditions have an internal friction angle and undrained shear strength equal to or greater than the input parameters used in the modelling of the standard footing design for non-cohesive (i.e., sand) and cohesive (i.e., soft clay) soils respectively, and therefore, the standard footing foundation design is suitable for the proposed sign support, provided that the sign board surface area also meets the standard requirements. If a larger sign board is adopted, a site-specific foundation design will be required.

6.6.3 Steel Column Breakaway Sign

As per MTO's *Sign Support Manual* (2019), a standard caisson foundation design for the steel column breakaway sign at Station 10+650 is not available and a site-specific design is required. The geotechnical parameters required for the site-specific design for the proposed breakaway sign are included in Table 1.

6.7 Construction Considerations

6.7.1 Excavations and Groundwater Control

Excavations for the foundations will extend through the surficial topsoil, existing fill materials, and the loose native silts, sands and silty sands where applicable. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities (Ontario Regulation 213/91). The existing fill materials and native granular soils above the groundwater level are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V. Granular soils (i.e., silts and sands) below the water table would be classified as Type 4 soil, based on OSHA, and excavations in these materials should be sloped no steeper than 3H:1V.

For the CVIF structures, depending on the time of year of construction, excavations for the foundations may extend below the groundwater level. Further, perched water may be present within the existing fill. As such, some form of groundwater control and dewatering will be required if the excavation base is within 0.6 m of the prevailing groundwater level at the time of construction. It is anticipated that the dewatering at this site can be achieved by gravity drainage and pumping from strategically placed and properly filtered sumps with side ditches.

For the overhead sign support at Station 11+500, the water-bearing cohesionless soils at this site should be expected to run or flow into the caisson holes during or after drilling of the caisson foundations. Therefore, appropriate equipment and procedures will be required to minimize ground loss during drilling and concrete placement, such as by using temporary or permanent caisson liners, and/or using drilling mud. It is recommended that a Notice to Contractor be included in the Contract Documents to warn the Contractor of this condition; such an NTC is provided in Appendix E.

Dewatering of all excavations should be carried out in accordance with OPSS.PROV 517 *(Dewatering)*, as modified by Special Provision (SP) 517F01 and SP FOUN0003 *(Dewatering of Structure Excavations)*. Given the presence of existing infrastructure in the vicinity of the site, a preconstruction condition survey should be carried out over a limited distance/radius from the site to capture all structures. As such, the foundation designer fill-in in Table A of (SP) 517F01 should indicate a distance of 150 m. If sensitive structures are identified to be present in the area (e.g. drinking water wells), consideration should be given to expanding the condition survey radius as may be warranted in consideration of the Contractor's dewatering operations and MTO's experience. In addition, the foundation insert requiring a minimum of 5 years experience for the dewatering system design engineer and design-checking engineer should be included in (SP) 517F01. These fill-ins should be completed by the design team during preparation of the contract package. The design and construction of the groundwater control systems is the responsibility of the Contractor.

The piezometer installed in Borehole 19-4 should be decommissioned during construction and a Non-Standard Special Provision (NSSP) should be added to the Contract Documents; an NSSP for this purpose is attached in Appendix E.

6.7.2 Obstructions

As discussed in Section 3, at about 1 m south of DCPT 19-9B, 5.2 m of augers were abandoned in the ground. The Contractor should be alerted of the presence of buried augers at this location, as the augers could affect excavations for the foundations and pavement structure. In addition, shallow refusal was encountered in CPTs 19-10 and 19-10B. A Notice to Contractor (NTC) should be included in the Contract Documents to identify to the Contractor the presence of buried augers within the fill and overburden soils as well as the shallow refusal at the two CPTs. An example NTC is included in Appendix E.

6.7.3 Preloading

As discussed in Section 6.3.4, a one-month preloading period is proposed within the footprint of the triage canopy, inspection canopy/bay, garage and the CVIF building locations to mitigate the total and differential settlement that will occur as a result of the grade raise over the existing relatively loose silts/sands and occasional firm clay layers. If this duration will present challenges for the construction schedule, other mitigation or management approaches can be adopted. If preloading is adopted, an Operational Constraint (OC) should be included in the Contract Documents to identify the one-month preloading period following completion of the grade raise and prior to construction of the foundations for these facilities. An example OC is included in Appendix E.

6.7.4 Recommendations for Construction Materials Based on Analytical Testing

The results of analytical testing completed on four samples, one sample of the native sand, one of the native silty sand and one of the native silt, are summarized in Section 4.2.12 and presented in Appendix C. The potential for sulphate attack and corrosion are discussed in the following paragraphs. However, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class and ensuring that all aspects of CSA A23.1-24 Section 4.1.1 "*Durability Requirements*" are followed when designing concrete elements.

The potential for sulphate attack on concrete was determined by comparing analytical test results to CSA A23.1-14 Table 3 "*Additional Requirements for Concrete Subjected to Sulphate Attack*". The water-soluble sulphate concentration measured in the native sand, native silty sand and native silt were all below 0.1 per cent, which is below the exposure class of S-3 (Moderate). Therefore, based on the test results when the designer is selecting the exposure class for the structure in contact with the native sand, native silty sand or native silt the effects of the sulphates may not need to be considered. Additionally, given the location of the structure along Highway 401, it may be exposed to de-icing salts and selection of the exposure class should consider this.

The native sand has a pH of 8.8 and a resistivity of 5620 ohm-cm. According to the MTO Gravity Pipe Guidelines, the pH is considered detrimental to structure durability as it is greater than a pH of 8.5. The resistivity is greater than 4,500 ohm-cm and less than 6,000 ohm-cm, which indicates that the soil corrosiveness is low (6,000 ohm-cm < R < 4,500 ohm-cm), as per Table 3.2 "*Soil Corrosiveness and Resistivity*" of the MTO Gravity Pipe Guidelines.

The native silty sand and native silt have a pH ranging between 8.2 and 8.0 and a resistivity ranging between 830 ohm-cm and 370 ohm-cm. According to the MTO Gravity Pipe Guidelines, the pH is not considered detrimental to structure durability as it is less than a pH of 8.5 but greater than a pH of 5.5. The resistivity is less than 2,000 ohm-cm, which indicates that the soil corrosiveness is severe (R < 2,000 ohm-cm), as per Table 3.2 "*Soil Corrosiveness and Resistivity"* of the MTO Gravity Pipe Guidelines.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Mo'oud Nasr, P.Eng., a geotechnical engineer with Golder and the technical aspects were reviewed by Mrs. Sarah E. M. Poot, P.Eng., Associate of Golder and the Senior Foundation Engineer for this project. Ms. Lisa C. Coyne, P.Eng., a Principal of Golder and MTO Foundations Designated Contact, conducted an independent technical and quality control review of the report.

Signature Page

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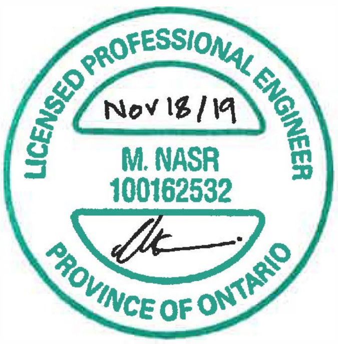

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Ontario Ministry of Transportation, 2019. *Sign Support Manual*. Provincial Highways Management Division, Highway Standards Branch, Bridge Office. February 2019.

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ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D7012	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Ontario Provisional Standard Drawing:

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario

Ontario Provincial Standard Specification:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering of Pipeline, Utility and Associated Structure Excavation
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 915	Construction Specification for Sign Support Structures
OPSS.PROV 1010	Construction Specification for Aggregates

SP 517F01	Amendment to OPSS 517, July 2017
SP FOUN0003	Dewatering of Structure Excavations

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

TABLE 1
GEOTECHNICAL DESIGN PARAMETERS FOR BREAKAWAY SIGN FOUNDATION
Gananoque South Commercial Vehicle Inspection Facility, GWP 4046-10-01

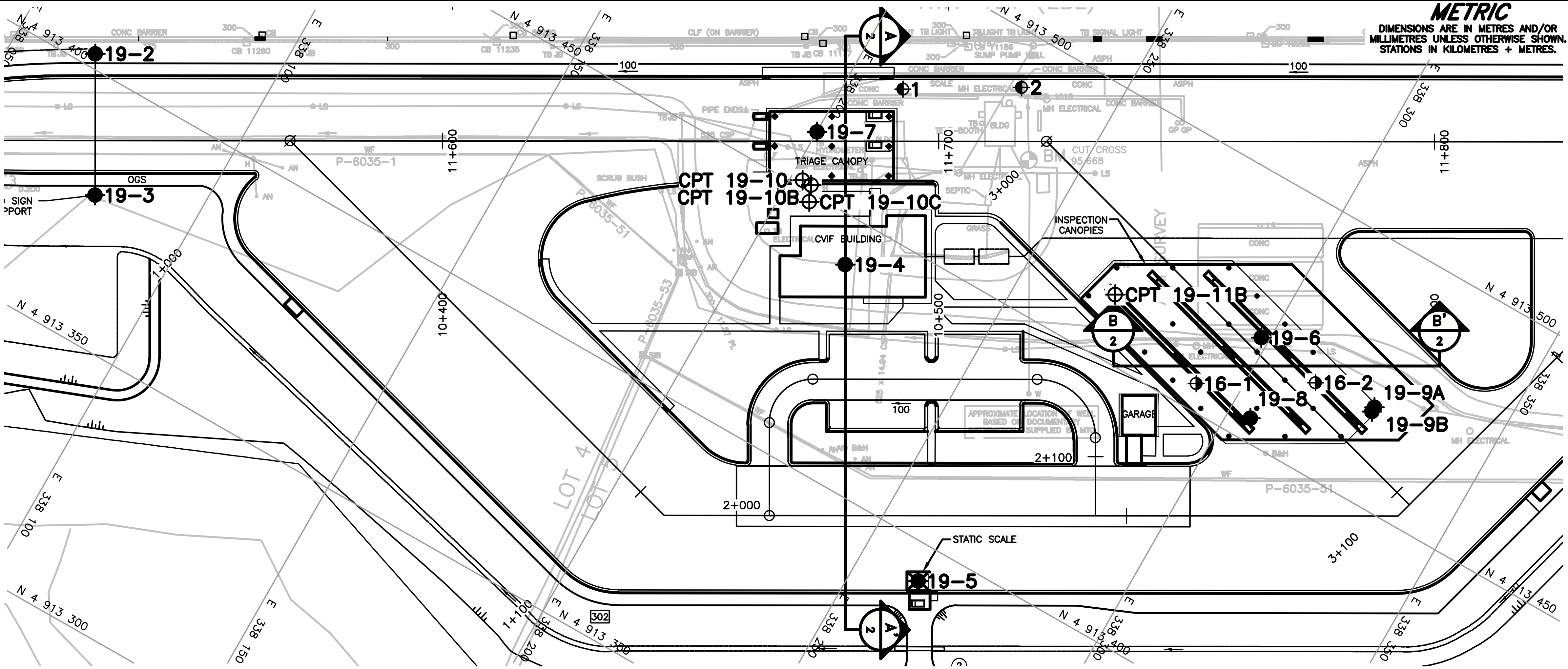
Sign ID (Location)	Reference Borehole	Ground Surface Elevation at Reference Borehole (m)	Assumed Ground Surface Elevation at Sign Foundation Location (m)	Standard or Site-Specific Foundation Design	Stratum	Depth Relative to Proposed Ground Surface (m)[1]	Elevation (m)	Groundwater Elevation (m)	Design Parameters[2, 3]				
									S_u (kPa)	Φ'	γ (kN/m^3)	γ' (kN/m^3)	K_p
Steel Column Breakaway Sign (Sta. 10+650)	19-1	85.9	85.9	Site-Specific	Loose sand and gravel fill	0.0 - 0.7	85.9 - 85.2	84.4	--	30	20	10	3.0
					Very stiff silty clay fill	0.7 - 2.2	85.2 - 83.7		100	26	19	9	2.6
					Stiff clay	2.2 - 4.5	83.7 - 81.4		75	25	19	9	2.5
					Stiff clayey silt	4.5 - 7.7	81.4 - 78.2		75	28	19	9	2.8

NOTES:

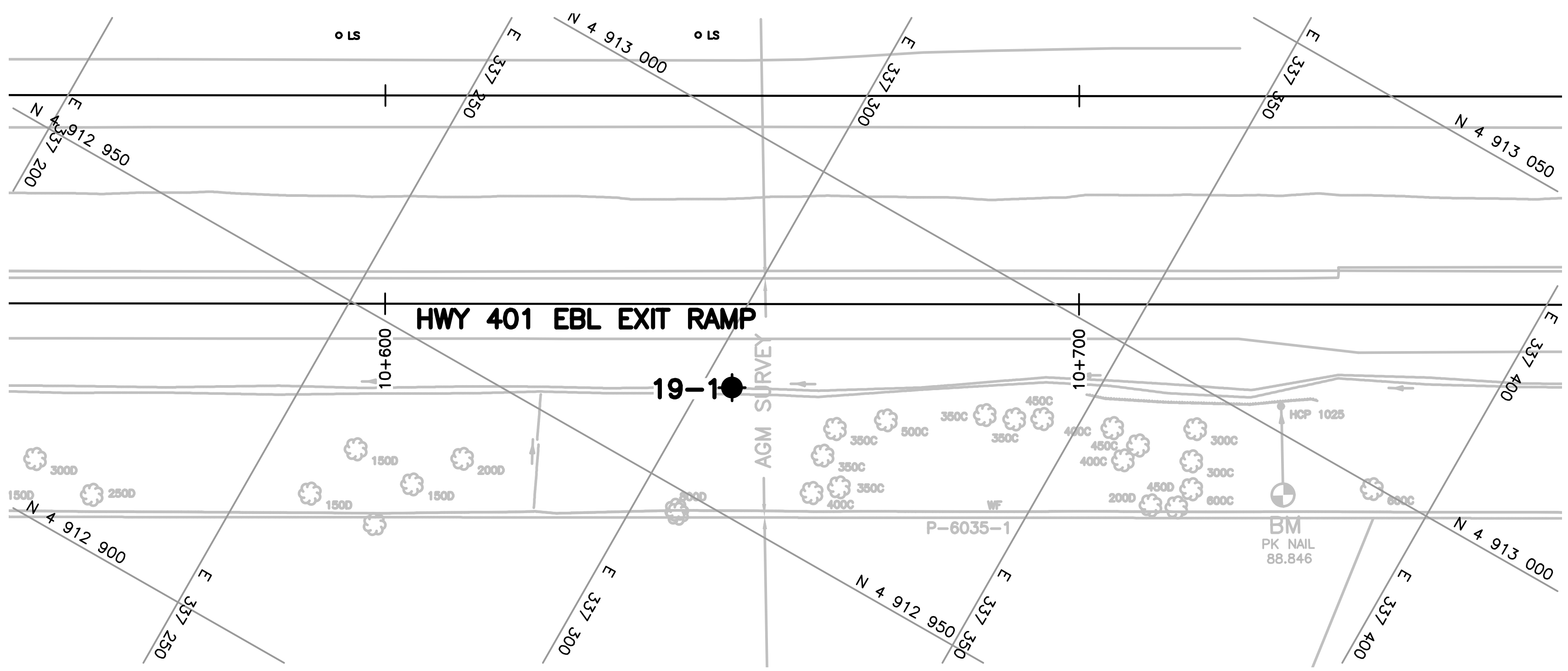
1. Depths are given at the proposed sign support locations relative to the existing ground surface. Although Su, ϕ' and Kp parameters are given for the full depth of the soil, the passive resistance in the upper 1.3 m should be neglected. to account for frost action.
2. Design parameters:
   - s_u = undrained shear strength (kPa);
   - φ' = effective friction angle (degrees);
   - γ = bulk unit weight (kN/m3);
   - γ' = effective unit weight below the groundwater level (kN/m3);
   - K_p = passive earth pressure coefficient; and
   - f_{horiz} = factored lateral geotechnical resistance of sound rock at Ultimate Limit States (kPa).
3. Where both undrained shear strength and effective friction angle parameters have been provided for fill materials, the structural assessment should be completed for both cohesive soil and cohesionless soil cases, and the selected design should be based on the more conservative approach.

Prepared by: MN
Reviewed by: AB

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PLAN
SCALE
10 0 10 20 m



PLAN
SCALE
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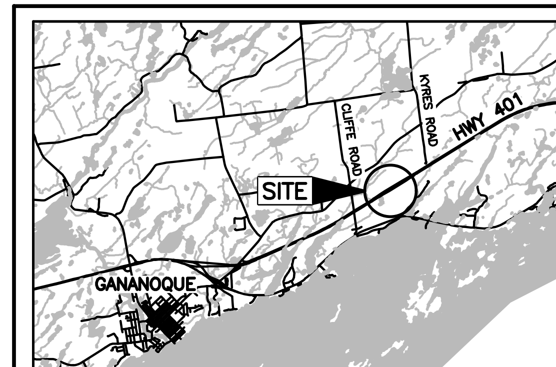
CONT No. 2020-4017
GWP No. 4009-14-00

COMMERCIAL VEHICLE INSPECTION FACILITY
GANANOQUE SOUTH
BOREHOLE LOCATION PLAN

SHEET 53

GOLDER



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- Borehole – 2016 Investigation
- Borehole – 1991 Investigation
- Cone Penetration Test

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
1	95.3	4913477.4	338210.8
2	95.3	4913489.6	338231.4
16-1	94.6	4913455.2	338291.6
16-2	94.7	4913467.3	338312.4
19-1	85.9	4912966.4	337304.0
19-2	95.2	4913402.8	338065.9
19-3	94.3	4913378.0	338080.0
19-4	94.8	4913440.9	338218.3
19-5	94.7	4913392.8	338262.7
19-6	94.4	4913469.8	338298.4
19-7	95.3	4913461.3	338200.0
19-8	94.5	4913454.6	338304.6
19-9A	94.6	4913468.9	338325.2
19-9B	94.6	4913467.9	338325.2
CPT 19-10	94.9	4913451.4	338202.3
CPT 19-10B	94.9	4913451.5	338204.3
CPT 19-10C	94.9	4913448.3	338205.7
CPT 19-11B	94.9	4913462.7	338268.5

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

REFERENCE

Base plans provided in digital format by Dillon, drawing file nos. 4009-Base.dwg and Alignments.dwg, received May 10, 2019 and 4009-New Construction.dwg, received September 16, 2019.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Sept. 17, 2019 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER M. NASR 100162532 Sept. 17, 2019 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31C-285

HWY. 401	PROJECT NO. 1780055		DIST.
SUBM'D. MN	CHKD. MN	DATE: 09/17/2019	SITE:
DRAWN: DD	CHKD. AB	APPD. LCC	DWG. 1

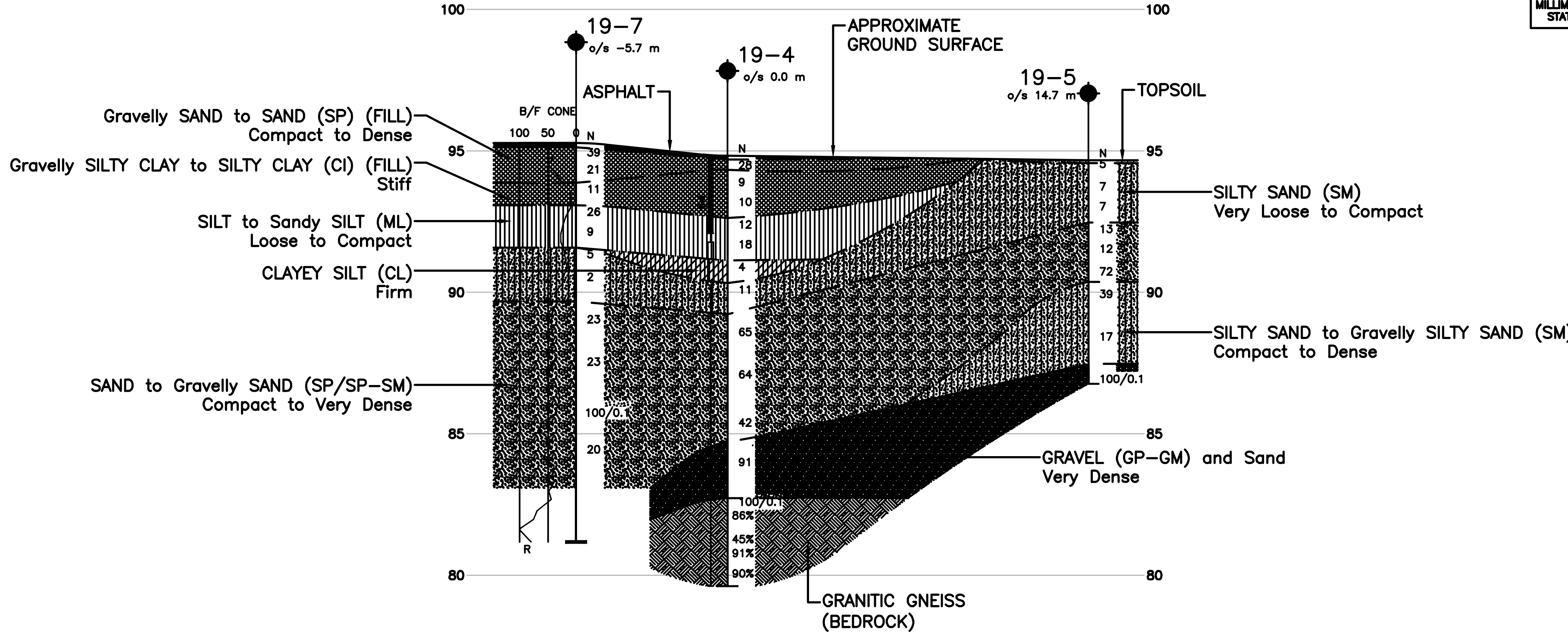
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2020-4017
GWP No. 4009-14-00

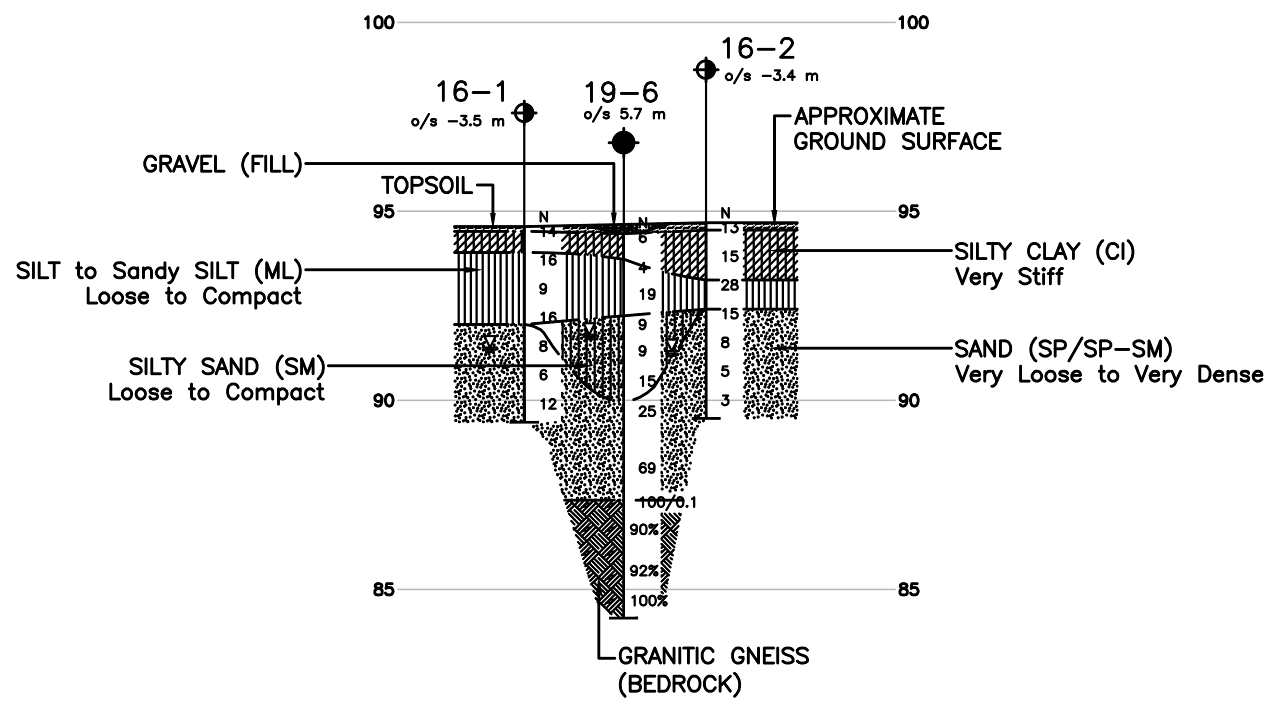
COMMERCIAL VEHICLE INSPECTION FACILITY
GANANOQUE SOUTH
SOIL STRATA

SHEET 54

GOLDER



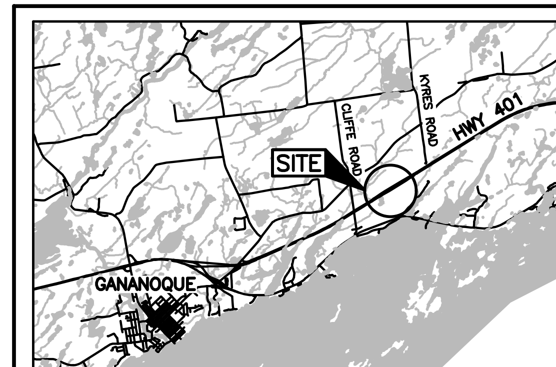
CROSS SECTION A–A'



CROSS SECTION B–B'

HORIZONTAL SCALE 10 0 10 20 m

VERTICAL SCALE 2 0 2 4 m



KEY PLAN
SCALE
10 0 10 20 m

LEGEND

- Borehole – Current Investigation
- Borehole – 2016 Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- R Refusal
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on May 9, 2019
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
16-1	94.6	4913455.2	338291.6
16-2	94.7	4913467.3	338312.4
19-4	94.8	4913440.9	338218.3
19-5	94.7	4913392.8	338262.7
19-6	94.4	4913469.8	338298.4
19-7	95.3	4913461.3	338200.0

NOTES

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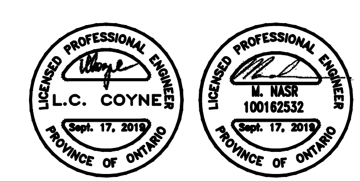
REFERENCE

Base plans provided in digital format by Dillon, drawing file nos. 4009-Base.dwg and Alignments.dwg, received May 10, 2019 and 4009-New Construction.dwg, received September 16, 2019.



NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31C-285			
HWY. 401		PROJECT NO. 1780055	DIST. .
SUBM'D. MN	CHKD. MN	DATE: 09/17/2019	SITE: .
DRAWN: DD	CHKD. AB	APPD. LCC	DWG. 2

APPENDIX A

Borehole Records – Previous Investigations (1991 & 2016)

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_p	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_p	%	PLASTICITY INDEX $= w_L - w_p$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1

METRIC

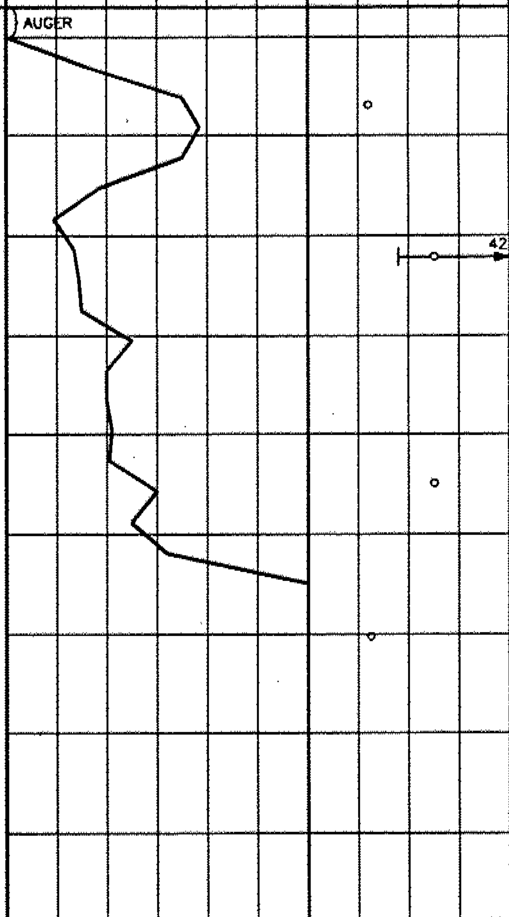
W.P. 2501-91-01/02	LOCATION Co-ords: N 4 913 254.7; E 338 186.2	ORIGINATED BY G.D
DIST 8 HWY 401	BOREHOLE TYPE H S Auger and Cone Test	COMPILED BY L.D
DATUM Geoditic	DATE 91 04 25	CHECKED BY T.K

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
95.3	Ground Surface								
0.0 / 94.7	Asphalt					95			
0.6 / 93.4	Fine Sand, some silt (Fill) Compact to Dense; Brown	1	SS	50		94			1 79 13 7
		2	SS	11					
1.9 / 91.9	Grey; Silty Clay, some sand, trace of organics; Firm to V. Stiff; Grey	3	SS	16	▼*	93	42		0 9 51 40
		4	SS	9		92			
3.4 / 90.1	Brown; Sandy Silt to Silty Sand; V. Loose	5	SS	1		91			
		6	SS	9					0 19 70 11
5.2 / 86.1	Sand, some gravel occ. boulders; Compact to V. Dense	7	SS	125		90			
		8	SS	74		89			13 75 (12)
						88			
		9	SS	28		87			
9.2	End of Borehole at probable Bedrock								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30



$+^3$, $\times^5$: Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

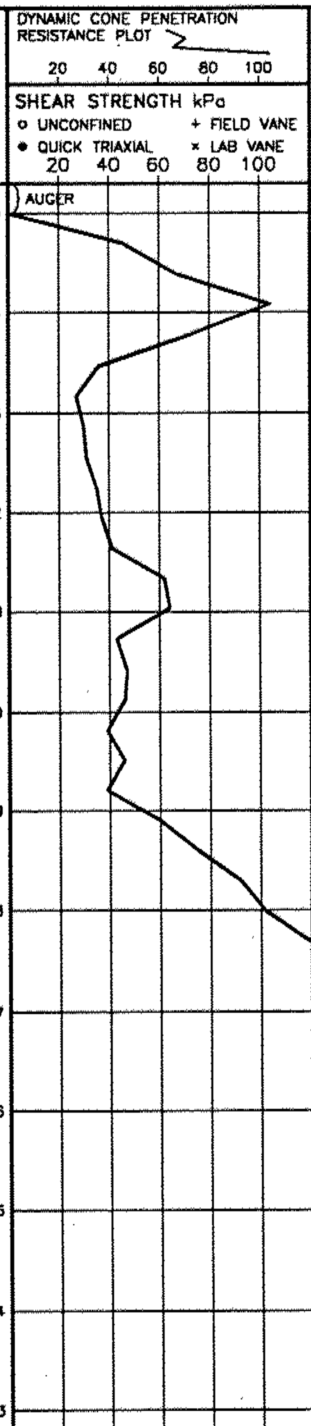
METRIC

W.P. 2501-91-01/02 LOCATION Co-ords: N 4 913 266.5; E 338 207.0 ORIGINATED BY G.D

DIST 8 HWY 401 BOREHOLE TYPE H S Auger, BXL Rock Coring & Cone Test COMPILED BY L.D

DATUM Geodetic DATE 91 04 24 CHECKED BY T.K

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
95.3	Ground Surface							
0.0 / 94.7	Asphalt					95		
0.6	Fine Sand, some silt (Fill) Compact to Dense	1	SS	41		94		
		2	SS	15	▼*			
93.2	Brown							
2.1	Grey Silty Clay, some sand, trace of organics Firm to V. Stiff	3	SS	17		93	44	0 11 55 34
		4	SS	4		92		
91.3	Grey							
4.0	Brown Sandy Silt to Silty Sand Loose to Dense	5	SS	18		91		
		6	SS	7		90		
		7	SS	42				0 53 39 8
		8	SS	12		89		
87.7						88		
7.6	Sand, some gravel Occ. boulders V. Dense	9	SS	112 /25cm		87		23 69 (8)
		10	SS	80 /3cm		86		
85.9								
9.4	Bedrock Hornblende-Biotite Gneiss with Granite of the Grenville Province	11	RC	REC 100%				RQD 92%
		12	RC	REC 87%		85		RQD 70%
		13	RC	REC 100%		84 / 83		RQD 74%
82.8 / 12.5	End of Borehole							



PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3

+3, ×3: Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:
1. $\tau = c' + \sigma' \tan\phi'$
2. shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index Relative Density	**N Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
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
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

Golder Associates

RECORD OF BOREHOLE No 16-1

 METRIC

PROJECT 1651503
G.W.P. 4046-10-01
LOCATION N 4913455.2; E 338291.6
ORIGINATED BY RI
DIST 8 HWY 401
BOREHOLE TYPE Hollow Stem Auger, Truck Mounted
COMPILED BY KL
DATUM GEODETIC
DATE July 29, 2016
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
94.6	GROUND SURFACE					
0.0 / 0.1	TOPSOIL, trace to some sand, trace grave					
	Stiff, Brown	1	SS	14		
93.9 / 0.7	Moist					94
	SILTY CLAY, trace sand, Very stiff, Brown to grey, Dry to moist	2	SS	16		
	SILT to Sandy SILT, trace clay, Loose to compact, Brown, Moist to wet	3	SS	9		93
92.0 / 2.6	SAND, some silt, Loose to compact, Brown, Wet	4	SS	16		92
		5	SS	8	∇	91
		6	SS	6		90
89.4		7	SS	12		
5.2	END OF BOREHOLE					

Note:

1. Water level at a depth of 3.2 m below ground surface (Elev. 91.4 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-2

 METRIC

PROJECT 1651503
G.W.P. 4046-10-01
LOCATION N 4913467.3; E 338312.4
ORIGINATED BY RI
DIST 8 HWY 401
BOREHOLE TYPE Hollow Stem Auger, Truck Mounted
COMPILED BY KL
DATUM GEODETIC
DATE July 29, 2016
CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.7	GROUND SURFACE						
0.0	TOPSOIL, trace to some sand						
0.2	Very stiff Dark brown Moist SILTY CLAY, trace to some sand Very stiff Brown to grey Dry to moist	1	SS	13		94	
		2	SS	15			
93.2 1.5	SILT to Sandy SILT Compact Brown Moist to wet	3	SS	28		93	0 9 83 8
92.4 2.3	SAND, some silt Very loose to compact Brown Moist to wet	4	SS	15		92	
		5	SS	8	▽		0 82 (18)
		6	SS	5		91	
		7	SS	3		90	
89.5 5.2	END OF BOREHOLE						

Column headings also printed: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

Note:

1. Water level at a depth of 3.4 m below ground surface (Elev. 91.3 m) upon completion of drilling.

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

APPENDIX B

Borehole and Drillhole Records – Current Investigation (2019)

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS
MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (*i.e.,* SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (*i.e.,* some sand)
≤ 10	trace (*i.e.,* trace fines)

1. Only applicable to components not described by Primary Group Name.
2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve friction (f_s) are recorded electronically at 25 mm penetration intervals.

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	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
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness[1]

Term	SPT 'N' (blows/0.3m)[2]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	➢ 50

3. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
4. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
U	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
E	void ratio
N	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90º angle is horizontal.

Description and Notes
An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

GOLDER

Foundation Design

RECORD OF BOREHOLE No 19-1

SHEET 1 OF 1 **METRIC**

PROJECT 1780055/ 3002
G.W.P. 4009-14-00
LOCATION N 4912966.4; E 337304.0 MTM NAD 83 ZONE 9 (LAT. 44.355985; LONG. -79.092262)
ORIGINATED BY MJB
DIST Central HWY 401
BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers
COMPILED BY MN
DATUM Geodetic
DATE April 23. 2019
CHECKED BY SEMP

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (w_P / w / w_L)	GR	SA	SI	CL
85.9	GROUND SURFACE									
0.0	SAND (SW) and gravel (FILL), Loose, Brown, Moist	1	SS	8						
85.2 / 0.7	SILTY CLAY (CI), trace sand, trace gravel (FILL), Very stiff, Brown and grey, Moist	2	SS	17	∇ (Elev. 84.4)					
		3	SS	17						
83.7 / 2.2	CLAY (CH), trace sand, Stiff, Brown to grey at 3.0 m depth, Moist to wet	4	SS	15		w_L = 52	0	1	34	65
		5	SS	14						
		6	SS	11						
81.4 / 4.5	CLAYEY SILT (CL), trace to some sand, trace gravel, Stiff, Grey to brown, Moist to wet	7	SS	9			1	2	70	27
		8	SS	14			0	16	59	25
	- Spoon bouncing at 7.7 m depth	9	SS	100/0.03						
78.2 / 7.7	END OF BOREHOLE, AUGER AND SPOON REFUSAL									

NOTES:

1. Water encountered at a depth of 3.8 m below ground surface (Elev. 82.1 m) during drilling.

2. Water measured in open borehole at a depth of 1.5 m below ground surface (Elev. 84.4 m) upon completion of drilling.

3. Borehole caved to a depth of 6.4 m below ground surface (Elev. 79.5 m) upon completion of drilling.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL. ELEVATION SCALE 85, 84, 83, 82, 81, 80, 79.

+ 3, × 3: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No 19-2

SHEET 1 OF 1 **METRIC**

PROJECT 1780055/ 3002

G.W.P. 4009-14-00 LOCATION N 4913402.8; E 338065.9 MTM NAD 83 ZONE 9 (LAT. 44.359878; LONG. -79.082677) ORIGINATED BY MJB

DIST Central HWY 401 BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers COMPILED BY MN

DATUM Geodetic DATE April 23. 2019 CHECKED BY SEMP

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
95.2	GROUND SURFACE
0.0	ASPHALT (140 mm)
0.1	Gravelly SAND (SW) (FILL) Very dense Brown Moist
94.5 0.7	SAND (SP), some gravel (FILL) Dense Brown Moist - Silty clay pockets from 1.4 m to 2.2 m depth
93.0 2.2	SAND (SP/SP-SM), trace gravel to gravelly, trace fines Compact to very dense Brown Moist to wet
87.0 8.2	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	58	
2	SS	36	
3	SS	33	
4	SS	23	
5	SS	22	
6	SS	30	
7	SS	39	
8	SS	68	24 70 (6)
9	SS	100/0.1	7 91 (2)

GROUND WATER CONDITIONS · ELEVATION SCALE (95 to 87) · DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 · SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100 · PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L · WATER CONTENT (%) 10 20 30 · UNIT WEIGHT γ kN/m³

NOTES:

1. Water encountered at a depth of 4.6 m below ground surface (Elev. 90.6 m) during drilling.

2. Water measured in open borehole at a depth of 1.8 m below ground surface (Elev. 93.4 m) upon completion of drilling.

3. Borehole caved to a depth of 3.4 m below ground surface (91.8 m) upon completion of drilling.

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No 19-3

SHEET 1 OF 1 **METRIC**

PROJECT 1780055/ 3002

G.W.P. 4009-14-00

LOCATION N 4913378.0; E 338080.0 MTM NAD 83 ZONE 9 (LAT. 44.359654; LONG. -79.082501)

ORIGINATED BY MJB

DIST Central HWY 401

BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers

COMPILED BY MN

DATUM Geodetic

DATE April 23. 2019

CHECKED BY SEMP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.3	GROUND SURFACE						
0.0	SAND (SP), trace fines, trace gravel Loose to very dense Brown Moist to wet	1	SS	5	94		
		2	SS	16	93		
		3	SS	19			
	- Some gravel from 2.3 m to 5.6 m depth	4	SS	30	92	~9	
		5	SS	27	91		
		6	SS	44	90		
		7	SS	100/0.1	89	~19	
88.7 5.6	Gravelly SILTY SAND (SM), trace clay Very dense Grey Wet	8	SS	50	88	~8	27 58 13 2
87.1 7.2	SAND (SP), trace fines, trace gravel Dense Brown Wet	9	SS	39	87	~20	2 95 (3)
86.1 8.2	END OF BOREHOLE						

NOTES:

1. Water encountered at a depth of 3.8 m below ground surface (Elev. 90.5 m) during drilling.

2. Borehole dry upon completion of drilling.

3. Borehole caved to a depth of 3.4 m below ground surface (Elev. 90.9 m) upon completion of drilling.

Column headings: GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT 1780055/ 3002

RECORD OF BOREHOLE No 19-4

SHEET 1 OF 2

METRIC

G.W.P. 4009-14-00 LOCATION N 4913440.9; E 338218.3 MTM NAD 83 ZONE 9 (LAT. 44.360214; LONG. -79.080763) ORIGINATED BY MJB

DIST Central HWY 401 BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers COMPILED BY AB

DATUM Geodetic DATE April 25, 2019 CHECKED BY SEMP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT / LIMITS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.8	GROUND SURFACE					
0.0	ASPHALT (125 mm)					
94.3 / 0.5	SAND (SW), gravelly, containing asphalt fragments (FILL) Compact Brown Moist	1	SS	28		
	SILTY CLAY (CI), some sand to sandy, rootlets (FILL) Stiff Brown and grey Moist	2	SS	9		
		3	SS	10		
92.6 / 2.2	SILT (ML), trace sand, some clay Compact Brown Moist	4	SS	12		
		5	SS	18	plotted	0 2 87 11
91.1 / 3.7	CLAYEY SILT (CL), some sand Firm Brown Moist	6	SS	4	plotted with w_P and w_L	0 11 62 27
90.3 / 4.5	SILTY SAND (SM) Compact Brown Wet	7	SS	11	plotted	
89.2 / 5.6	SAND (SP/SP-SM), trace gravel to gravelly Dense to very dense Brown Wet	8	SS	65	plotted	
		9	SS	64	plotted	28 66 (6)
		10	SS	42		
84.7 / 10.1	GRAVEL (GP-GM) and sand, some fines Very dense Brown Wet	11	SS	91	plotted	46 43 (11)
82.7 / 12.1	GRANITIC GNEISS (BEDROCK) Bedrock cored from a depth of 12.1 m to 15.2 m For bedrock coring details, refer to Record of Drillhole 19-4.	12	SS	100/0.1		
		1	RC	REC 100%		RQD = 86%
		2	RC	REC 100%		RQD = 45%
		3	RC	REC 100%		RQD = 91%
		4	RC	REC 100%		RQD = 90%

Elevation scale: 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No 19-4

SHEET 2 OF 2

METRIC

PROJECT 1780055/ 3002

G.W.P. 4009-14-00

LOCATION N 4913440.9; E 338218.3 MTM NAD 83 ZONE 9 (LAT. 44.360214; LONG. -79.080763)

ORIGINATED BY MJB

DIST Central HWY 401

BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers

COMPILED BY AB

DATUM Geodetic

DATE April 25, 2019

CHECKED BY SEMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
79.6			4	RC							RQD = 90%
15.2	END OF BOREHOLE										

NOTES:

1. Water encountered at a depth of 4.6 m below ground surface (Elev. 90.2 m) during drilling.

2. Water measured in open borehole at a depth of 1.5 m below ground surface (Elev. 93.3 m) upon completion of drilling.

3. Water level in standpipe piezometer measured as follows:

DATE	DEPTH (m)	Elev. (m)
25-4-19	1.8	93.0
09-5-19	1.7	93.1

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$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT: 1780055/ 3002

RECORD OF DRILLHOLE: 19-4

SHEET 1 OF 1

LOCATION: N 4913440.9 ;E 338218.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 25, 2019

DRILL RIG: CME 75

DRILLING CONTRACTOR: Pontil Drilling

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole 19-4				
13–15	HQ Core	Slightly weathered to fresh, slightly foliated, pink to red, coarse grained, faintly porous, GRANITIC GNEISS		82.74 12.09	1, 2, 3, 4	Bentonite
		END OF DRILLHOLE		79.62 15.21		

Depth scale markings: 13, 14, 15, 16, 17, 18, 19, 20, 21

Column headings: RECOVERY (TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.25 m (5 10 15 20); DISCONTINUITY DATA (B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja); ROCK STRENGTH INDEX (R4 R3 R2 R1); WEATHERING INDEX (W1 W2 W3 W4 W5 W6); FEATURES; R0/R1 ZONES

TYPE AND SURFACE DESCRIPTION	Jr	Ja
BC,UN,RO SA	3	2
JN,IR,RO SO	3	1
JN,PL,SM CC, Ch	1	4
BC,PL,SM PC, Ch	1	4
BC,PL,SM PC, Ch	1	4
JN,UN,RO SO	3	1
JN,CU,RO PC, M	3	4
JN,UN,RO CL	3	1
BC,PL,SM CC, Ch	1	4
FO,UN,RO SO, He	3	1
JN,ST,SM SO	2	1
JN,CU,RO PC, Ca	3	3
JN,CU,RO PC, Ca	3	3
JN,PL,RO PC, Ca	1.5	3
JN,PL,RO SA	1.5	2
JN,PL,RO SA	1.5	2
JN,PL,RO PC, He	1.5	3
JN,PL,RO SA	1.5	2
JN,CU,RO PC, Ca	3	3
JN,CU,RO PC, Ca	3	3

FEATURES LEGEND

BROKEN CORE CLAY SEAM LIMESTONE LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: MJB

CHECKED: AB

GOLDER

RECORD OF BOREHOLE No 19-5

SHEET 1 OF 1 **METRIC**

PROJECT 1780055/ 3002

G.W.P. 4009-14-00

LOCATION N 4913392.8; E 338262.7 MTM NAD 83 ZONE 9 (LAT. 44.359779; LONG. -79.080209)

ORIGINATED BY MJB

DIST Central HWY 401

BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers

COMPILED BY MN

DATUM Geodetic

DATE April 23. 2019

CHECKED BY SEMP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
94.7	GROUND SURFACE							
0.0	TOPSOIL							
0.1	SILTY SAND (SM), trace rootlets Loose Brown, oxidation staining to 1.4 m depth Moist to wet	1	SS	5				
		2	SS	7				
		3	SS	7				
92.5 / 2.2	SAND (SP), trace gravel, trace fines Compact to very dense Brown, oxidation staining Moist - Becoming coarse at a depth of 3.7 m	4	SS	13	1	94	3	2
		5	SS	12				
		6	SS	72	9	77	(3)	
90.4 / 4.3	Gravelly SILTY SAND (SM) Dense Brown Moist	7	SS	39				
89.1 / 5.6	SILTY SAND (SM), trace gravel Compact Brown Wet	8	SS	17				
87.5 / 7.2	GRAVEL (GP-GM) and sand Very dense Moist	9	SS	100/0.1				
86.8 / 7.9	- Spoon bouncing at 7.9 m depth END OF BOREHOLE AUGER AND SPOON REFUSAL							

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (94, 93, 92, 91, 90, 89, 88, 87); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED — 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

NOTES:

1. Water encountered at a depth of 1.5 m below ground surface (Elev. 93.2 m) during drilling.

2. Borehole dry upon completion of drilling.

3. Borehole caved to a depth of 4.6 m below ground surface (Elev. 90.1 m) upon completion of drilling.

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No 19-6

SHEET 1 OF 1 **METRIC**

PROJECT 1780055/ 3002
G.W.P. 4009-14-00
DIST Central HWY 401
DATUM Geodetic
LOCATION N 4913469.8; E 338298.4 MTM NAD 83 ZONE 9 (LAT. 44.360470; LONG. -79.079755)
BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers
DATE April 22, 2019
ORIGINATED BY MJB
COMPILED BY AB
CHECKED BY SEMP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.4	GROUND SURFACE							
0.0	GRAVEL (25 mm) (FILL)							
	SILTY CLAY (CI), trace sand (FILL) Firm Brown Wet	1	SS	6		94		
93.7 / 0.7	SILT (ML), some sand, trace clay Loose to compact Brown Wet	2	SS	4		93		
		3	SS	19			~21	0 13 82 5
92.2 / 2.2	SILTY SAND (SM), trace clay Loose to compact Brown Wet	4	SS	9	▽	92		
		5	SS	9		91	~26	0 65 33 2
		6	SS	15		90		
89.9 / 4.5	SAND (SP/SP-SM), trace gravel to gravelly Compact to very dense Brown Wet	7	SS	25		89		
		8	SS	69		88	~12	24 67 (9)
		9	SS	100/0.1				
87.3 / 7.1	GRANITIC GNEISS (BEDROCK) Bedrock cored from a depth of 7.1 m to 10.2 m For bedrock coring details, refer to Record of Drillhole 19-6.	1	RC	REC 97%		87, 86		RQD = 90%
		2	RC	REC 100%				RQD = 92%
		3	RC	REC 100%		85		RQD = 100%
84.2 / 10.2	END OF BOREHOLE							

NOTES:

1. Water encountered at a depth of 0.8 m below ground surface (Elev. 93.6 m) during drilling.

2. Water measured in open borehole at a depth of 2.7 m below ground surface (Elev. 91.7 m) upon completion of drilling.

3. Borehole caved to a depth of 3.0 m below ground surface (Elev. 91.3 m) upon completion of drilling.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: 19-6

PROJECT: 1780055/ 3002
LOCATION: N 4913469.8 ;E 338298.4
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 22, 2019
DRILL RIG: CME 75
DRILLING CONTRACTOR: Pontil Drilling

SHEET 1 OF 1
DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole 19-6	87.36					
	HQ Core	Fresh, slightly foliated, pink to red, coarse grained, faintly porous, strong, GRANITIC GNEISS	7.08					UCS = 179.1 MPa
				1	JN,PL,RO SA, He	1.5	2	
8					BC,UN,RO SO	3	1	UCS = 141.3 MPa
					JN,UN,RO CL	3	1	
					JN,UN,RO CC, He	3	3	
9				2				
					JN,PL,RO CL	1.5	1	
					JN,IR,RO CC, He	3	3	
				3				
10			84.24					
		END OF DRILLHOLE	10.20					
11								
12								
13								
14								
15								
16								

Column headings: SYMBOLIC LOG; RECOVERY (TOTAL CORE %, SOLID CORE %: 80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.25 m (5 10 15 20); B Angle (0 90 180 270); DIP w.r.t. CORE AXIS (0 30 60 90); ROCK STRENGTH INDEX (R4 R3 R2 R1); WEATHERING INDEX (W1 W2 W3 W4 W5 W6); FEATURES; R0/R1 ZONES

FEATURES LEGEND

BROKEN CORE   CLAY SEAM   LIMESTONE   LOST CORE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: MJB
CHECKED: AB

RECORD OF BOREHOLE No 19-7

SHEET 1 OF 2 **METRIC**

PROJECT 1780055/ 3002
G.W.P. 4009-14-00
DIST Central HWY 401
DATUM Geodetic

LOCATION N 4913461.3; E 338200.0 MTM NAD 83 ZONE 9 (LAT. 44.360398; LONG. -79.080990)
BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers, DCPT
DATE April 23, 2019

ORIGINATED BY MJB
COMPILED BY MN
CHECKED BY SEMP

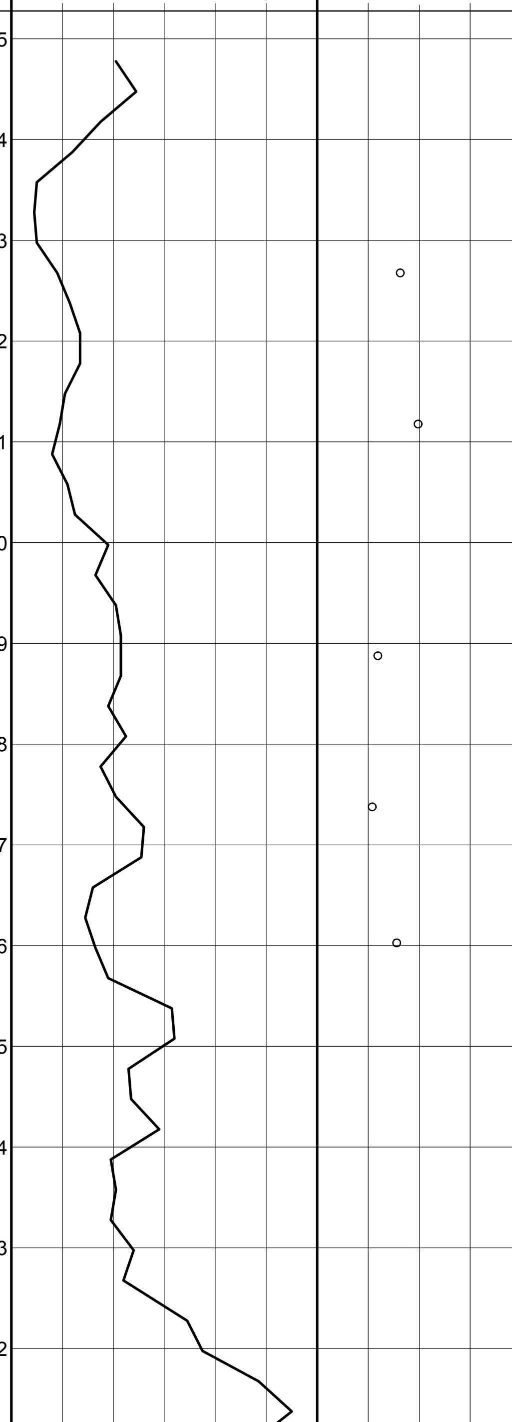
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
95.3	GROUND SURFACE
0.0	ASPHALT (150 mm)
0.2	Gravelly SAND (SP) (FILL) Dense Brown Moist
94.6 0.7	SAND (SP) (FILL) Compact Brown Moist
93.9 1.4	Gravelly SILTY CLAY (CI), rootlets (FILL) Stiff Dark brown / black Moist
93.1 2.2	Sandy SILT (ML), trace clay Loose to compact Brown Moist
91.6 3.7	SILTY SAND (SM), some gravel Very loose to loose Brown Wet
89.7 5.6	SAND (SP/SP-SM), trace gravel, trace fines Compact to very dense Brown Wet - 0.9 m of heave inside augers at 12.2 m depth
83.1 12.2	END OF SPT AT 12.2 m DEPTH - Hammer bouncing at 14.1 m depth
81.2 14.1	END OF DCPT DCPT REFUSAL

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	39	
2	SS	21	
3	SS	11	
4	SS	26	0 23 72 5
5	SS	9	
6	SS	5	0 82 12 6
7	SS	2	
8	SS	23	
9	SS	23	18 76 (6)
10	SS	100/0.1	
11	SS	20	

GROUND WATER CONDITIONS | ELEVATION SCALE (95 to 82) | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³



Continued Next Page

+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

GOLDER

Foundation Design

PROJECT 1780055/ 3002

RECORD OF BOREHOLE No 19-7

SHEET 2 OF 2

METRIC

G.W.P. 4009-14-00

LOCATION N 4913461.3; E 338200.0 MTM NAD 83 ZONE 9 (LAT. 44.360398; LONG. -79.080990)

ORIGINATED BY MJB

DIST Central HWY 401

BOREHOLE TYPE Track Mount CME 75, 216 mm O.D. Hollow Stem Augers, DCPT

COMPILED BY MN

DATUM Geodetic

DATE April 23, 2019

CHECKED BY SEMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	NOTES: 1. Water encountered at a depth of 3.8 m below ground surface (Elev. 91.5 m) during drilling. 2. Borehole dry upon completion of drilling. 3. Borehole caved to a depth of 4.9 m below ground surface (Elev. 90.4 m) upon completion of drilling. 4. Dynamic Cone Penetration Test conducted from ground surface adjacent to borehole.										

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No 19-8

SHEET 1 OF 1

METRIC

PROJECT 1780055

G.W.P. 4009-14-00

LOCATION N 4913454.6; E 338304.6 MTM NAD 83 ZONE 10 (LAT. 44.360333; LONG. -79.079679)

ORIGINATED BY MJB

DIST Central HWY 401

BOREHOLE TYPE Track Mount CME 75

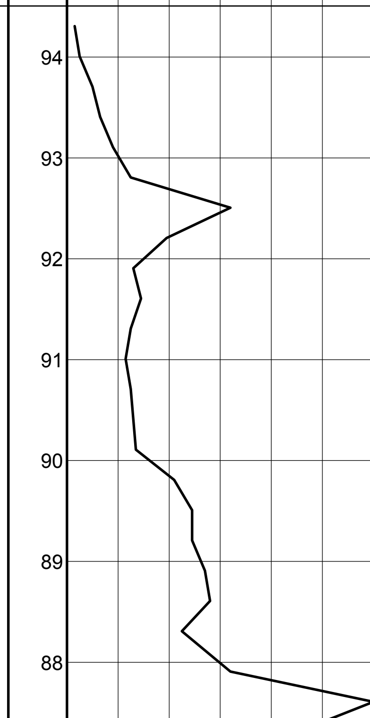
COMPILED BY MN

DATUM Geodetic

DATE April 22, 2019

CHECKED BY SEMP

SOIL PROFILE			SAMPLES								
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.5	GROUND SURFACE										
0.0	Dynamic Cone Penetration Test (DCPT)						94, 93, 92, 91, 90, 89, 88				
87.4 / 7.1	END OF DCPT DCPT REFUSAL										



$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No 19-9A

SHEET 1 OF 1

METRIC

PROJECT 1780055

G.W.P. 4009-14-00

LOCATION N 4913468.9; E 338325.2 MTM NAD 83 ZONE 10 (LAT. 44.360461; LONG. -79.079420)

ORIGINATED BY MJB

DIST Central HWY 401

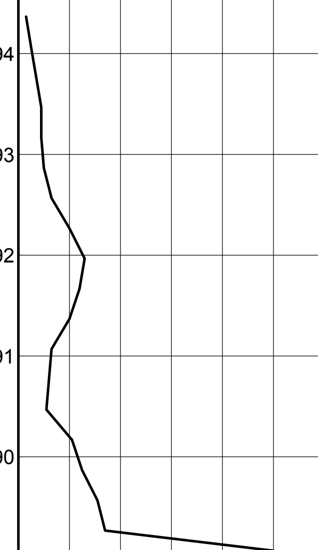
BOREHOLE TYPE Track Mount CME 75

COMPILED BY MN

DATUM Geodetic

DATE April 22, 2019

CHECKED BY SEMP

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.6	GROUND SURFACE										
0.0	Dynamic Cone Penetration Test (DCPT)						94 93 92 91 90				
89.1 5.5	END OF DCPT DCPT REFUSAL										

GTA-MTO 001 S:\CLIENTS\MTO\GANANOQUE_CVIF\02_DATA\GINT\GANANOQUE_CVIF.GPJ GAL-GTA.GDT 11/06/19

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF DCPT No 19-9B

SHEET 1 OF 1

METRIC

PROJECT 1780055

G.W.P. 4009-14-00

LOCATION N 4913467.9; E 338325.2 MTM NAD 83 ZONE 10 (LAT. 44.360452; LONG. -79.079420)

ORIGINATED BY MJB

DIST Central HWY 401

BOREHOLE TYPE Track Mount CME 75

COMPILED BY MN

DATUM Geodetic

DATE April 25, 2019

CHECKED BY SEMP

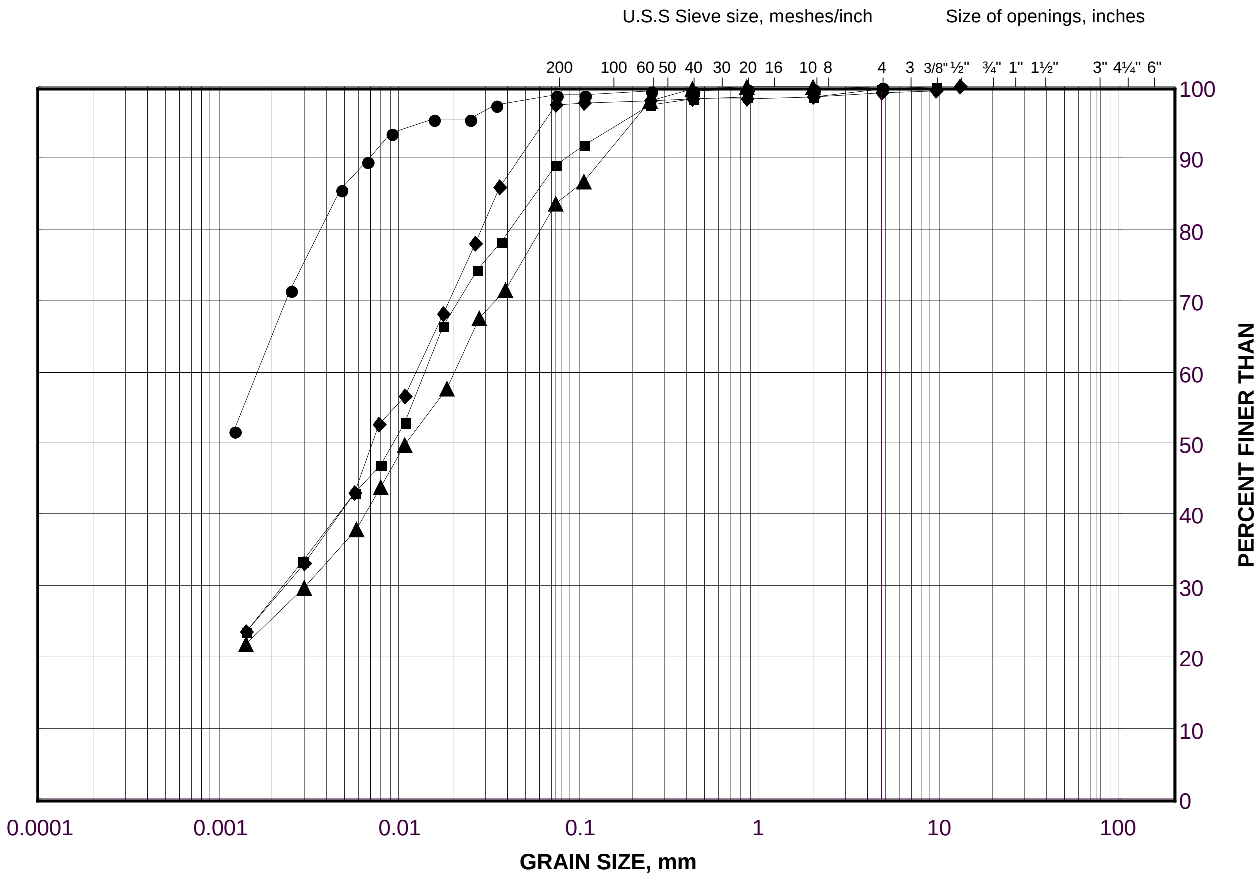
SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.6	GROUND SURFACE										
0.0	Dynamic Cone Penetration Test (DCPT)						94, 93, 92, 91, 90, 89				
89.0 5.6	END OF DCPT DCPT REFUSAL										

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

APPENDIX C

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION Clayey Silt (CL) to Clay (CH)	FIGURE C1



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	19-1	4	83.3
■	19-4	6	90.7
◆	19-1	7	81.0
▲	19-1	8	79.5

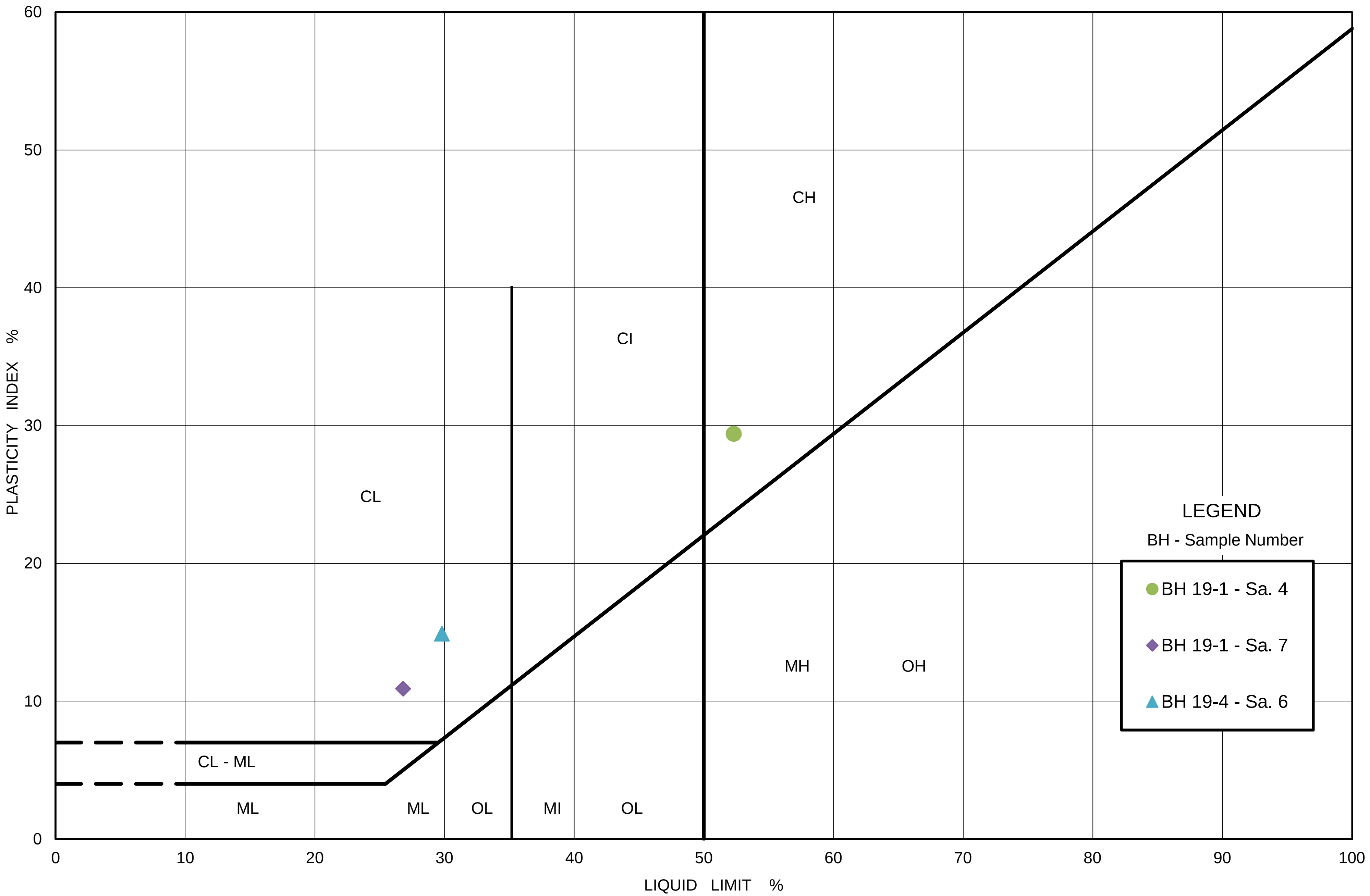
Project Number: 1780055

Checked By: MN

Golder Associates

Date: 28-Jun-19

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (MTO LS 703/704)



MINISTRY OF TRANSPORTATION Ontario

PLASTICITY CHART

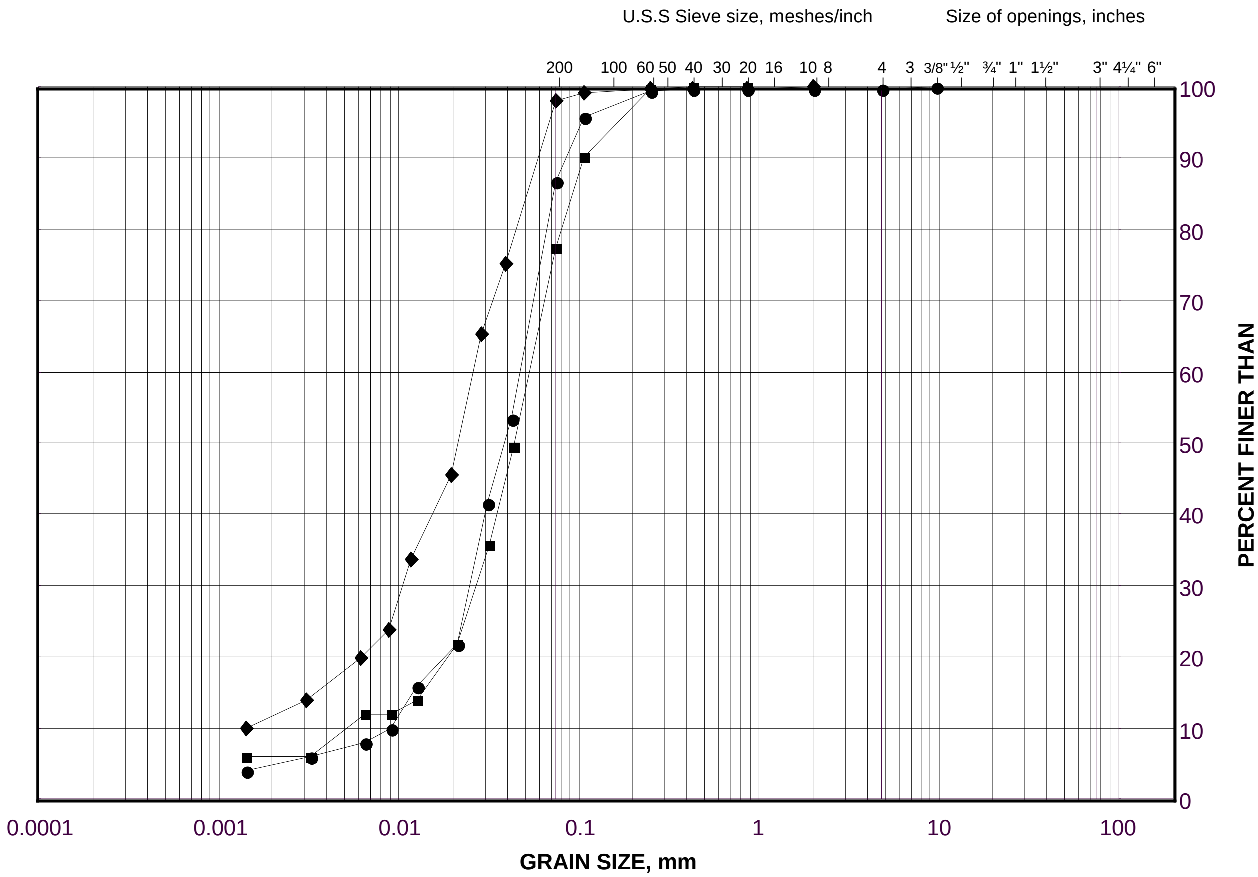
Clayey Silt (CL) to Clay (CH)

Figure No.:	C2
Project No.:	1780055
Checked By:	MN

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt (ML)

FIGURE C3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	STATION	SAMPLE	ELEVATION(m)
●	19-6	3	92.6
■	19-7	4	92.7
◆	19-4	5	91.5

Project Number: 1780055

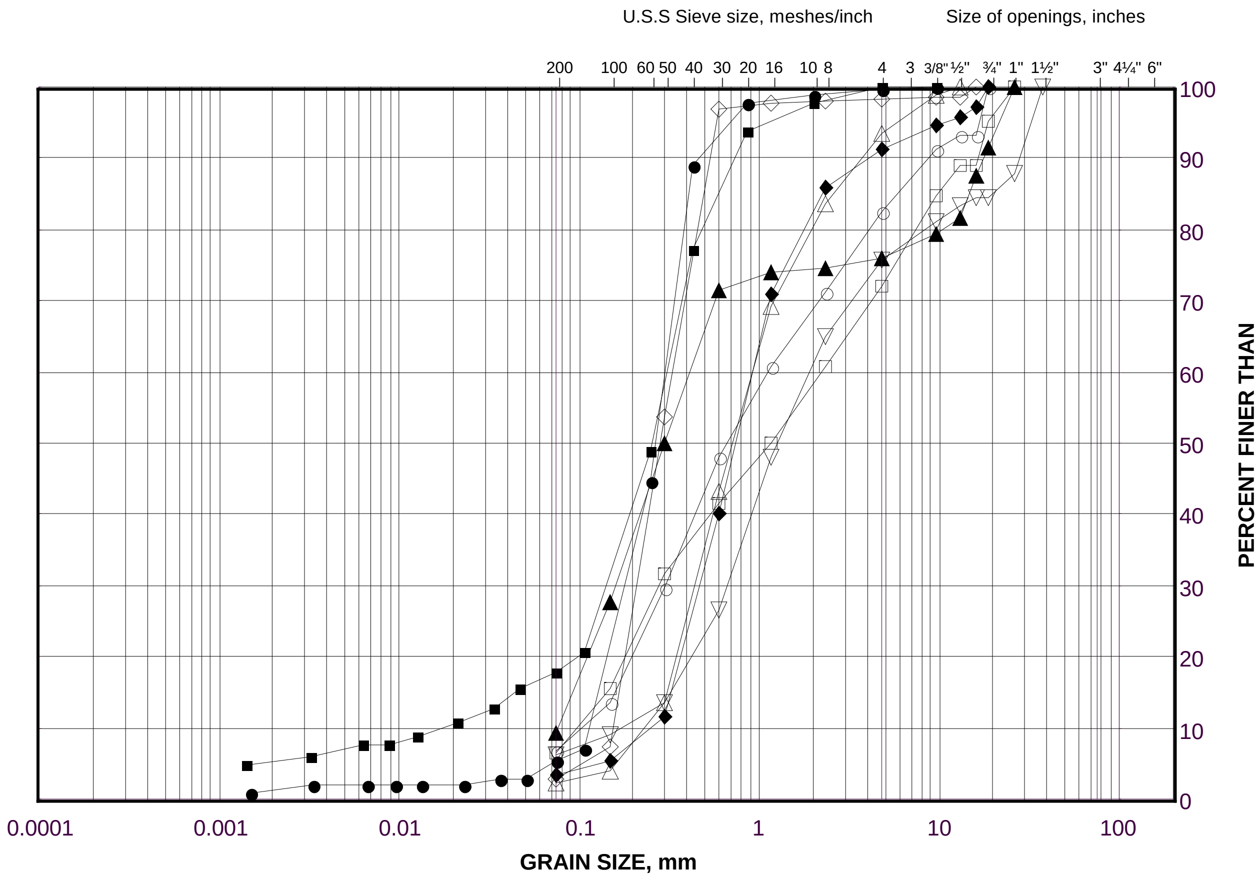
Checked By: MN

Golder Associates

Date: 28-Jun-19

GRAIN SIZE DISTRIBUTION
Silty Sand (SM) to Sand (SP/SP-SM)

FIGURE C4



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	19-5	4	92.1
■	19-7	6	91.2
◆	19-5	6	90.6
▲	19-6	8	88.0
▽	19-2	8	88.8
❍	19-7	9	87.4
☐	19-4	9	86.9
△	19-3	9	86.4
▽	19-2	9	87.4

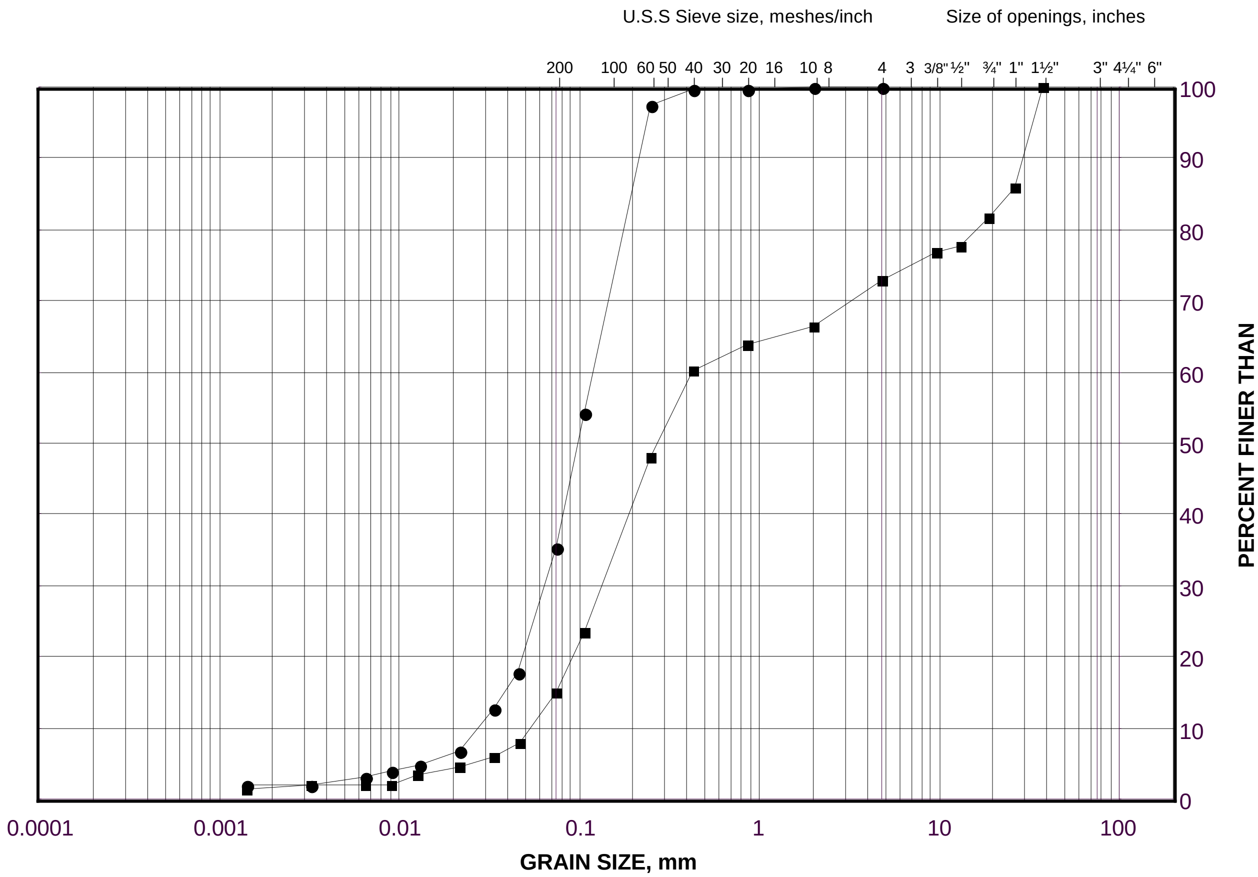
Project Number: 1780055

Checked By: MN

Golder Associates

Date: 11-Jul-19

GRAIN SIZE DISTRIBUTION Silty Sand (SM) to Sand (SP/SP-SM)	FIGURE C5



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	19-6	5	91.1
■	19-3	8	87.9

Project Number: 1780055

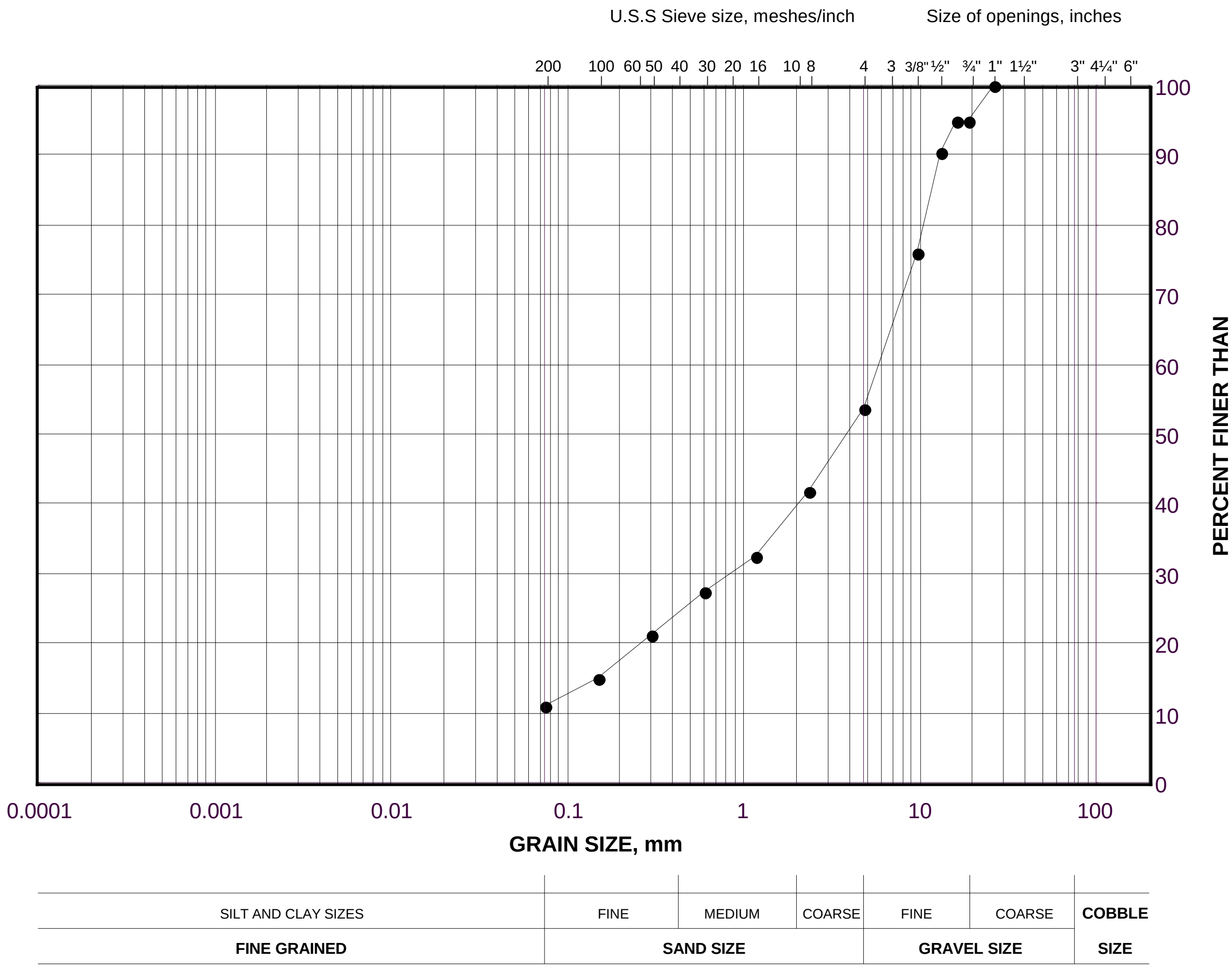
Checked By: MN

Golder Associates

Date: 11-Jul-19

GRAIN SIZE DISTRIBUTION
Gravel (GP-GM) and Sand

FIGURE C6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	19-4	11	83.8

Project Number: 1780055

Checked By: MN

Golder Associates

Date: 28-Jun-19

Borehole 19-4

Start of Run No.1 (12.09 m)

Start of Run No.4 (14.32 m)

Start of Run No.3 (13.77 m)

Start of Run No.2 (13.33 m)

Box 1: 12.09 m – 15.21 m

0 m 0.25 m 0.5 m 0.75 m 1.0 m 1.25 m 1.5 m

0 ft 1 ft 2 ft 3 ft 4 ft 5 ft

Scale

PROJECT

Dillon/Eastern CVIF Ret/4017E0003

TITLE

Bedrock Core Photographs
Borehole 19-4 (12.09 m – 15.21 m)

GOLDER

PROJECT No. 1780055			FILE No. xxxxxxxxX	
DRAFT	AB	MAY 2019	SCALE AS SHOWN	VER. 1.
CADD	--		**FIGURE C7**	
CHECK	MN	MAY 2019		
REVIEW	SP	MAY 2019		

REVISION DATE: December, 2018 BY: MPL Project: 18109417

Borehole 19-6

Start of Run No.1 (7.08 m)

Start of Run No.3 (9.27 m)

Start of Run No.2 (8.62 m)

Box 1: 7.08 m – 10.20 m

0 m | 0.25 m | 0.5 m | 0.75 m | 1.0 m | 1.25 m | 1.5 m

0 ft | 1 ft | 2 ft | 3 ft | 4 ft | 5 ft

Scale

PROJECT	Dillon/Eastern CVIF Ret/4017E0003				
TITLE	Bedrock Core Photographs Borehole 19-6 (7.08 m – 10.20 m)				
GOLDER	PROJECT No. 1780055			FILE No. xxxxxxxxX	
	DRAFT	AB	MAY 2019	SCALE AS SHOWN	VER. 1.
	CADD	--		FIGURE C8	
	CHECK	MN	MAY 2019		
	REVIEW	SP	MAY 2019		

REVISION DATE: December, 2018 BY: MPL Project: 18109417

GOLDER

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1780055	SAMPLE NUMBER	SA-02
PROJECT NAME	Dillon/Eastern CVIF Ret/4017E0003	SAMPLE DEPTH, m	7.57-7.86
BOREHOLE NUMBER	19-6	DATE:	May 6, 2019

TEST CONDITIONS

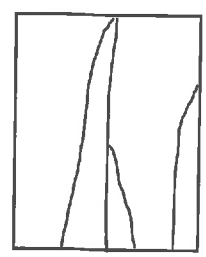
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.94	WATER CONTENT, (specimen) %	0.10
SAMPLE DIAMETER, cm	6.31	UNIT WEIGHT, kN/m^3	25.80
SAMPLE AREA, cm^2	31.30	DRY UNIT WT., kN/m^3	25.77
SAMPLE VOLUME, cm^3	436.22	SPECIFIC GRAVITY	-
WET WEIGHT, g	1148.06	VOID RATIO	-
DRY WEIGHT, g	1146.91		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	179.1

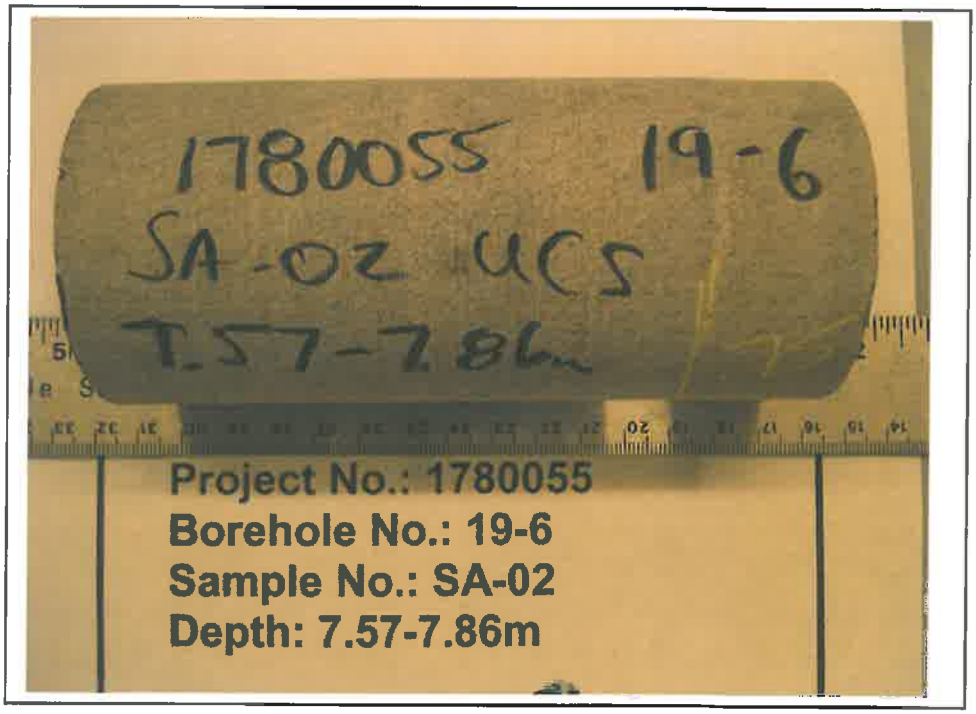
REMARKS:

Checked By: LM

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE



BEFORE COMPRESSION

AFTER COMPRESSION

Date May 6, 2019
Project 1780055

Golder Associates

Drawn Frank
Chkd. [illegible]

GOLDER

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1780055	SAMPLE NUMBER	SA-01
PROJECT NAME	Dillon/Eastern CVIF Ret/4017E0003	SAMPLE DEPTH, m	8.27-8.55
BOREHOLE NUMBER	19-6	DATE:	May 6, 2019

TEST CONDITIONS

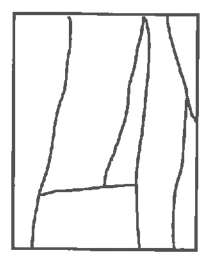
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.20

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.92	WATER CONTENT, (specimen) %	0.10
SAMPLE DIAMETER, cm	6.32	UNIT WEIGHT, kN/m^3	25.67
SAMPLE AREA, cm^2	31.32	DRY UNIT WT., kN/m^3	25.65
SAMPLE VOLUME, cm^3	435.96	SPECIFIC GRAVITY	-
WET WEIGHT, g	1141.63	VOID RATIO	-
DRY WEIGHT, g	1140.49		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	141.3

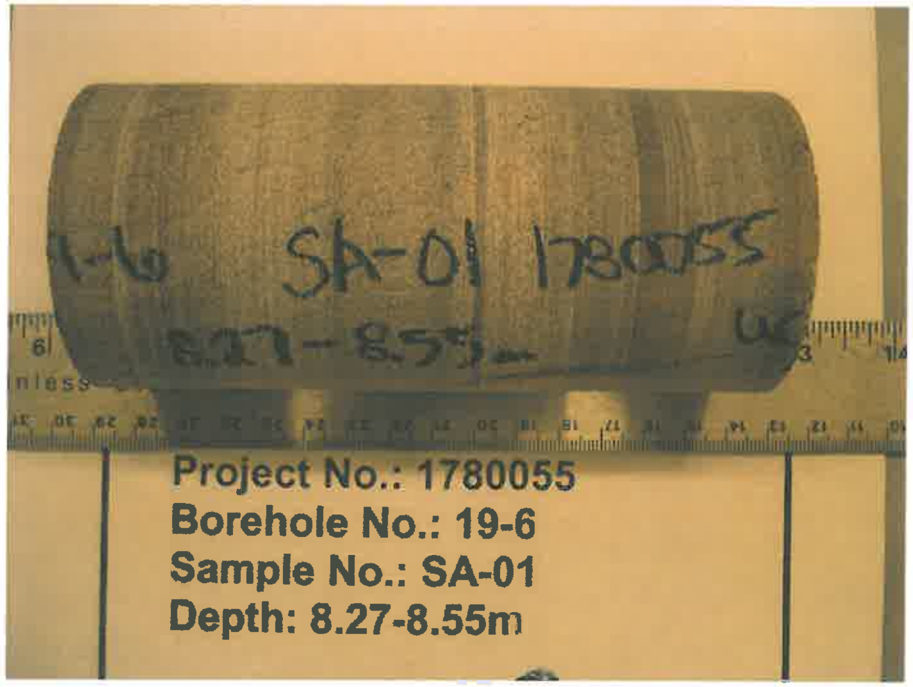
REMARKS:

Checked By: [signature]

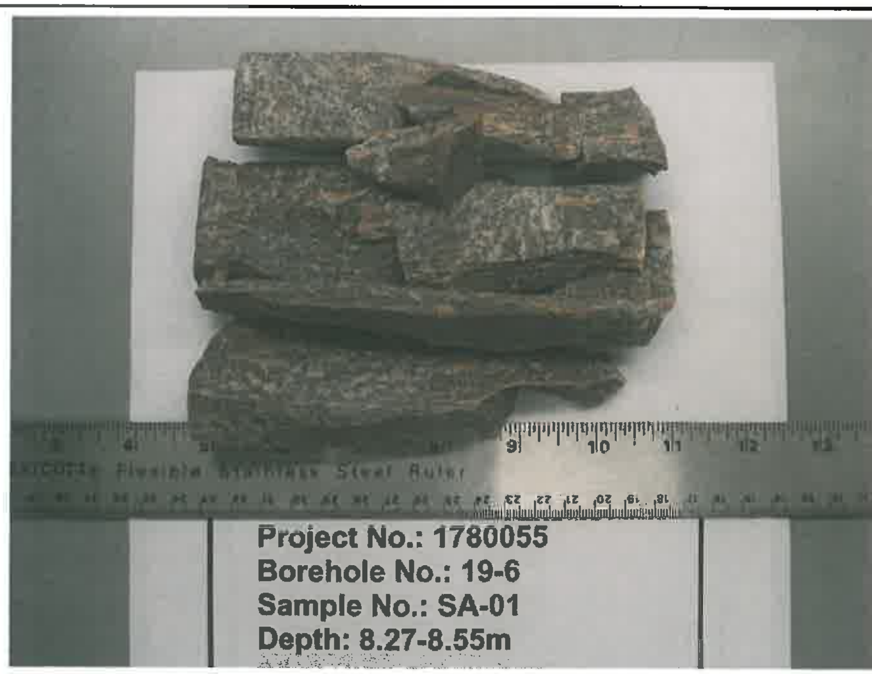
Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date May 6, 2019
Project 1780055

Golder Associates

Drawn Frank
Chkd.

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: GOLDER ASSOCIATES LTD.
100 SCOTIA COURT
WHITBY, ON L1N8Y6
(905) 723-2727

ATTENTION TO: Mike Bentley

PROJECT: 1780055

AGAT WORK ORDER: 19T464428

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Supervisor

DATE REPORTED: May 16, 2019

PAGES (INCLUDING COVER): 6

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

AGAT Laboratories

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
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FAX (905)712-5122
http://www.agatlabs.com

Certificate of Analysis

AGAT WORK ORDER: 19T464428
PROJECT: 1780055

CLIENT NAME: GOLDER ASSOCIATES LTD. **ATTENTION TO: Mike Bentley**
SAMPLING SITE: **SAMPLED BY:**

Corrosivity Package

DATE RECEIVED: 2019-05-07 **DATE REPORTED: 2019-05-16**

			SAMPLE DESCRIPTION:	19-3 Sa6		19-4 Sa4		19-6 Sa4
			SAMPLE TYPE:	Soil		Soil		Soil
			DATE SAMPLED:	2019-04-23		2019-04-25		2019-04-22
Parameter	**Unit**	**G / S**	**RDL**	**178386**	**RDL**	**178387**	**RDL**	**178388**
Sulfide (S2-)	%		0.05	<0.05	0.05	<0.05	0.05	<0.05
Chloride (2:1)	µg/g	NA	2	49	4	575	8	1430
Sulphate (2:1)	µg/g		2	5	4	28	8	20
pH (2:1)	pH Units		NA	8.87	NA	8.15	NA	7.99
Electrical Conductivity (2:1)	mS/cm	0.57	0.005	0.178	0.005	**1.20**	0.005	**2.70**
Resistivity (2:1)	ohm.cm		1	5620	1	833	1	370
Redox Potential 1	mV		NA	218	NA	281	NA	313
Redox Potential 2	mV		NA	226	NA	296	NA	319
Redox Potential 3	mV		NA	240	NA	294	NA	323

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard: Refers to Table 1: Full Depth Background Site Condition Standards - Soil - Residential/Parkland/Institutional/Industrial/Commercial/Community Property Use
Guideline values are for general reference only. The guidelines provided may or may not be relevant for the intended use. Refer directly to the applicable standard for regulatory interpretation.

178386 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.
*Sulphide analyzed at AGAT 5623 McAdam
Pl note: Redox Potential is not an accredited parameter.
Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.

178387-178388 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.
*Sulphide analyzed at AGAT 5623 McAdam
Pl note: Redox Potential is not an accredited parameter.
Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.
Elevated RDL indicates the degree of sample dilution prior to the analysis in order to keep analytes within the calibration range of the instrument and to reduce matrix interference.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By: Amanjot Bhela

CHARTERED AMANJOT BHELA CHEMIST — ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO

Results relate only to the items tested. Results apply to samples as received.

Guideline Violation

AGAT WORK ORDER: 19T464428

PROJECT: 1780055

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
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http://www.agatlabs.com

CLIENT NAME: GOLDER ASSOCIATES LTD. **ATTENTION TO: Mike Bentley**

SAMPLEID	SAMPLE TITLE	GUIDELINE	ANALYSIS PACKAGE	PARAMETER	UNIT	GUIDEVALUE	RESULT
178387	19-4 Sa4	ON T1 S RPI/ICC	Corrosivity Package	Electrical Conductivity (2:1)	mS/cm	0.57	1.20
178388	19-6 Sa4	ON T1 S RPI/ICC	Corrosivity Package	Electrical Conductivity (2:1)	mS/cm	0.57	2.70

AGAT Laboratories

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FAX (905)712-5122
http://www.agatlabs.com

Quality Assurance

CLIENT NAME: GOLDER ASSOCIATES LTD.

PROJECT: 1780055

SAMPLING SITE:

AGAT WORK ORDER: 19T464428

ATTENTION TO: Mike Bentley

SAMPLED BY:

Soil Analysis

RPT Date: May 16, 2019			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Corrosivity Package															
Sulfide (S2-)	178386	178386	< 0.05	< 0.05	NA	< 0.05	98%	80%	120%						
Chloride (2:1)	178386	178386	49	50	2.0%	< 2	93%	80%	120%	93%	80%	120%	97%	70%	130%
Sulphate (2:1)	178386	178386	5	5	NA	< 2	92%	80%	120%	96%	80%	120%	97%	70%	130%
pH (2:1)	178386	178386	8.87	8.92	0.6%	NA	100%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	178386	178386	0.178	0.182	2.2%	< 0.005	109%	90%	110%	NA			NA		
Redox Potential 1	1					NA	101%	70%	130%		70%	130%		70%	130%

Comments: NA signifies Not Applicable.
Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL

Certified By: Amanjot Bhela

CHARTERED AMANJOT BHELA CHEMIST
ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO

AGAT Laboratories

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MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
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Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

PROJECT: 1780055

SAMPLING SITE:

AGAT WORK ORDER: 19T464428

ATTENTION TO: Mike Bentley

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulfide (S2-)	MIN-200-12025	ASTM E1915-09	GRAVIMETRIC
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION
Redox Potential 1	INOR-93-6066	G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 2	INOR-93-6066	G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE
Redox Potential 3	INOR-93-6066	G200-09, SM 2580 B	REDOX POTENTIAL ELECTRODE

AGAT Laboratories

5835 Coopers Avenue
Mississauga, Ontario L4Z 1Y2
Ph: 905.712.5100 Fax: 905.712.5122
webearth.agatlabs.com

Laboratory Use Only

Work Order #: 19T464428

Cooler Quantity: Med

Arrival Temperatures: 3.5 | 3.3 | 3.1
2.4 | 2.7 | 2.5

Custody Seal Intact: ☐Yes ☐No ☐N/A

Notes:

Chain of Custody Record

If this is a Drinking Water sample, please use Drinking Water Chain of Custody Form (potable water consumed by humans)

Report Information:

Company: Golder Associates

Contact: Michael Bentley

Address: 100 Scotia Ct
Whitby ON L1N 8Y6

Phone: 905-723-2727 Fax: 905-723-2182

Reports to be sent to:
1. Email: mibentley@golder.com
2. Email: mo'oud_nasr@golder.com

Regulatory Requirements:

☐ No Regulatory Requirement

(Please check all applicable boxes)

☑ Regulation 153/04
Table 1 *Indicate One*
☐Ind/Com
☐Res/Park
☐Agriculture

Soil Texture *(Check One)*
☑Coarse
☐Fine

☐ Sewer Use
☐Sanitary
☐Storm
Region ____ *Indicate One*
☐MISA

☐ Regulation 558
☐ CCME
☐ Prov. Water Quality Objectives (PWQO)
☐ Other
Indicate One

Is this submission for a **Record of Site Condition**? ☐ Yes ☑ No

Report Guideline on Certificate of Analysis ☑ Yes ☐ No

Turnaround Time (TAT) Required:

Regular TAT ☑ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

☐ 3 Business Days ☐ 2 Business Days ☐ Next Business Day

OR Date Required (Rush Surcharges May Apply):

Please provide prior notification for rush TAT
**TAT is exclusive of weekends and statutory holidays*

For 'Same Day' analysis, please contact your AGAT CPM

Project Information:

Project: 1780055

Site Location:

Sampled By: M BENTLEY

AGAT Quote #: PO:

Please note: If quotation number is not provided, client will be billed full price for analysis.

Invoice Information:

Bill To Same: Yes ☑ No ☐

Company:
Contact:
Address:
Email:

Sample Matrix Legend

B	Biota
GW	Ground Water
O	Oil
P	Paint
S	Soil
SD	Sediment
SW	Surface Water

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI (Y/N)	Metals and Inorganics	☐ All Metals ☐ 153 Metals (excl. Hydrides)	☐ Hydride Metals ☐ 153 Metals (incl. Hydrides)	ORPs: ☐ B-HWS ☐ Cl ☐ CN	☐ Cr6+ ☐ EC ☐ FOC ☐ Hg	☐ pH ☐ SAR	Full Metals Scan	Regulation/Custom Metals	Nutrients: ☐ TP ☐ NH3 ☐ TKN ☐ NO3 ☐ NO2 ☐ NO3+NO2	Volatiles: ☐ VOC ☐ BTEX ☐ THM	PHCs F1 - F4	ABNs	PAHs	PCBs: ☐ Total ☐ Aroclors	Organochlorine Pesticides	TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	Sewer Use	Corrosivity
19-3 Sa 6	April 23/19	2:15PM	1	Soil																				X
19-4 Sa 4	April 25/19	7:15AM	1	Soil																				X
19-6 Sa 4	April 22/19	2:30PM	1	Soil																				X

(O. Reg 153 spans the metals/inorganics columns.)

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
M BENTLEY	May 6/19	3:00PM		May 7/19	11:33
	May 7/19	3:07			

Page 1 of 1

Nº: T087931

APPENDIX D

Results of Cone Penetration Testing

PRESENTATION OF SITE INVESTIGATION RESULTS

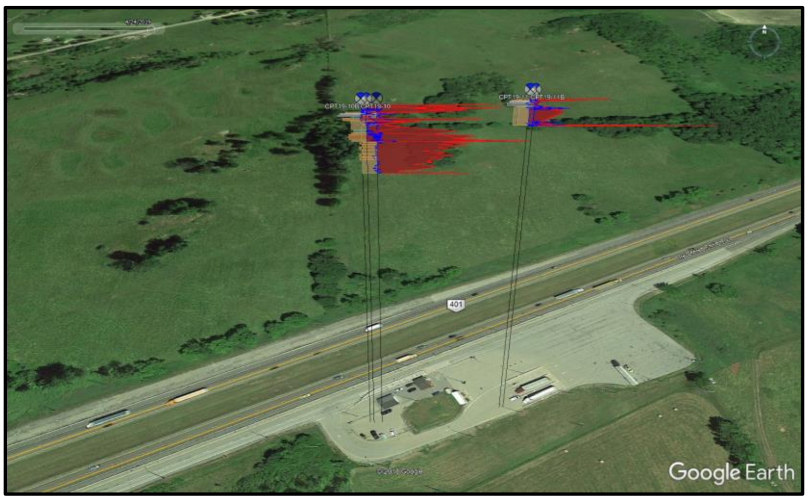
Gananoque Truck Inspection Station

Prepared for:

Golder Associates

ConeTec Job No: 19-05025

Project Start Date: 25-Apr-2019
Project End Date: 25-Apr-2019
Report Date: 02-May-2019



Prepared by:

ConeTec Investigations Ltd.
9033 Leslie Street, Unit 15
Richmond Hill, ON L4B 4K3

Tel: (905) 886-2663
Fax: (905) 886-2664
Toll Free: (800) 504-1116

Email: conetecon@conetec.com
www.conetec.com
www.conetecdataservices.com

CONETEC

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates at Gananoque Truck Inspection Station, Ontario. The program consisted of five cone penetration tests (CPT).

Project Information

Project	
Client	Golder Associates
Project	Gananoque Truck Inspection Station
ConeTec project number	19-05025

An image from Google Earth including the CPT test locations is presented below.

Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	CPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT	Consumer-grade GPS	32618

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Advanced CPT plots with I_c, $Su(N_{kt})$, Phi and $N_{1(60)}$(IcRW1998) as well as SBT Scatter plots are provided in the release package.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
549:T1500F15U500	549	15	225	1500	15	500
Cone AD549 was used for all the soundings.						

CPT Calculated Parameters	
Additional information	The Normalized Soil Behavior Type Chart based on Q_{tn} (SBT Qtn) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s), and pore pressure (u_2). Hydrostatic conditions were assumed for the calculated parameters. Effective stresses are calculated based on unit weights that have been assigned to the individual soil behavior type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behavior Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Limitations

This report has been prepared for the exclusive use of Golder Associates (Client) for the project titled "Gananoque Truck Inspection Station". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in 5 cm^2, 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

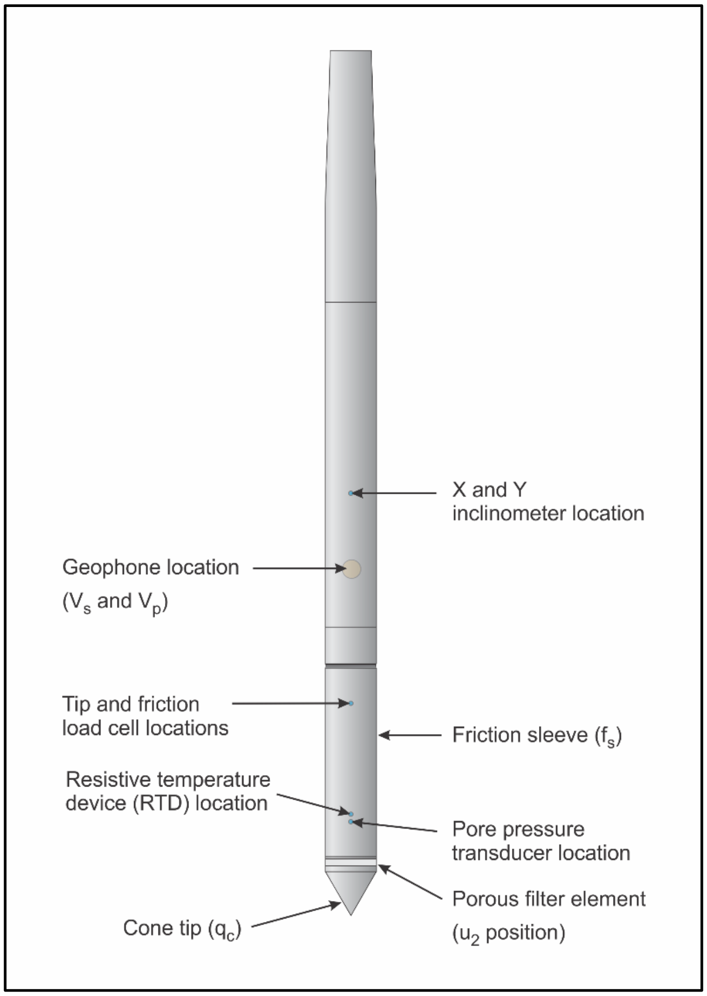


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 cm; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 mm are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behaviour type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

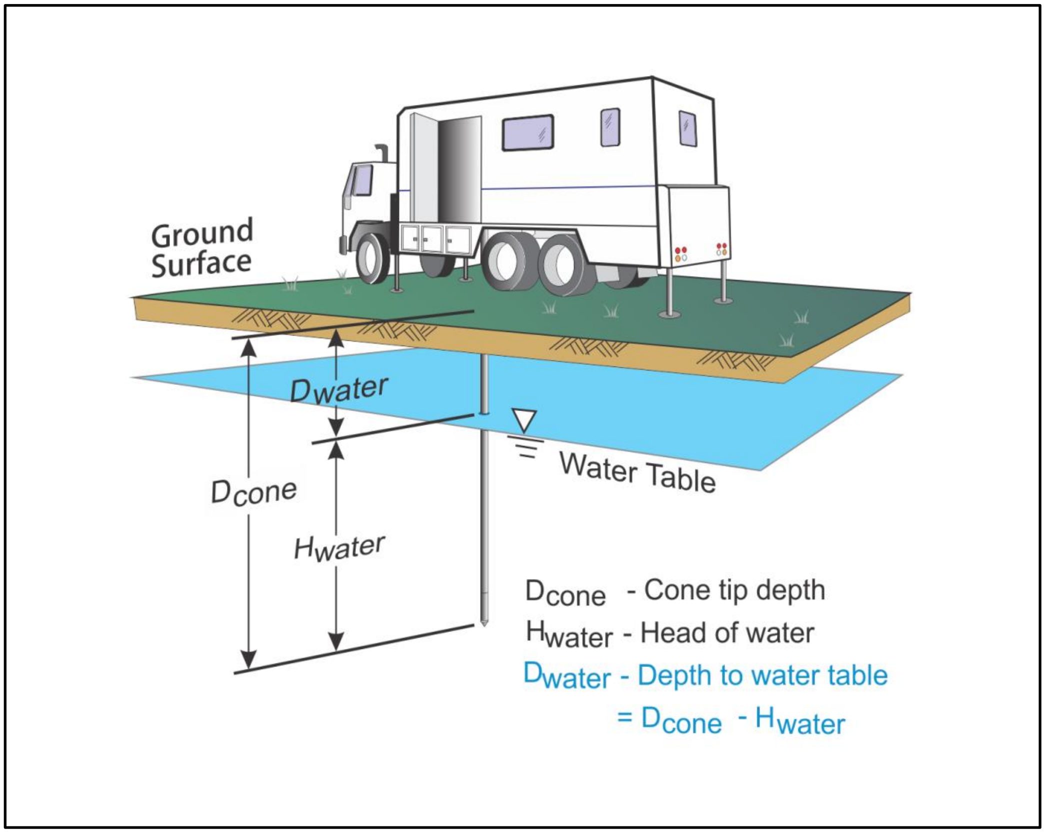


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

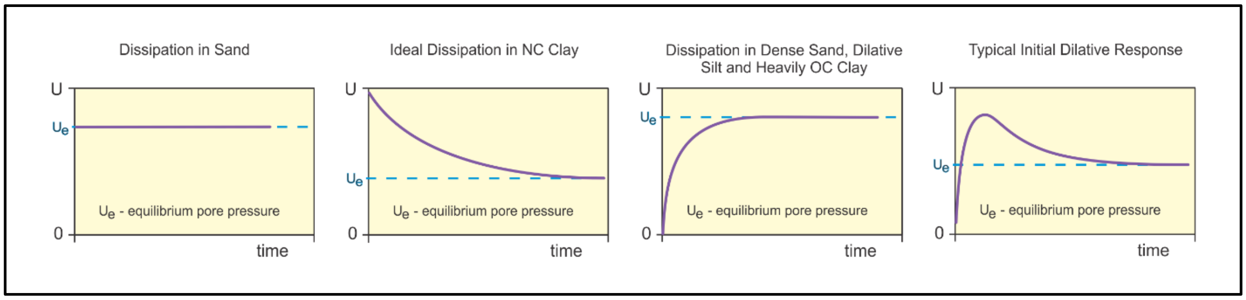


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

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Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158.

Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355.

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Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with I_c, $Su(N_{kt})$, Phi and $N_{1(60)}$ (IcRW1998)
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and
Standard Cone Penetration Test Plots

CONETEC

Job No: 19-05025
Client: Golder Associates
Project: Gananoque Truck Inspection Station
Start Date: 25-Apr-2019
End Date: 25-Apr-2019

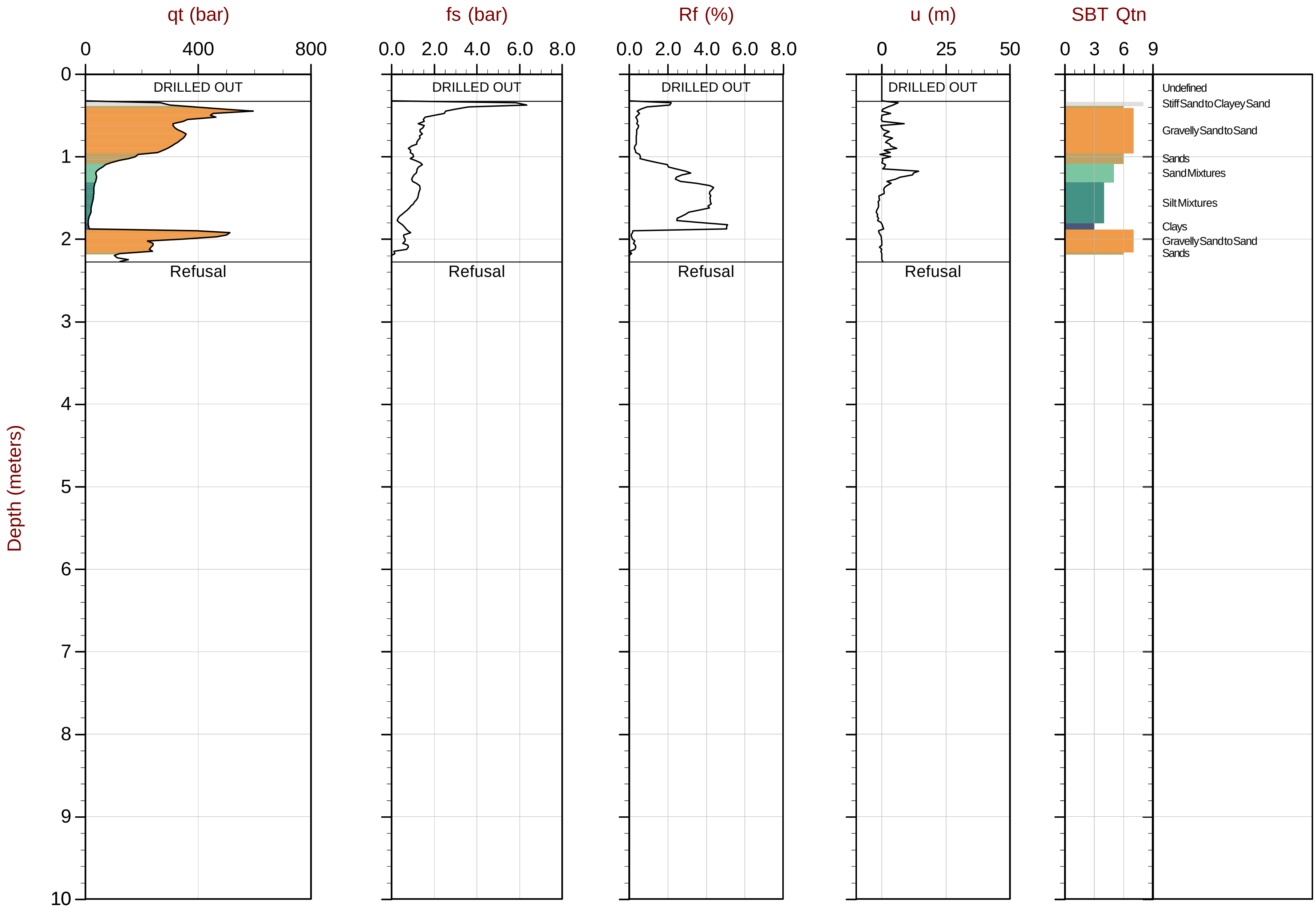
CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting[2] (m)	Refer to Notation Number
CPT19-10	19-05025_CP10	25-Apr-2019	549:T1500F15U500		2.275	4912461	413861	
CPT19-10B	19-05025_CP10B	25-Apr-2019	549:T1500F15U500	3.9	5.025	4912461	413863	3
CPT19-10C	19-05025_CP10C	25-Apr-2019	549:T1500F15U500	3.9	9.600	4912461	413867	
CPT19-11	19-05025_CP11	25-Apr-2019	549:T1500F15U500	3.4	4.150	4912471	413930	
CPT19-11B	19-05025_CP11B	25-Apr-2019	549:T1500F15U500	3.1	4.250	4912471	413928	

1. The assumed phreatic surface was based on pore pressure dissipations tests, unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected using a consumer grade GPS device in datum WGS84 / UTM Zone 18 North.
3. Phreatic surface based on CPT19-10C.

Golder Associates

Job No: 19-05025
Date: 2019-04-25 09:33
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-10
Cone: 549:T1500F15U500



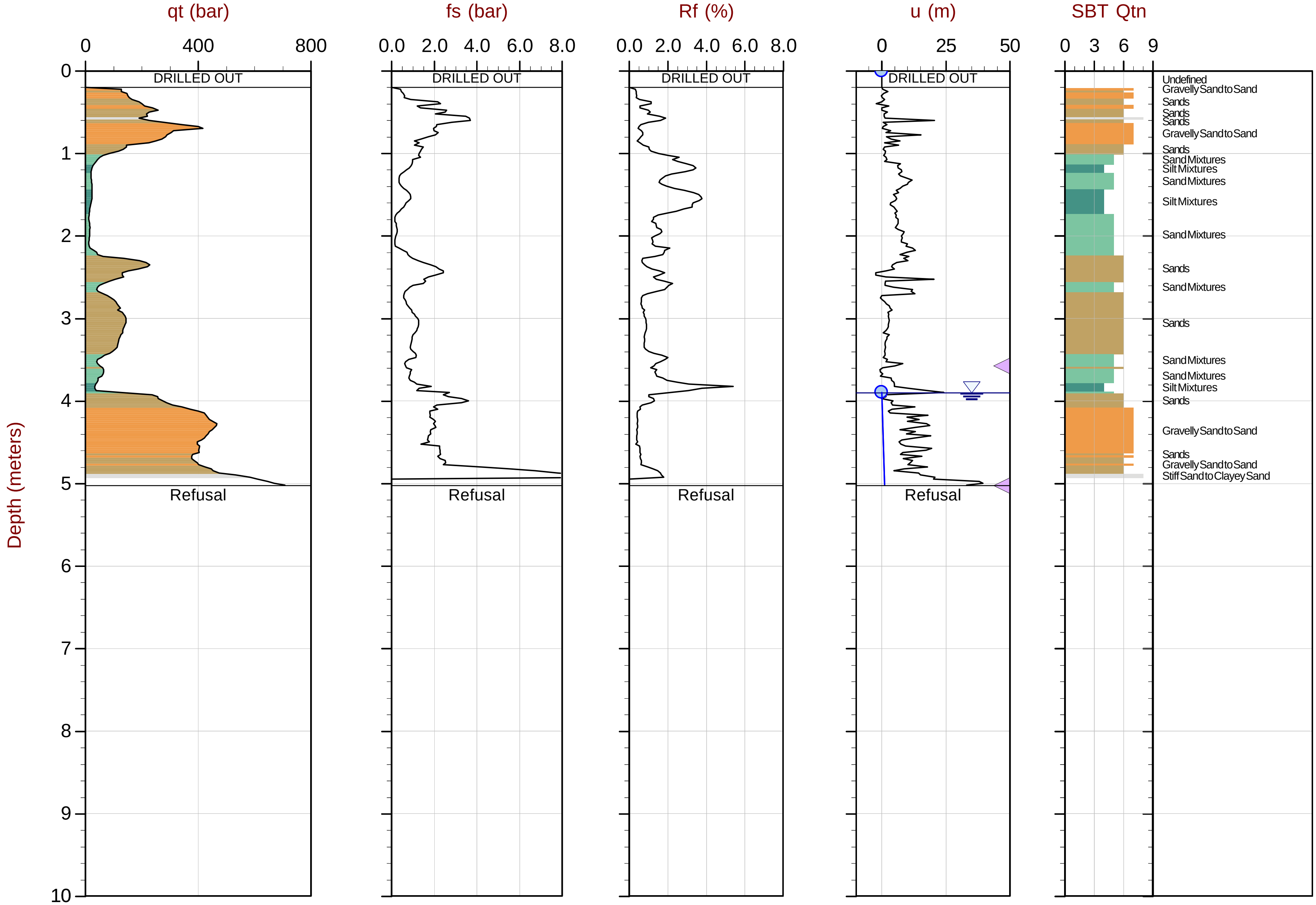
Max Depth: 2.275 m / 7.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line

File: 19-05025_CP10.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912461m E: 413861m
Page No: 1 of 1

Golder Associates

Job No: 19-05025
Date: 2019-04-25 09:46
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-10B
Cone: 549:T1500F15U500



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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-05025_CP10B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912461m E: 413863m
Page No: 1 of 1

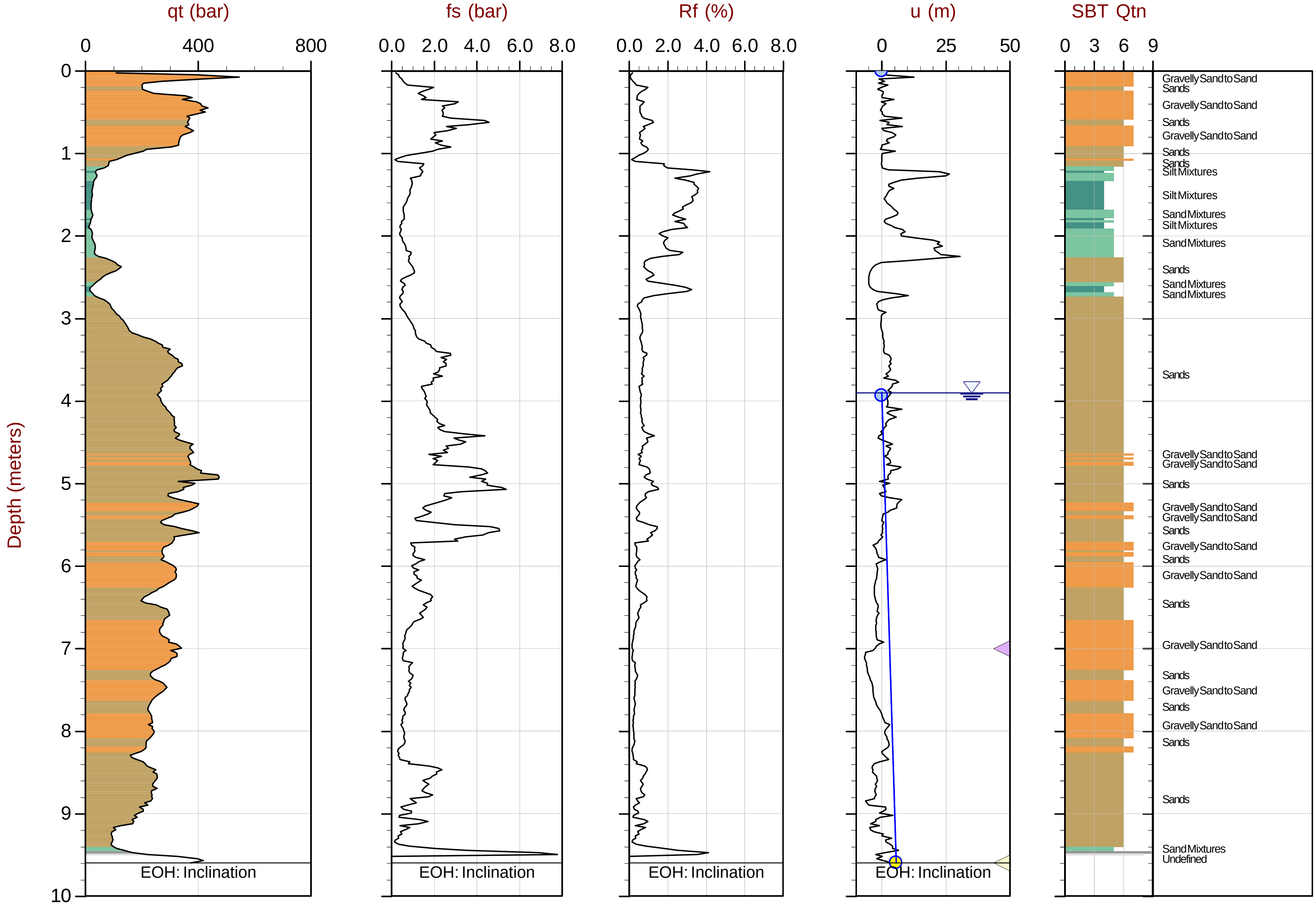
Overplot Item: Assumed Ueq | Ueq | Dissipation, equilibrium achieved | Dissipation, equilibrium not achieved | Hydrostatic Line

CONETEC

Golder Associates

Job No: 19-05025
Date: 2019-04-25 10:34
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-10C
Cone: 549:T1500F15U500



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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-05025_CP10C.COR
Unit Wt: SBTQtn (PKR2009)

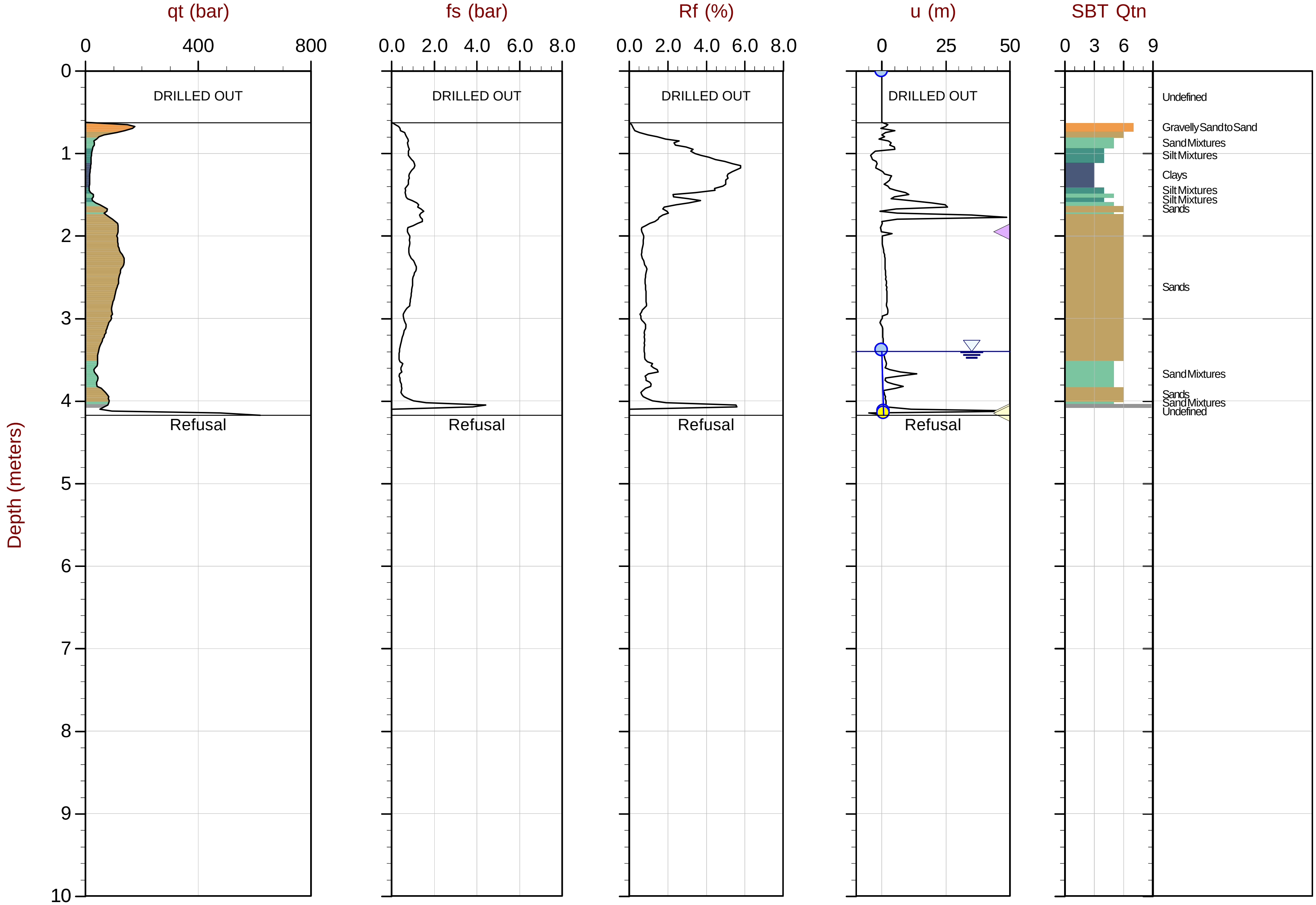
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912461m E: 413867m
Page No: 1 of 1

Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

Golder Associates

Job No: 19-05025
Date: 2019-04-25 07:36
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-11
Cone: 549:T1500F15U500



Max Depth: 4.175 m / 13.70 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-05025_CP11.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912471m E: 413930m
Page No: 1 of 1

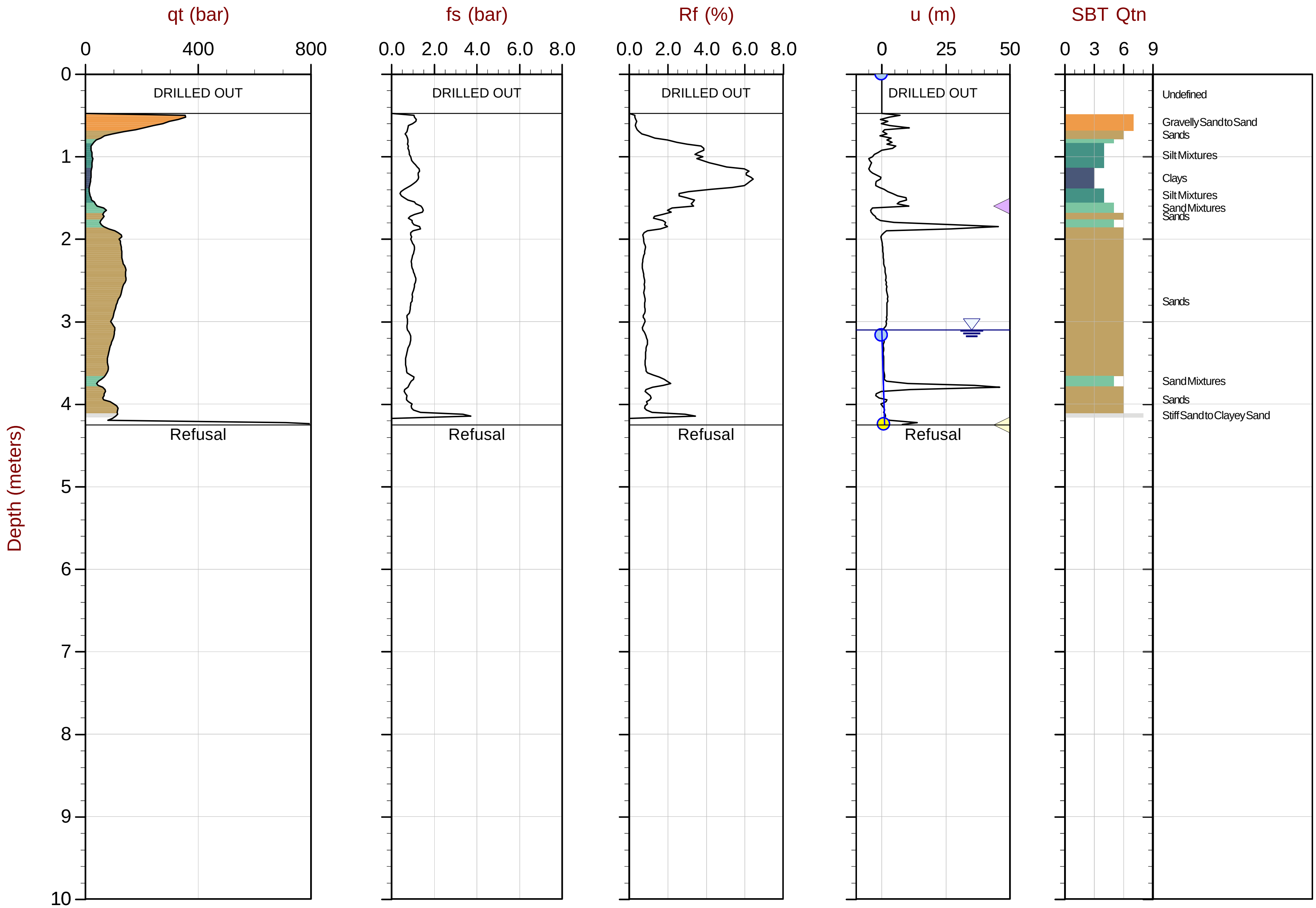
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line

CONETEC

Golder Associates

Job No: 19-05025
Date: 2019-04-25 08:33
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-11B
Cone: 549:T1500F15U500



Max Depth: 4.250 m / 13.94 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-05025_CP11B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912471m E: 413928m
Page No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line

Advanced Cone Penetration Test Plots with I_c, Su(N_{kt}), Phi and $N_{1(60)}$(Ic RW1998)

Golder Associates

Job No: 19-05025
Date: 2019-04-25 09:33
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-10
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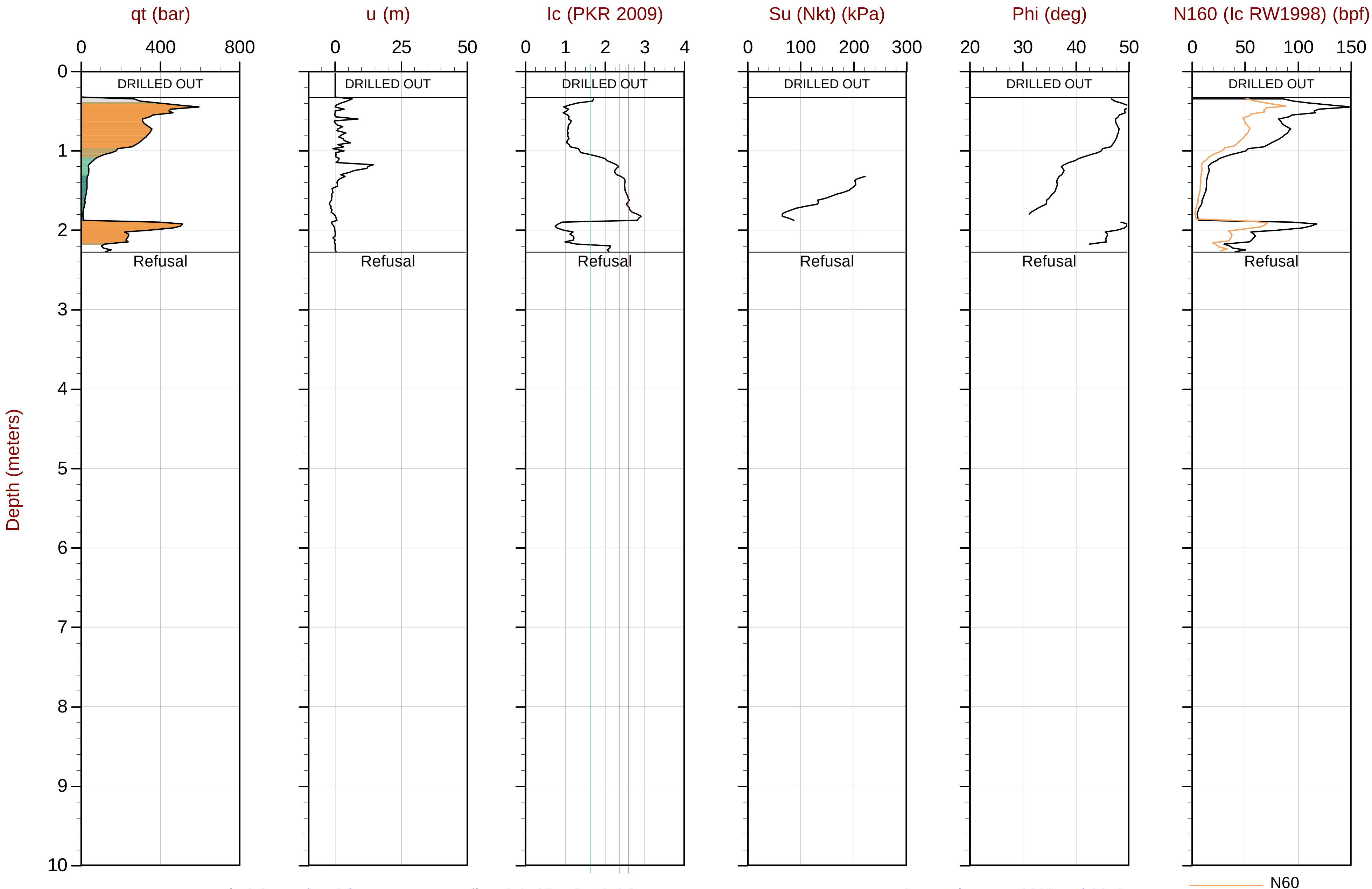

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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-05025_CP10.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912461m E: 413861m
Page No: 1 of 1

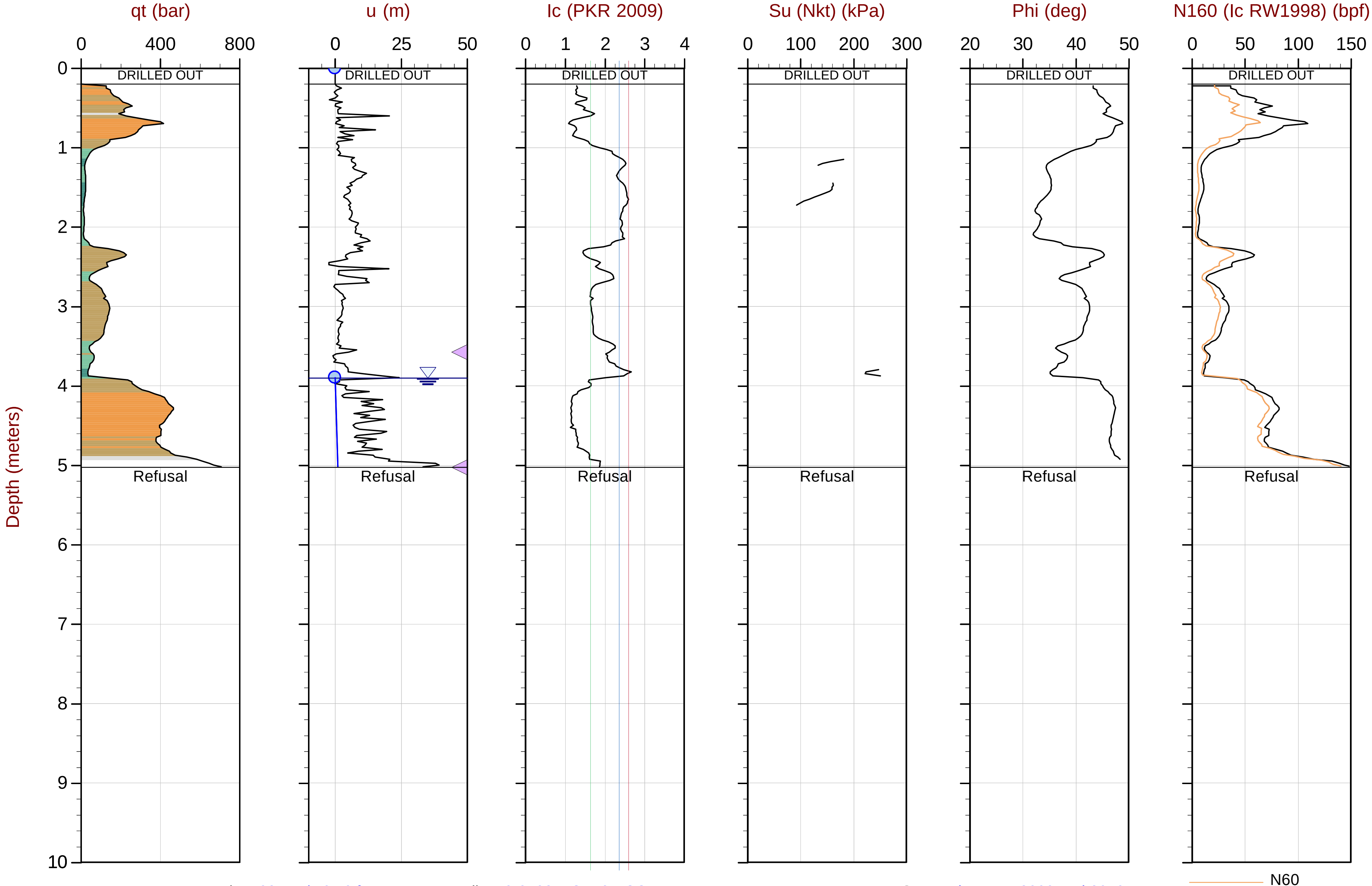
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line

CONETEC

Golder Associates

Job No: 19-05025
Date: 2019-04-25 09:46
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-10B
Cone: 549:T1500F15U500



Max Depth: 5.025 m / 16.49 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item: Assumed Ueq, Ueq

File: 19-05025_CP10B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved

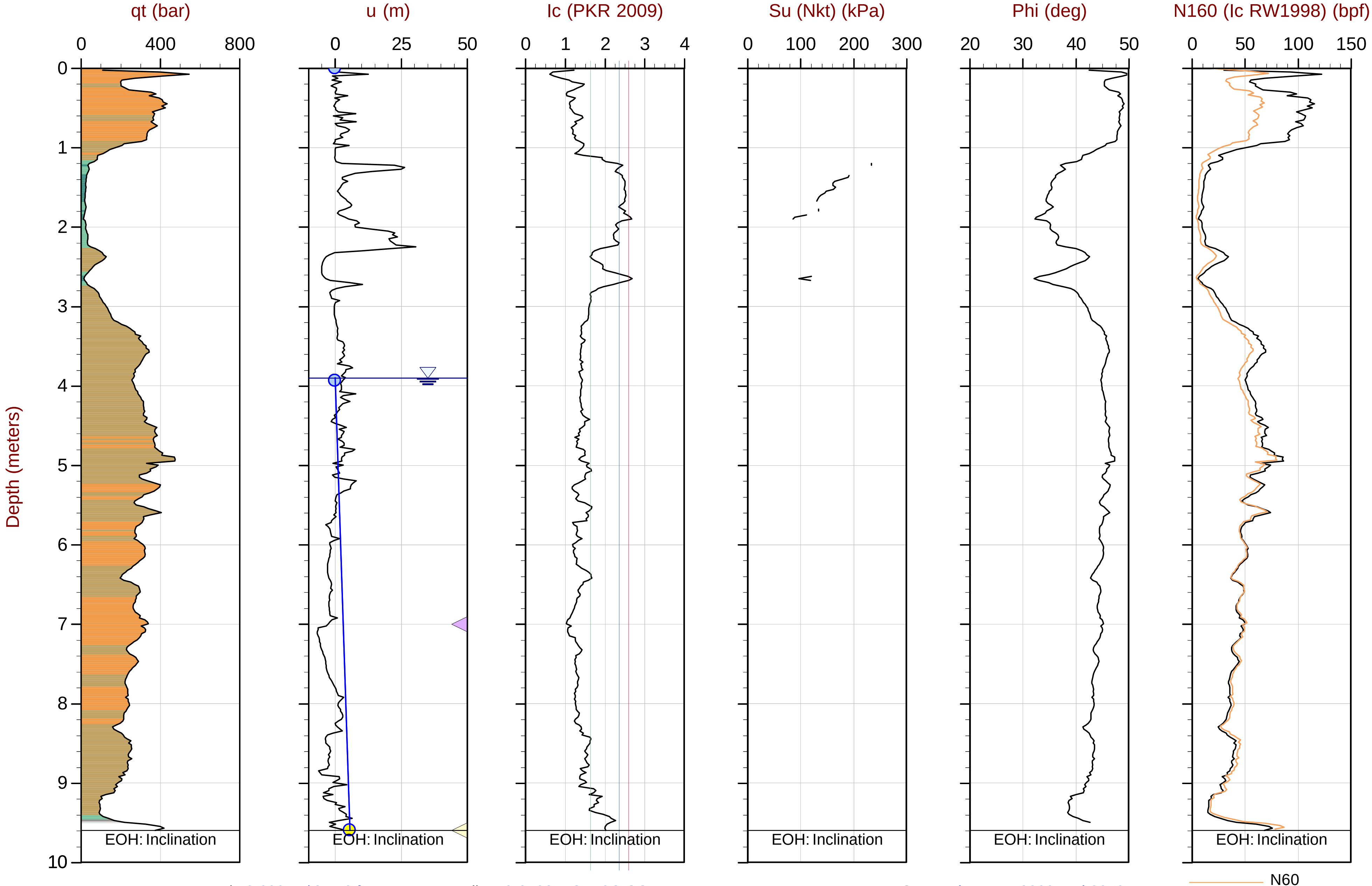
Hydrostatic Line

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912461m E: 413863m
Page No: 1 of 1

CONETEC

Golder Associates

Job No: 19-05025
Date: 2019-04-25 10:34
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-10C
Cone: 549:T1500F15U500



Max Depth: 9.600 m / 31.50 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item:

File: 19-05025_CP10C.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912461m E: 413867m
Page No: 1 of 1

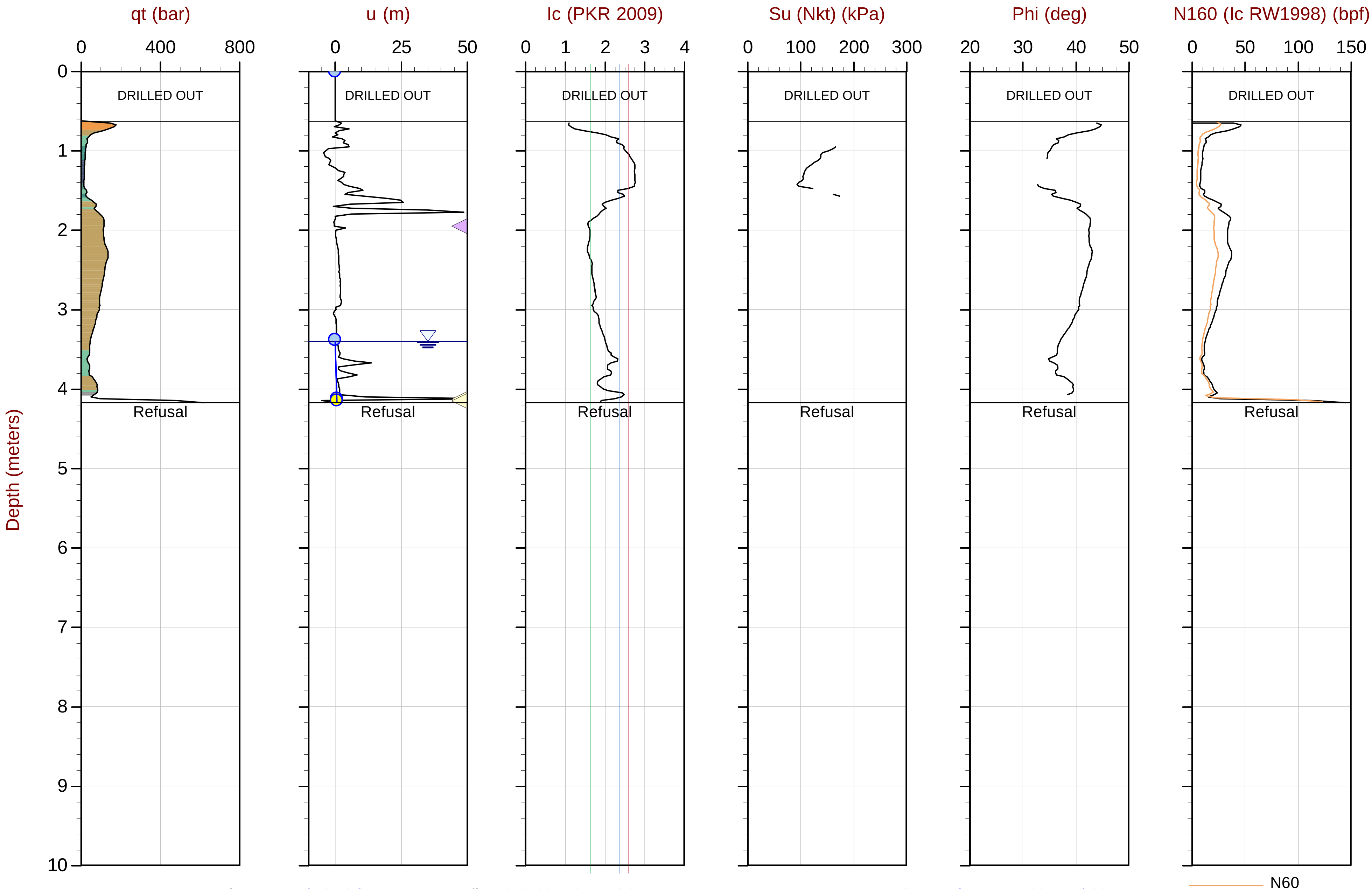
Assumed Ueq
Ueq
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved
Hydrostatic Line

ConeTec

Golder Associates

Job No: 19-05025
Date: 2019-04-25 07:36
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-11
Cone: 549:T1500F15U500



Max Depth: 4.175 m / 13.70 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-05025_CP11.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912471m E: 413930m
Page No: 1 of 1

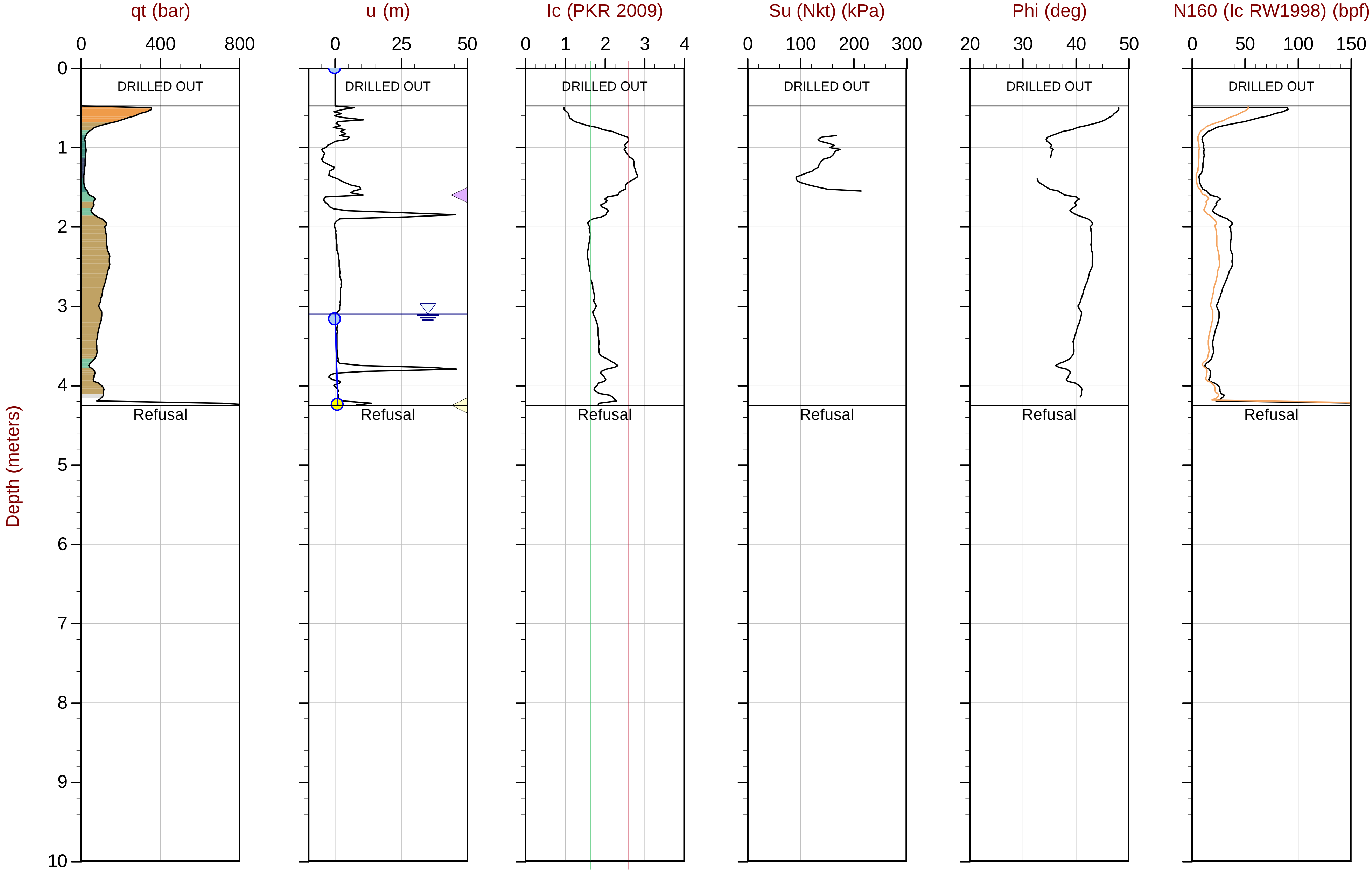
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line

CONETEC

Golder Associates

Job No: 19-05025
Date: 2019-04-25 08:33
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-11B
Cone: 549:T1500F15U500



N60

Max Depth: 4.250 m / 13.94 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item: Assumed Ueq, Ueq

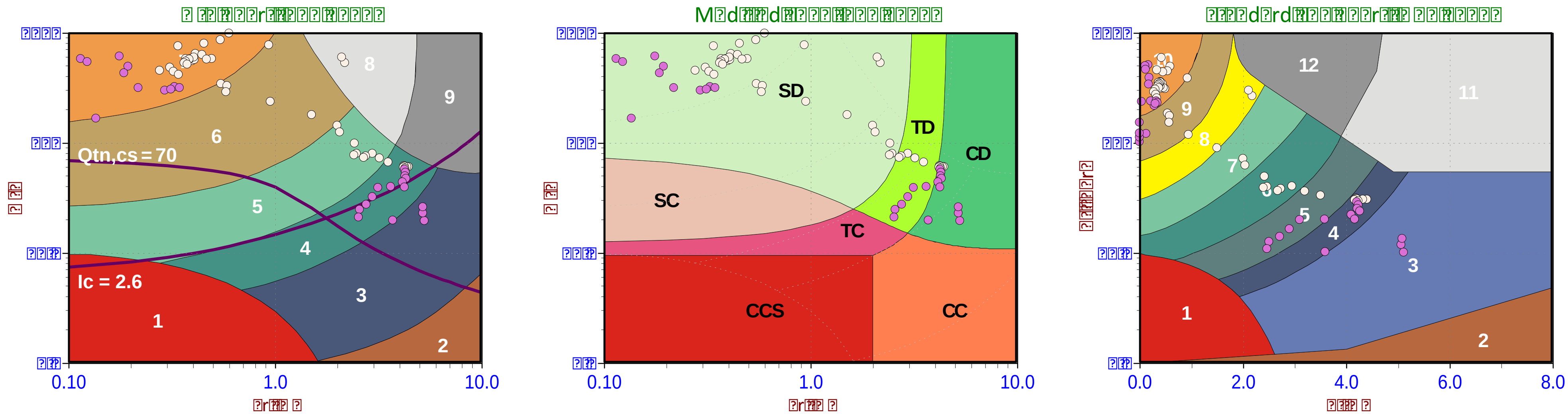
File: 19-05025_CP11B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0
Dissipation, equilibrium achieved
Dissipation, equilibrium not achieved

Hydrostatic Line

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4912471m E: 413928m
Page No: 1 of 1

Soil Behaviour Type (SBT) Scatter Plots

CONETEC



Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

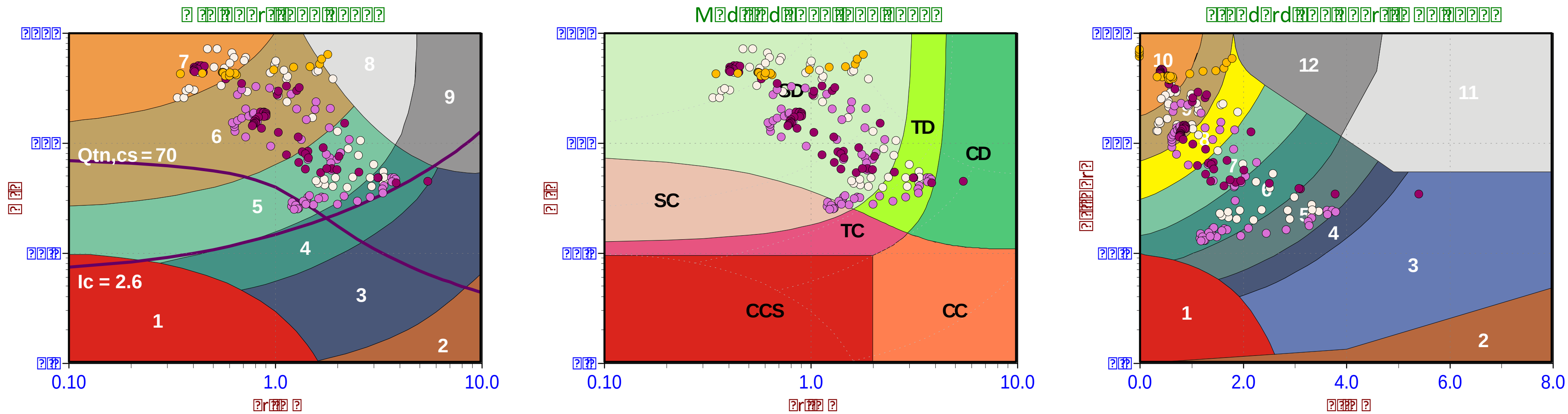
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

Golder Associates

Job No: 19-05025
Date: 2019-04-25 09:46
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-10B
Cone: 549:T1500F15U500



Depth Ranges
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- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

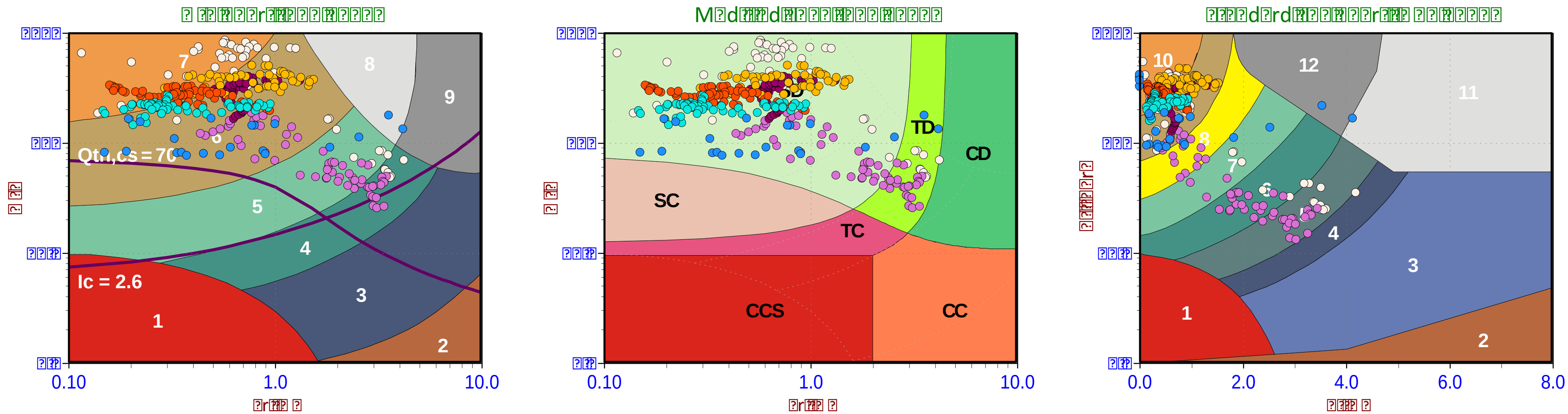
Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Golder Associates

Job No: 19-05025
Date: 2019-04-25 10:34
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-10C
Cone: 549:T1500F15U500



Depth Ranges
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- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

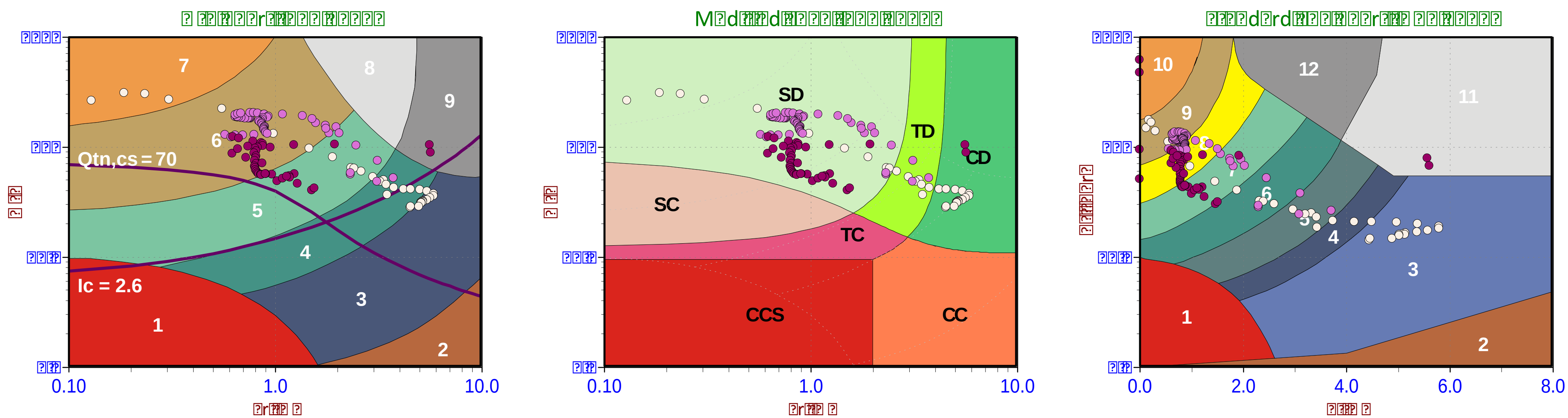
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

Golder Associates

Job No: 19-05025
Date: 2019-04-25 07:36
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-11
Cone: 549:T1500F15U500



Depth Ranges
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- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

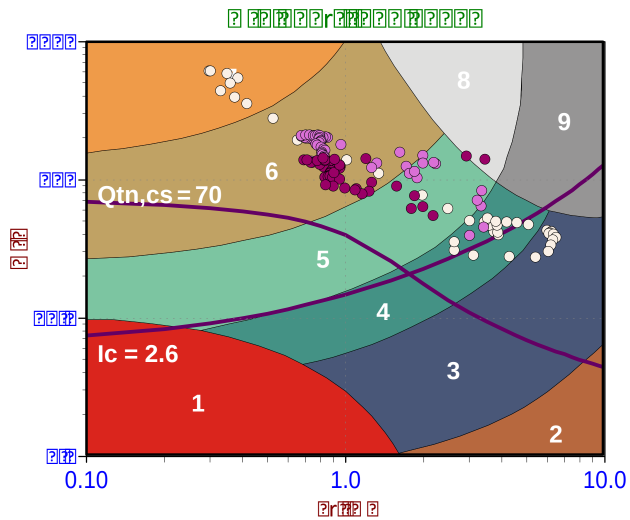
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

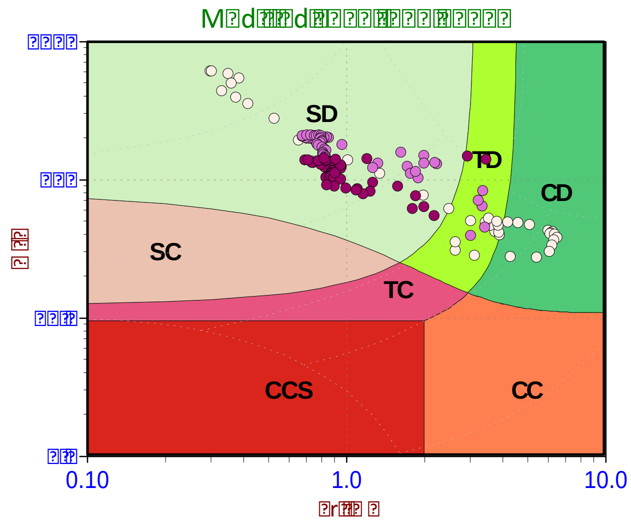
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

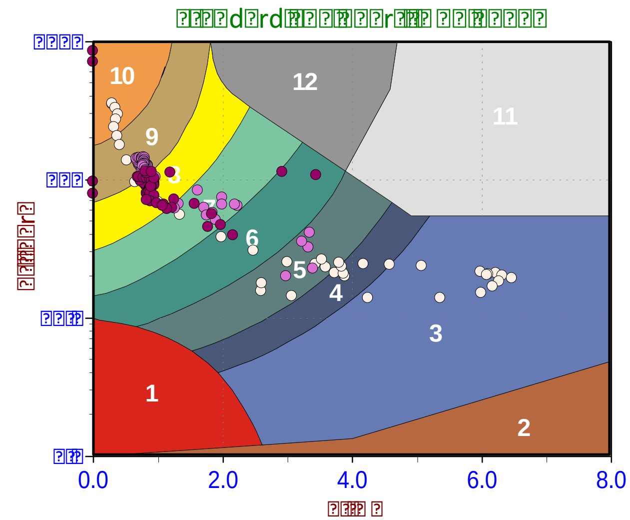
ConeTec

Golder Associates

Job No: 19-05025
Date: 2019-04-25 08:33
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-11B
Cone: 549:T1500F15U500







Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

Job No:	19-05025
Client:	Golder Associates
Project:	Gananoque Truck Inspection Station
Start Date:	25-Apr-2019
End Date:	25-Apr-2019

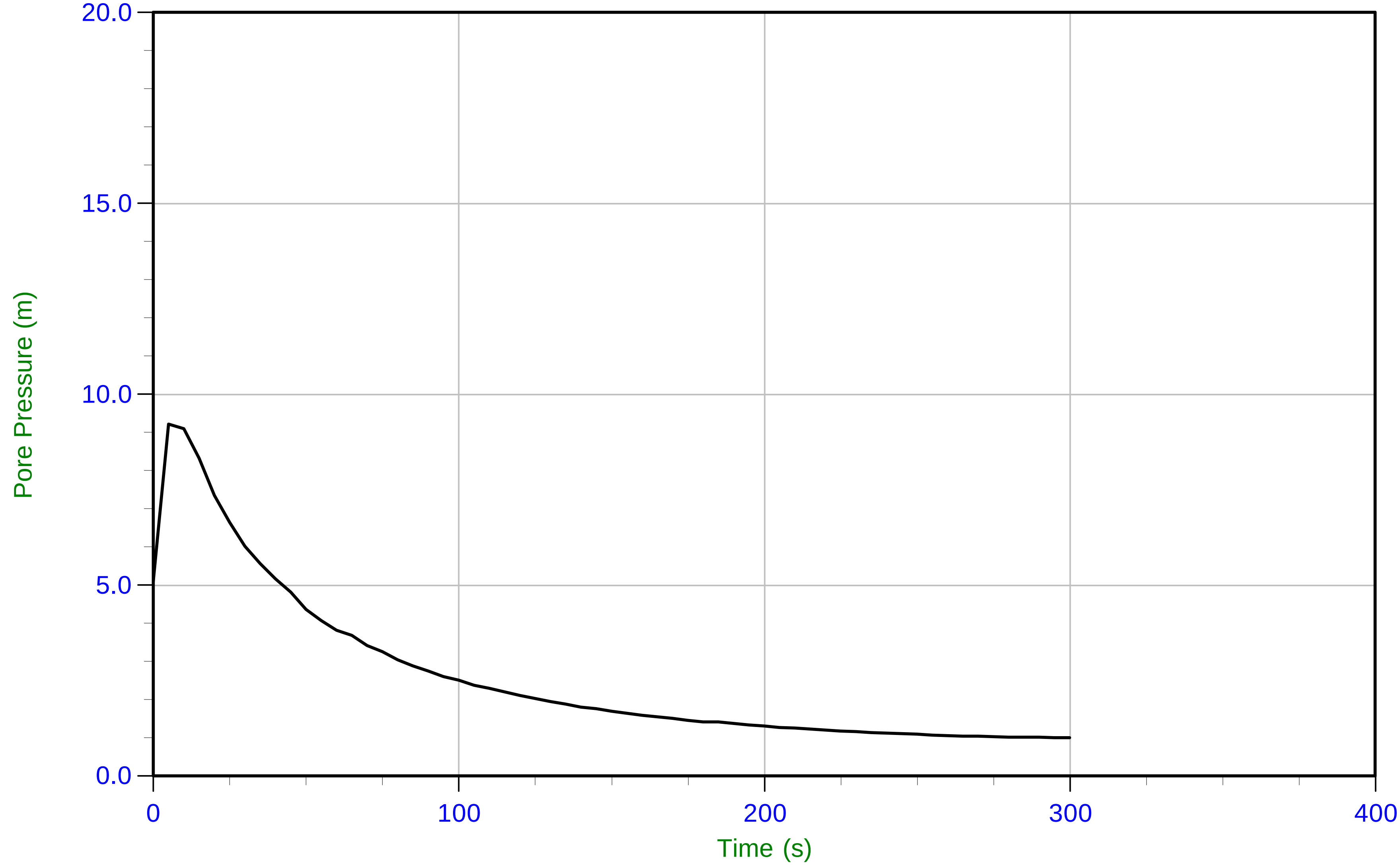
CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)
CPT19-10B	19-05025_CP10B	15	300	3.575	Not Achieved	
CPT19-10B	19-05025_CP10B	15	520	5.025	Not Achieved	
CPT19-10C	19-05025_CP10C	15	300	7.000	Not Achieved	
CPT19-10C	19-05025_CP10C	15	300	9.600	5.7	3.9
CPT19-11	19-05025_CP11	15	635	1.950	Not Achieved	
CPT19-11	19-05025_CP11	15	300	4.125	0.7	3.4
CPT19-11	19-05025_CP11	15	470	4.150	0.8	3.4
CPT19-11B	19-05025_CP11B	15	300	1.600	Not Achieved	
CPT19-11B	19-05025_CP11B	15	300	4.250	1.1	3.2

ConeTec

Golder Associates

Job No: 19-05025
Date: 04/25/2019 09:46
Site: Gananoque Truck Inspection 401 Hwy Eastbound
Sounding: CPT19-10B
Cone: 549:T1500F15U500 Area=15 cm²



Trace Summary:

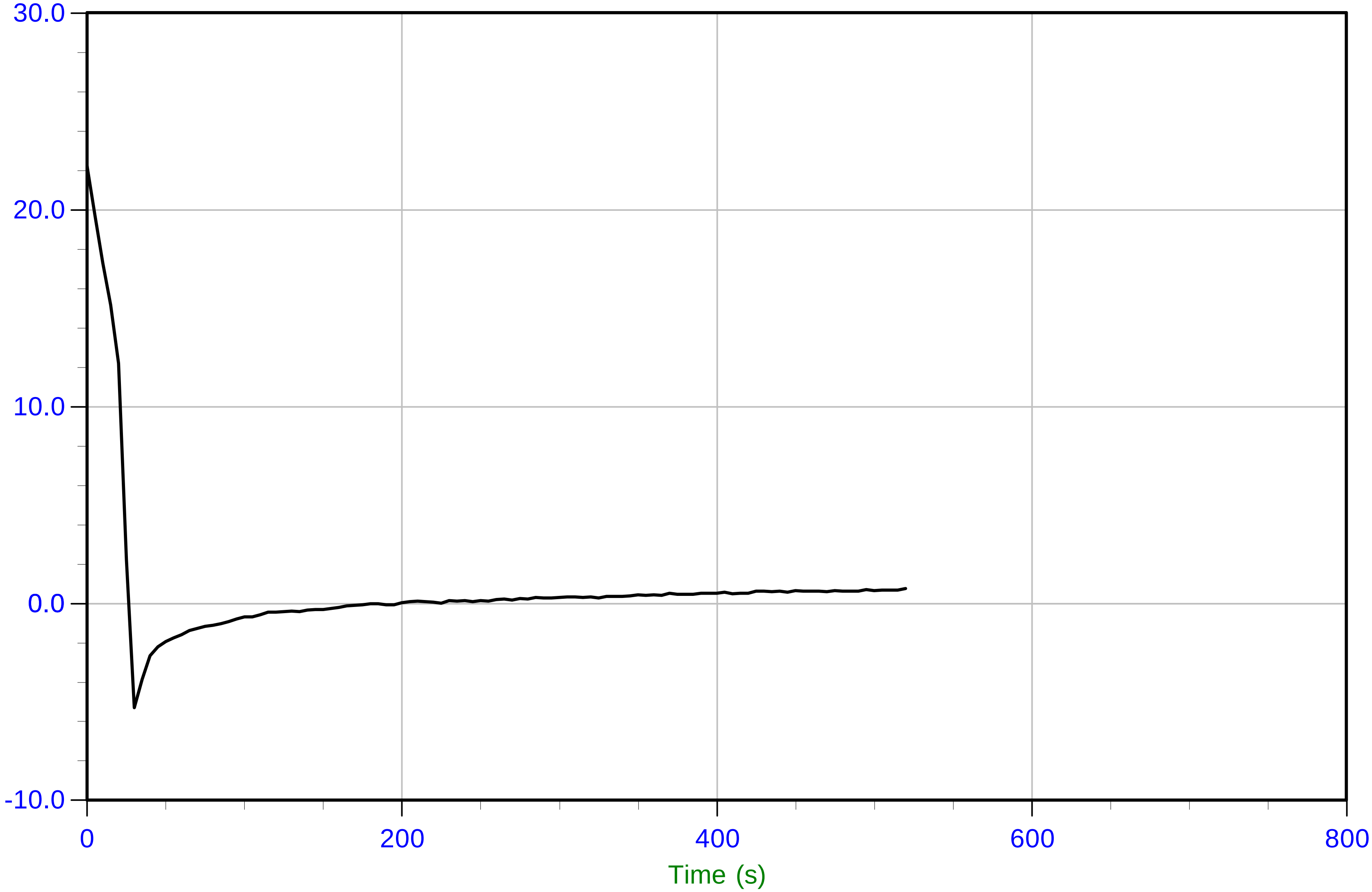
Filename: 19-05025_CP10B.PPF
Depth: 3.575 m / 11.729 ft
Duration: 300.0 s

u Min: 1.0 m
u Max: 9.2 m
u Final: 1.0 m

Golder Associates

Job No: 19-05025
Date: 04/25/2019 09:46
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-10B
Cone: 549:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05025_CP10B.PPF
Depth: 5.025 m / 16.486 ft
Duration: 520.0 s

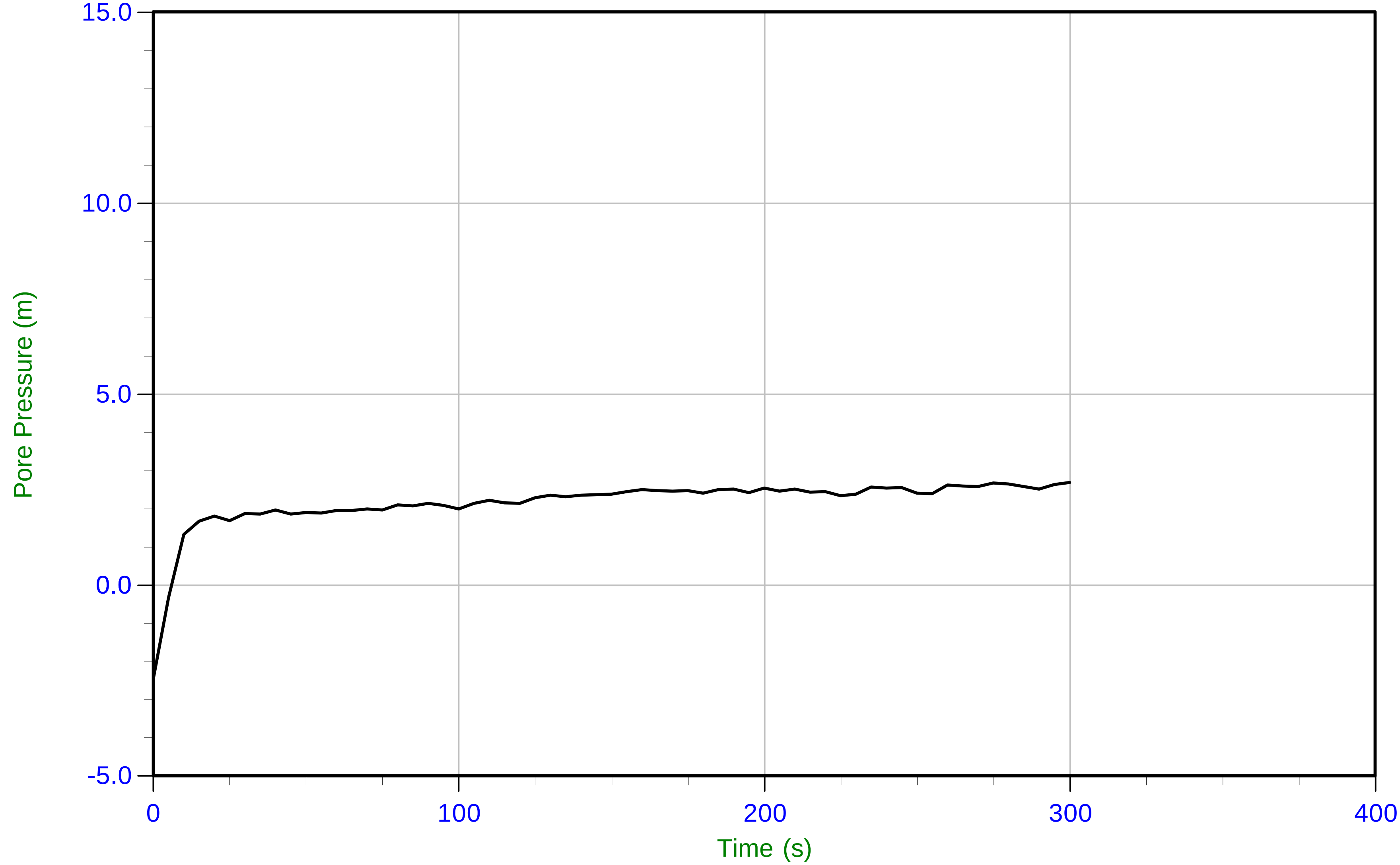
u Min: -5.3 m
u Max: 22.2 m
u Final: 0.7 m

CONETEC

Golder Associates

Job No: 19-05025
Date: 04/25/2019 10:34
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-10C
Cone: 549:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05025_CP10C.PPF
Depth: 7.000 m / 22.966 ft
Duration: 300.0 s

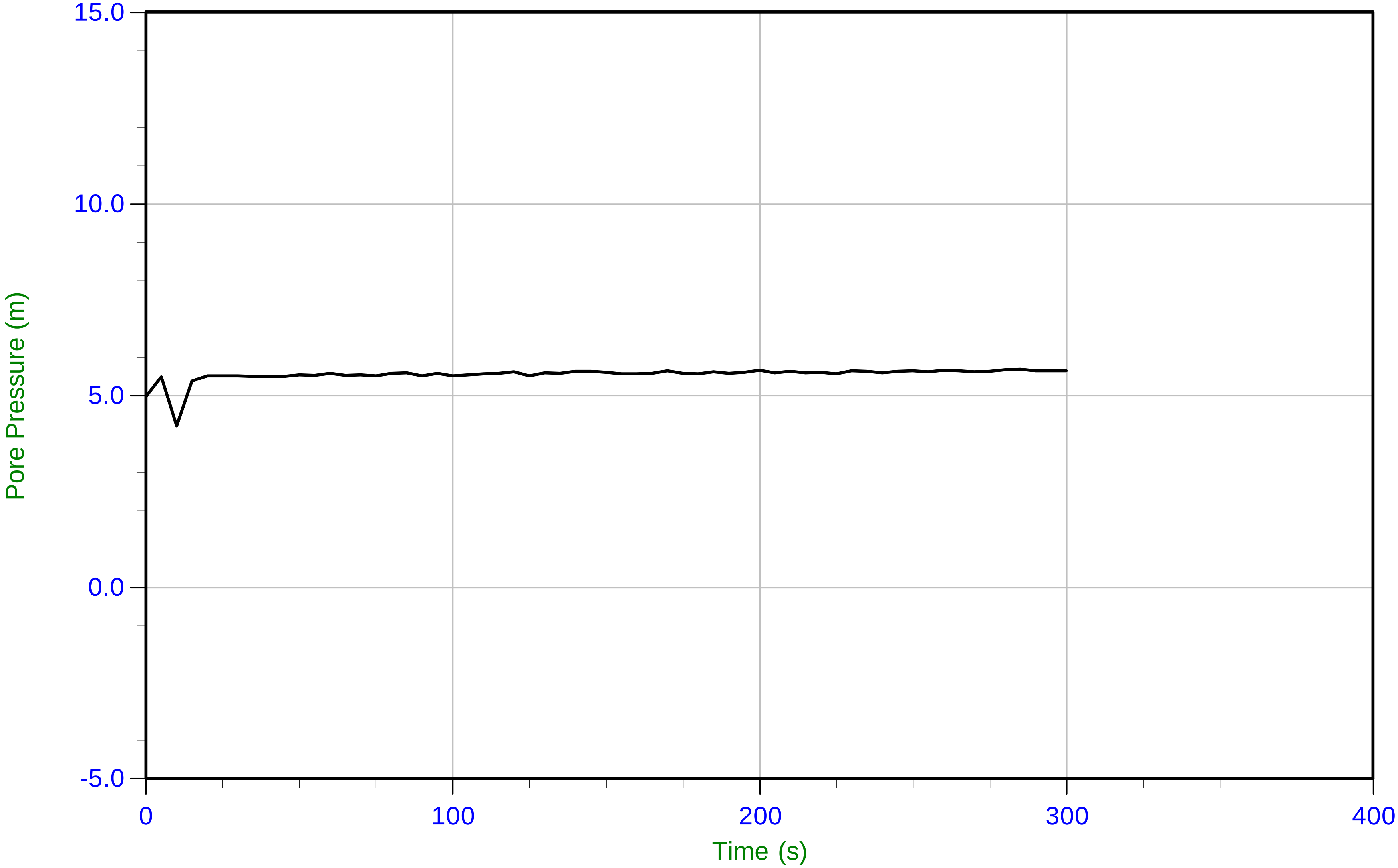
u Min: -2.5 m
u Max: 2.7 m
u Final: 2.7 m

ConeTec

Golder Associates

Job No: 19-05025
Date: 04/25/2019 10:34
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-10C
Cone: 549:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05025_CP10C.PPF
Depth: 9.600 m / 31.496 ft
Duration: 300.0 s

u Min: 4.2 m
u Max: 5.7 m
u Final: 5.6 m

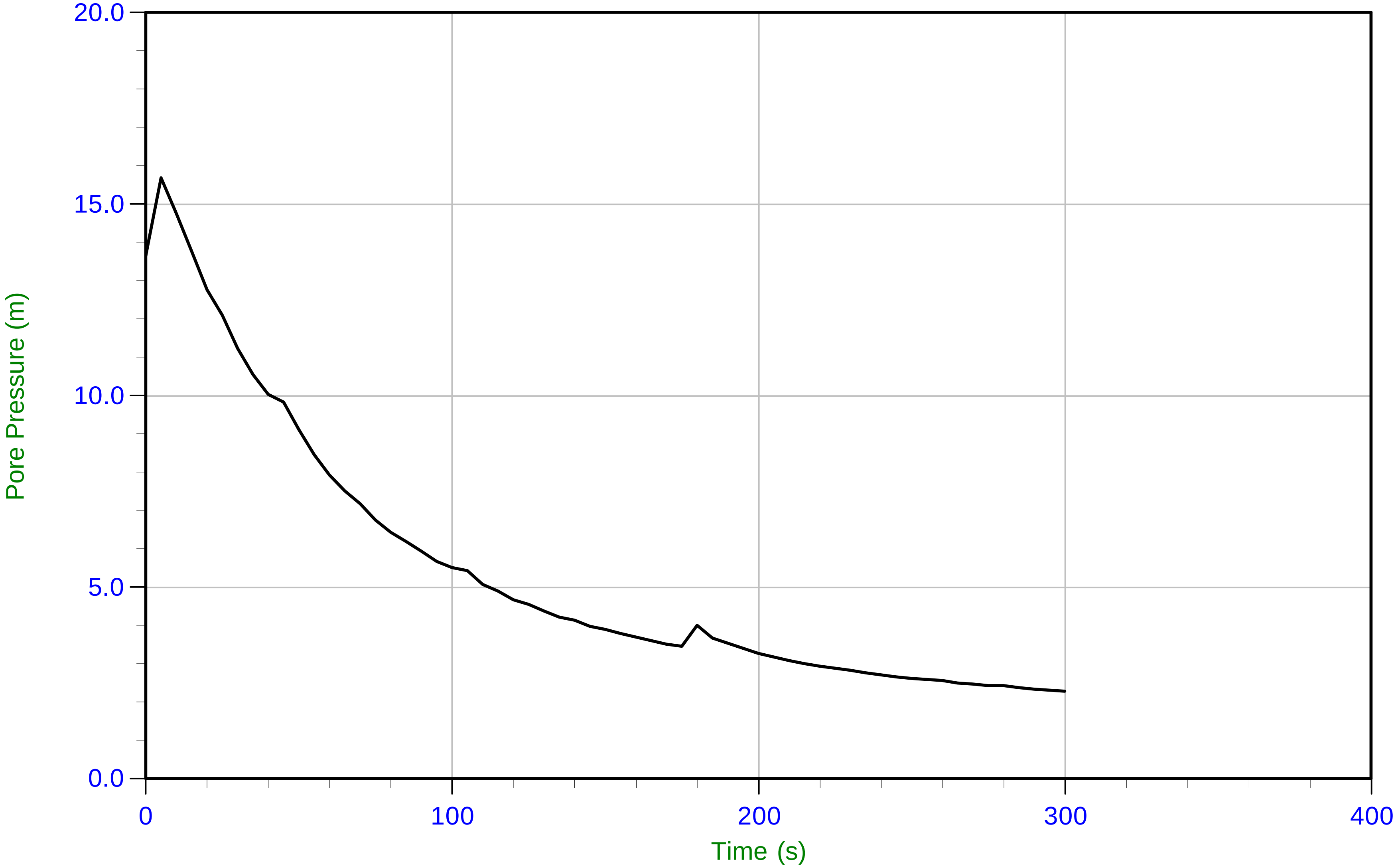
WT: 3.936 m / 12.914 ft
Ueq: 5.7 m

CONETEC

Golder Associates

Job No: 19-05025
Date: 04/25/2019 08:33
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-11B
Cone: 549:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05025_CP11B.PPF
Depth: 1.600 m / 5.249 ft
Duration: 300.0 s

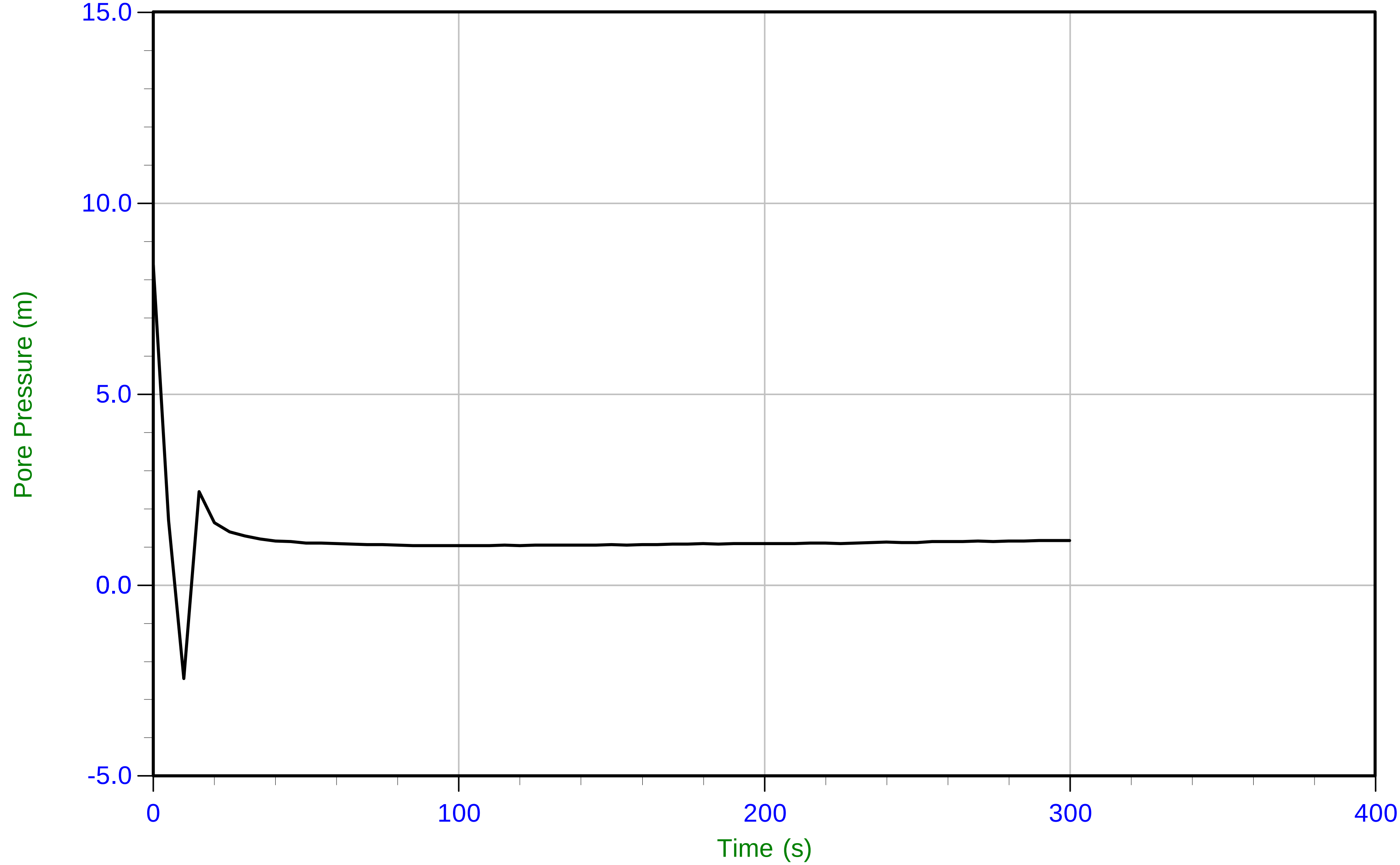
u Min: 2.3 m
u Max: 15.7 m
u Final: 2.3 m

CONETEC

Golder Associates

Job No: 19-05025
Date: 04/25/2019 08:33
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-11B
Cone: 549:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05025_CP11B.PPF
Depth: 4.250 m / 13.943 ft
Duration: 300.0 s

u Min: -2.4 m
u Max: 8.4 m
u Final: 1.2 m

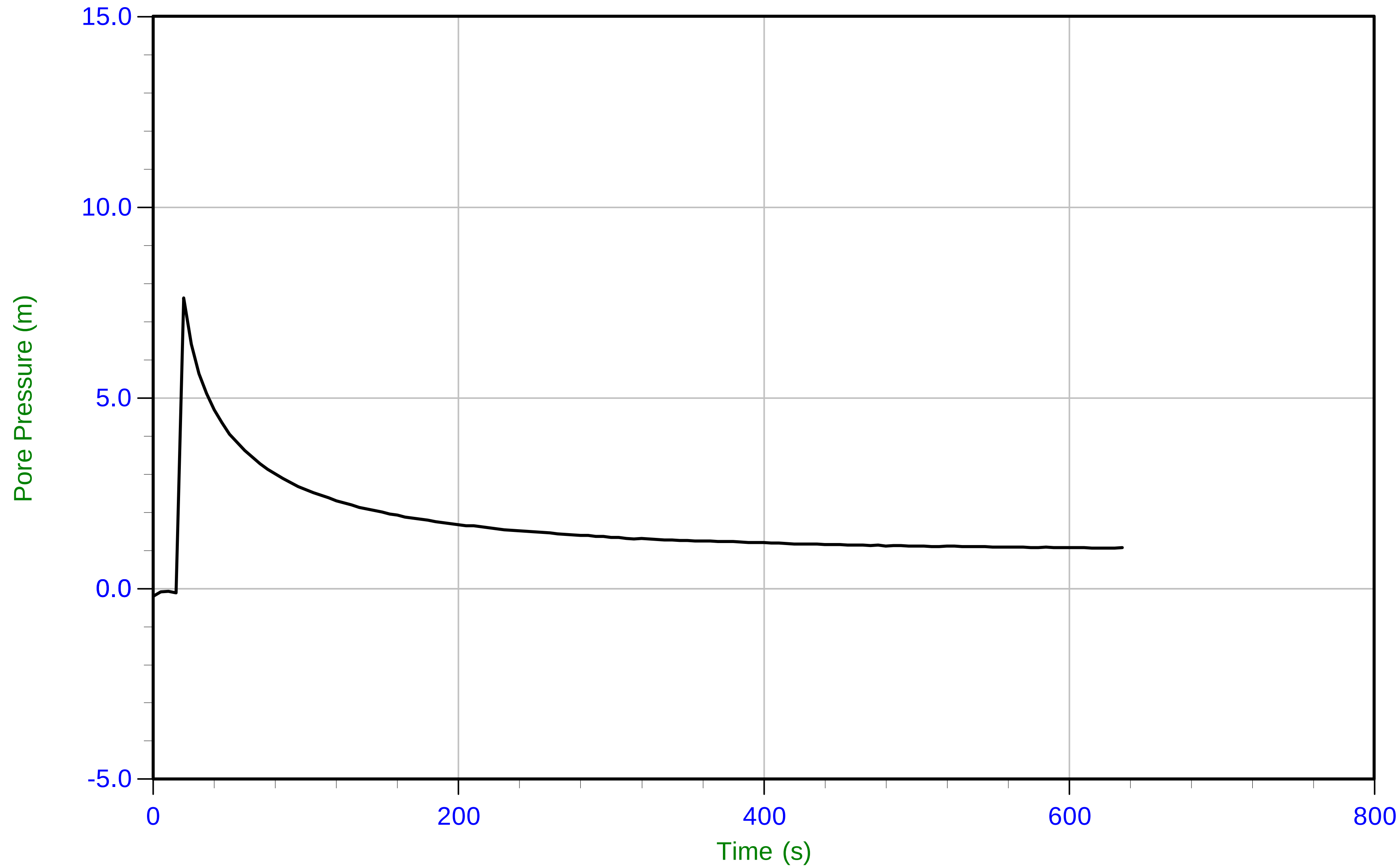
WT: 3.169 m / 10.396 ft
Ueq: 1.1 m

ConeTec

Golder Associates

Job No: 19-05025
Date: 04/25/2019 07:36
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-11
Cone: 549:T1500F15U500 Area=15 cm²

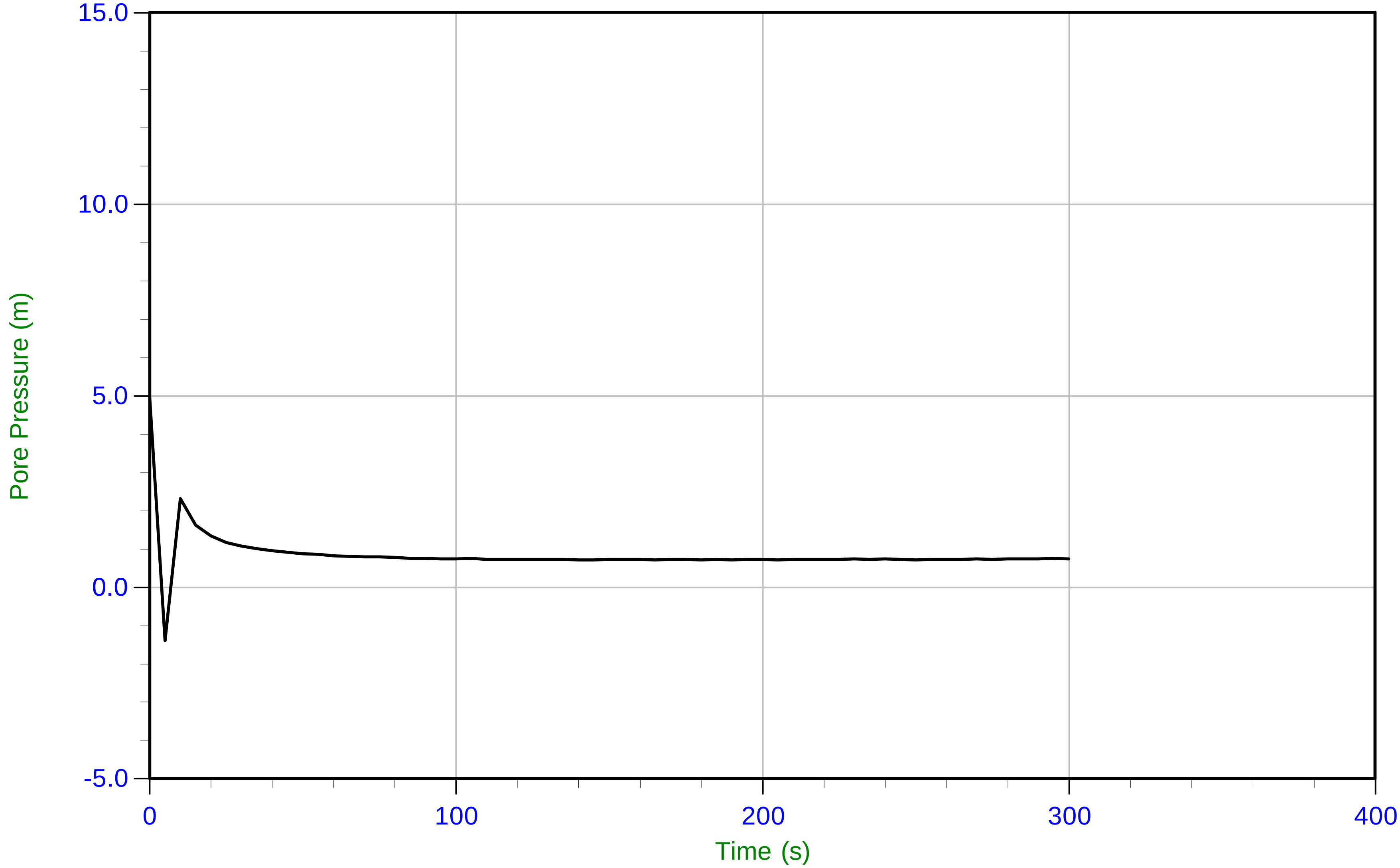


Trace Summary:

Filename: 19-05025_CP11.PPF
Depth: 1.950 m / 6.398 ft
Duration: 635.0 s

u Min: -0.2 m
u Max: 7.6 m
u Final: 1.1 m

Golder Associates	Job No: 19-05025	Sounding: CPT19-11	
	Date: 04/25/2019 07:36	Cone: 549:T1500F15U500	Area=15 cm²
	Site: Gananoque Truck Inspection 401 Hwy Eastbound		



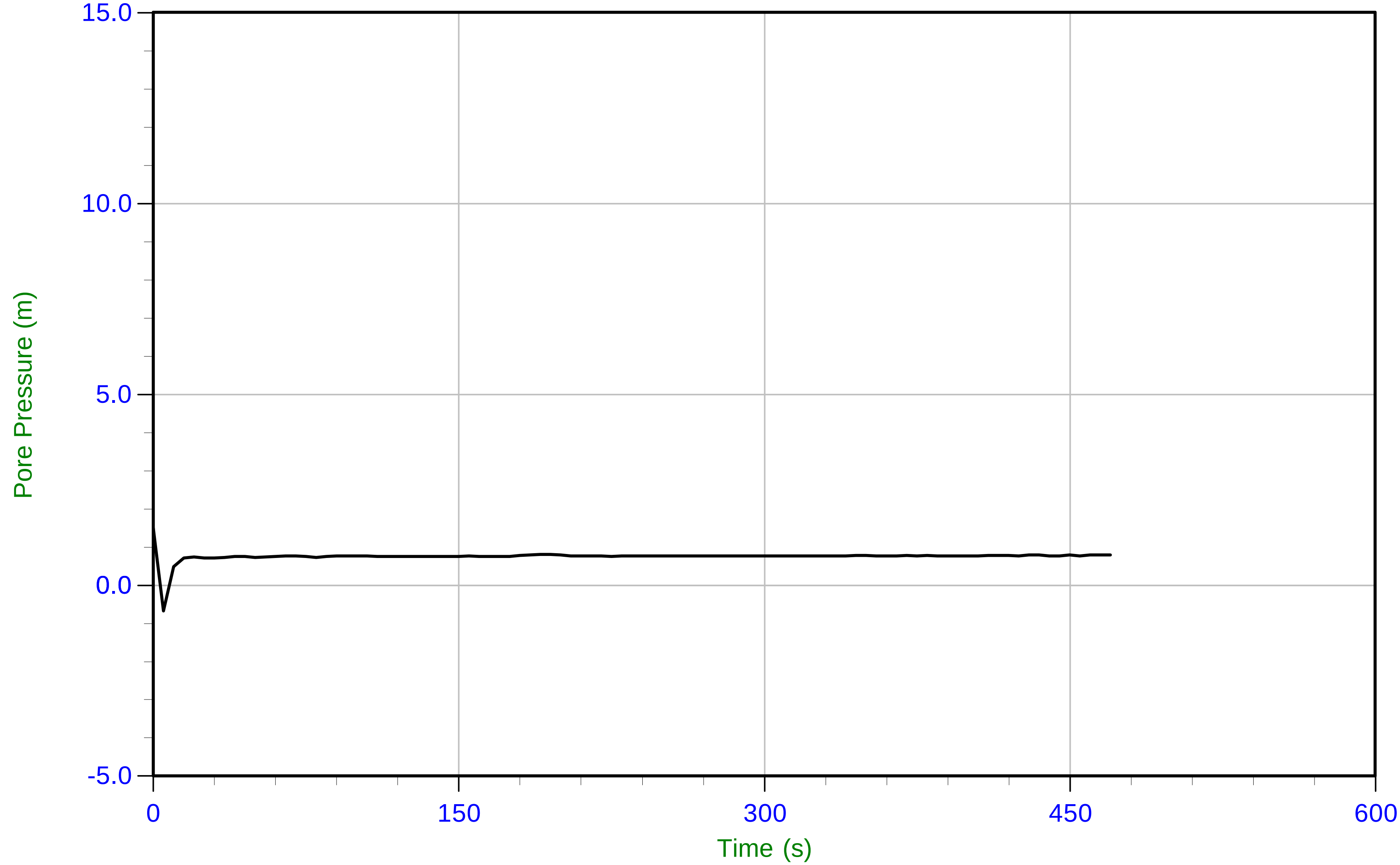
Trace Summary:

Filename: 19-05025_CP11.PPF	u Min: -1.4 m	WT: 3.386 m / 11.110 ft
Depth: 4.125 m / 13.533 ft	u Max: 4.9 m	Ueq: 0.7 m
Duration: 300.0 s	u Final: 0.7 m	

Golder Associates

Job No: 19-05025
Date: 04/25/2019 07:36
Site: Gananoque Truck Inspection 401 Hwy Eastbound

Sounding: CPT19-11
Cone: 549:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-05025_CP11.PPF
Depth: 4.150 m / 13.615 ft
Duration: 470.0 s

u Min: -0.7 m
u Max: 1.5 m
u Final: 0.8 m

WT: 3.368 m / 11.051 ft
Ueq: 0.8 m

APPENDIX E

Operational Constraints, Notices to Contractor and Non-Standard Special Provisions

GOLDER

EMBANKMENT CONSTRUCTION IN PRELOAD AND SURCHARGE AREAS

Operational Constraint

The Contractor shall confirm that the elevation of the top of the preload is within 150 mm of the design top of preload. Elevations shall be provided to the Contract Administrator within five (5) working days of placement of the preload. The Contractor shall keep records of the thickness of each layer of fill placed and provide these records to the Contract Administrator within five (5) working days of reaching the top of each layer.

After the subgrade has been properly prepared and all organics and softened/loosened material has been removed, fill placement may proceed to the preload level. The preload shall remain in place for a minimum of 1 month prior to construction of spread/strip footings.

Control of Overburden Soils for Overhead Sign Foundation – Item No.

Notice to Contractor

The Contractor shall be alerted that the overburden soils at the overhead sign support at Station 11+500 consist of cohesionless and potential water-bearing sands, which are susceptible to sloughing, boiling or caving into the excavation unless appropriate groundwater controls are in place for caisson construction. The Contractor is to design and install an appropriate excavation protection system (e.g. temporary liners, drilling fluids) and an unwatering system as may be required to provide for both side wall and basal stability of the soils during foundation construction, and place concrete by tremie methods as may be appropriate.

Obstructions – Item No.

Notice to Contractor

The Contactor shall be alerted to the presence of a steel obstruction approximately 1 m south of Borehole 19-9B in the area of the proposed inspection canopy and bays. This obstruction consists of buried 5.2 m long hollow stem augers due to abandonment of original borehole at this location.

The Contractor shall also be alerted that shallow refusal was encountered at the location of CPTs 19-10 and 19-10B near the location CPT 19-10C in the area of the proposed CVIF building. Refusal to cone penetration was encountered at the location of CPTs 19-10 and 19-10B at 2.3 m and 5.0 m depths, respectively.

Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations for the foundations.

DECOMMISSION OF PIEZOMETERS - Item No.

Non-Standard Special Provision

A standpipe piezometer was installed in Borehole 19-4 as part of the Foundation Investigation for the Gananoque South Commercial Vehicle Inspection Facility. The standpipe piezometer installed as part of the Foundation Investigation is listed below; additional information regarding installation details and location are found within the contract documents and the Foundation Investigation Report.

Standpipe Piezometer Identification	Approximate Location		PVC Pipe and Screen diameter / Borehole diameter	Depth (Below Ground Surface) to Tip of Screen
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
19-4	4,913,440.9 (44.360214)	338,218.3 (-79.080763)	50 mm / 216 mm	4.5 m

The standpipe piezometer is registered as Well Tag Number A269601. The registered owner is the Ministry of Transportation, Ontario.

The standpipe piezometer has been left in place to allow for monitoring of groundwater levels up to construction.

As part of the construction activities the contractor shall properly decommission the standpipe piezometer prior to the start of the construction works. The abandonment method for standpipe piezometer must satisfy the minimum requirements of Ontario Regulation 903 Wells, as amended under the Ontario Water Resources Act. In addition, the contractor shall provide a written record of the decommissioning procedure to the Contract Administrator. The record shall include plugging material used, depth of plugging material and limit of the PVC standpipe/screen removal.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

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